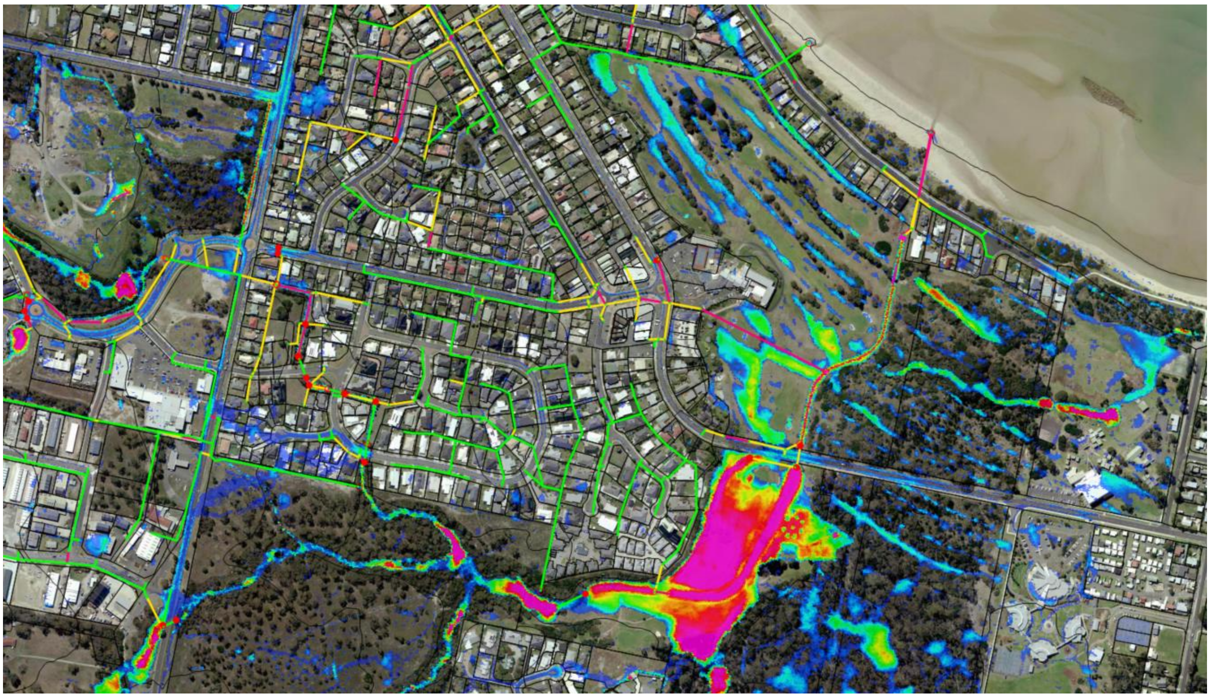
CATCHMENT ASSESSMENT STAGE 1 REPORT

SHEARWATER



SEPTEMBER 2023

HYDRODYNAMICA
44 PENQUITE ROAD LAUNCESTON TAS 7250
T 04312 08450 E cameron.oakley@h-dna.com.au

Project: Shearwater Stormwater Modelling Report

Authors: Cameron Oakley
Consulting Engineer
BEng (Hons), BTech (Env.), MBA
Building services provider No. 949718126

DATE	NATURE OF REVISION	REVIEWED	APPROVED	REVISION NUMBER
23/03/2023	Draft for comment	JM		0.1
12/04/2023	Revised draft for comment	JO		0.2
14/07/2023	Final draft	JO/JM		0.3
12/09/2023	FINAL	JO/JM	JM	1

The cover page presents an extract of a 10% AEP flood depth map.

CONTENTS

1. CONTEXT

Latrobe Council (LC) engaged Hydrodynamica to assess the behaviour of the Shearwater catchment in order to provide a better understanding of overland flow paths within the catchment, flood impacts in the lower parts of the catchment, and the capacity of the existing drainage system to accept additional flows from the upper catchment. New residential subdivisions are planned for this area.

Port Sorell Golf Club, located in the lower catchment adjacent to Shearwater Esplanade and Freers Beach, has been impacted by historical flooding and there is particular concern that increased development upstream will worsen flooding into the future. The Latrobe Council Stormwater System Management Plan V0.1 highlights the following concerns in the catchment:

Catchment	Description	Flood Model	Flood Issues	Infrastructure and Infrastructure Limitations	Potential Improvements for management plan
Freers Beach	Outfalls of varying size including Poyston Creek, Freer Street, Anderson Street, Shearwater Park (north and south) and surf club.	Modelling by CSE Tasmania using drains and subsequent report is complete.	Golf course subject to flooding and loss of play time.	Sea level rise, storm surge, sand build up across faces of outfalls are known issues. Poyston Creek outfall onto Freers Beach duplicated in 2018/19 financial year. High levels of sand may require reworking and / or flap valves. Poyston Creek crossing of Pitcairn St to be monitored.	Break into sub-catchments and adjacent smaller outfalls. Commencing with largest outfall initially after prioritising any emergent hot-spots.

Table 1. Extract from S7 of Latrobe Council Stormwater System Management Plan V0.1

Flooding of the golf course has recently come under scrutiny due to a high rainfall event which coincided with a high tide, when the outlet to Freers Beach was submerged. It is understood that surcharging occurred, from the beach back up the outlet pipes and into the golf course, causing flooding to be exacerbated.

The focus of this report is to define the mechanisms contributing to, and the extent of flooding within the catchment. It is also to provide recommendations as to how to reduce such flooding and how future development, including how the proposed residential development west of Burgess Drive, can potentially contribute to improvement.

2. THE CATCHMENT

The overall Shearwater catchment is presented in Figure 1. The stormwater system has several outlets to Freers Beach, however for this assessment the subcatchments only draining through the golf course were considered. The eastern-most outlet comprises of twin DN900 pipes, and the western outlet consists of a single DN1050. The 550 hectare modelled area is shown in Figure 2.

Land use consists mainly of general residential urban lots to the east of the catchment, except for the golf course, refer to Figure 3.

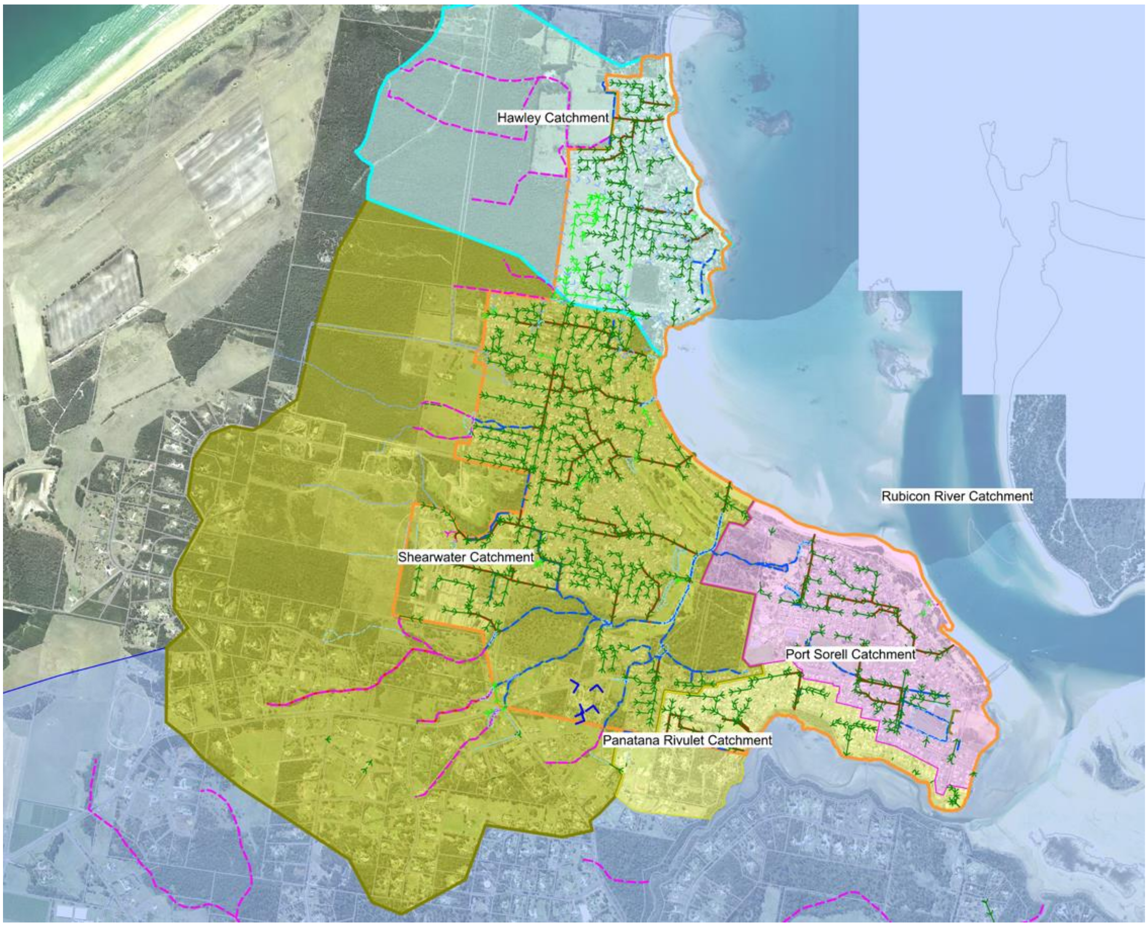


Figure 1. Shearwater catchments (provided by Latrobe Council)

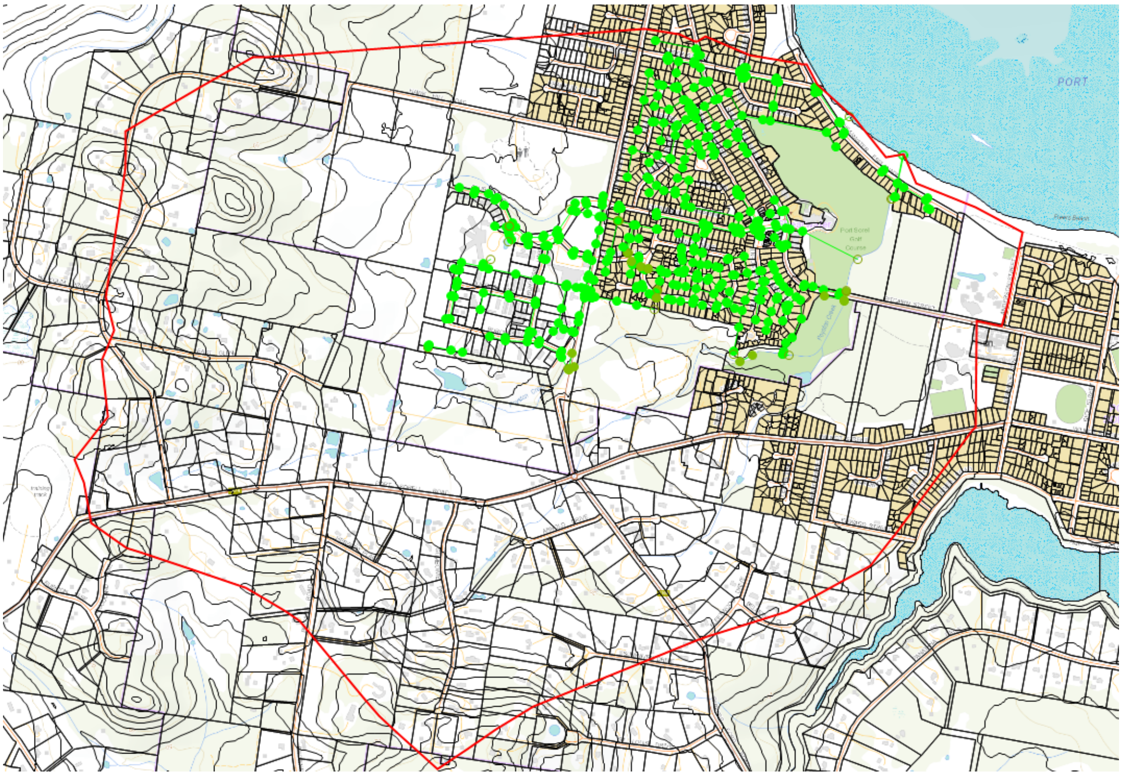


Figure 2. Modelled catchment

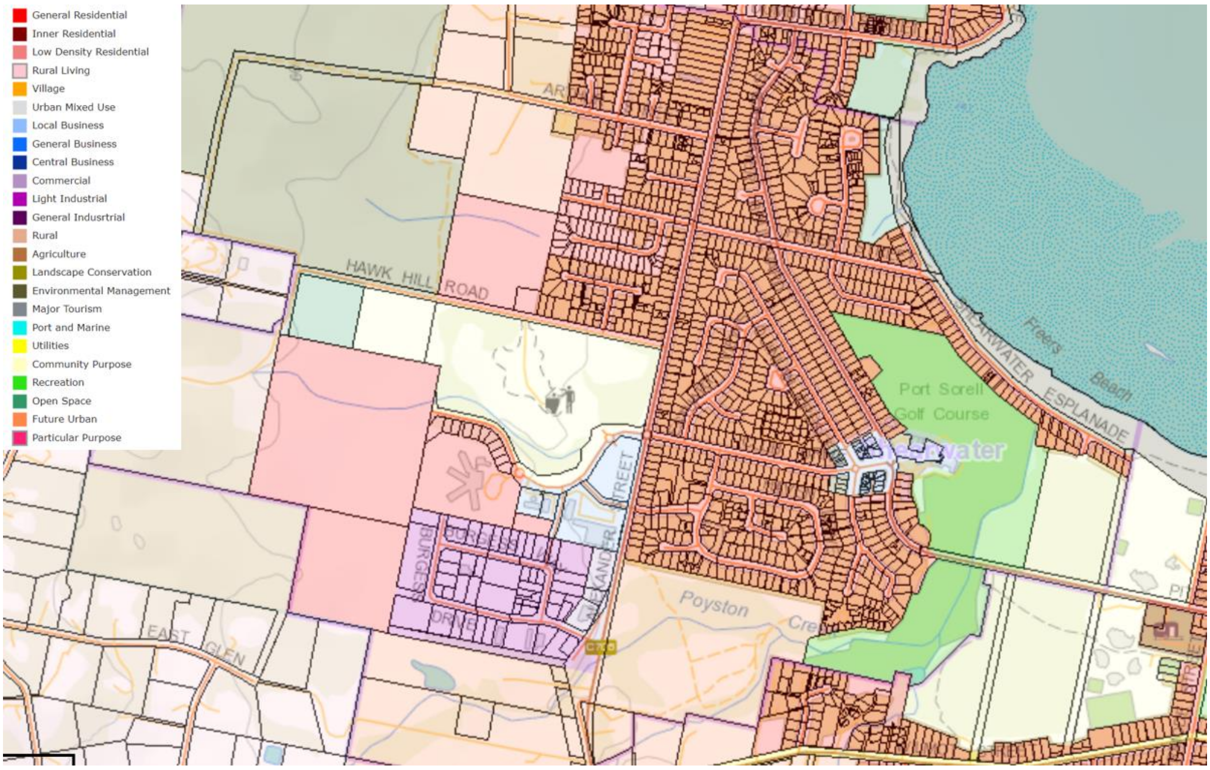


Figure 3. Latrobe Local provisions Schedule Zoning – source: http://maps.thelist.tas.gov.au

There are centrally located Local Business and Light Industrial areas in the centre, and General Residential, Rural Living and Environmental Management in the upper catchment. The central General Residential area includes the Rubicon Grove OneCare facility, and an area of upcoming residential subdivision is located west and north-west of Burgess Drive.

3. MODELLING INFORMATION

1D-2D modelling of the catchment was undertaken using Infoworks ICM in order to help understand the performance of the stormwater system, particularly in the area of Burgess Drive and at the Port Sorell Golf Course. This will help identify what stormwater controls are necessary to enable potential development to proceed, and help to provide further insight into how inundation of the lower catchment can be mitigated.

Infoworks ICM allows the integration of 1D (pipes and pits) and 2D (overland flow) simulations. The urban area was modelled by using a mixture of 1D rainfall directed straight to pipes and pits, and 2D direct rain-on-grid modelling. The upper largely undeveloped catchment and the golf course were modelled using rain-on-grid.

Further information about the model's hydrological and hydraulic settings are provided in Appendix A. Resultant flood maps are provided in Appendix B.

3.1 ASSET DATA

Pipe and pit/manhole data for the subcatchment downstream of Burgess Drive was surveyed and provided by Esk Mapping. Asset data for the subcatchment in the vicinity of Freer Street was provided by Latrobe Council. The modelled pipes and pits are shown in green in Figure 3.

Not all manholes were able to be found or opened, and there was some other missing data. The process of model validation identified the following issues with some of the assets:

- Missing surface levels (SLs) and invert levels (ILs) for pits and manholes;
- Missing ILs for pipes and culverts; and
- Upstream pipe and culvert invert levels which were lower than downstream invert levels.

When necessary, culvert invert, manhole and pit surface level data was added by inferring levels from the 2013 1m digital elevation model (DEM) obtained from ELVIS (https://elevation.fsdf.org.au/). Other missing data was assumed based on engineering judgement.

Where initially missing data resulted in potential flood situations not consistent with the catchment further detail was obtained. Missing data was minor in nature and is not expected to tangibly change the modelling results.

3.2 INITIAL & BOUNDARY CONDITIONS

Freers Beach is located approximately halfway between Devonport and Low Head. The tide datums for these two ports, and inferred datums for Freers Beach, are presented in Table 2.

Table 2 gives the Lowest Astronomical Tide (LAT), Mean Sea Level (MSL), and the Highest Astronomical Tide (HAT) in m AHD:

Port	LAT	MSL	HAT
Devonport	-1.97	-0.02	1.68
Low Head	-2.02	-0.03	1.62
Freers Beach (inferred)	-2.00	-0.03	1.65

Table 2. Tide datums (source nre.tas.gov.au)

The tide levels above do not include storm surge.

Latrobe Council technical staff have provided anecdotal evidence of high tides in excess of 1.8m AHD at Freers Beach. Monthly tidal data from Devonport and Low Head was obtained from the Bureau of Meteorology (http://www.bom.gov.au/oceanography/projects/ntc/monthly/) for between 2013 and 2020. This suggested monthly average minimum low and maximum high tides similar to the LAT and HAT levels provided by NRE Tasmania. For example, at Devonport the average minimum monthly low tide was approximately -1.8m AHD, and the average maximum monthly high tide was 1.66m AHD. For modelling purposes these tidal extremes were adopted for some of the modelled scenarios.

The invert level of the twin DN900 pipe outlets, which discharge to Freers Beach in the target subcatchment, is 0.35m AHD. The invert level provided for the DN1050 outlet at Freers Beach near the northern end of the golf course is 0.21m AHD. If these levels are accurate, the outlets are only 200-300mm above MSL. High tides will routinely be affecting the outlet and often completely submerging it.

The DN900 inlets at the north-eastern corner of the golf course have inferred invert levels of 1.17m AHD. If this level is correct, it is likely that tidal water will routinely make its way up the pipes to the inlets in the golf course. This aligns with Council officer observations. When significant runoff in the catchment is coincident with an incoming tide, additional flooding of the golf course will likely occur.

Latrobe Council officers have also noted that both sets of outlets routinely have some level of blockage from sand. Normal tidal actions will redeposit sand in and around the outlets during the next successive high tides after the outlets have been cleared. An operation issue is that low flows may only use one pipe and the other stays blocked.

An initial round of simulations of the Burgess Drive subcatchment were undertaken (Scenarios 1 to 3) assuming free unrestricted discharge to Freers Beach, i.e., no blockages or tidal action on the outlets. Flows peaked from the twin DN900s during the 1% AEP 6 hour storm duration, although the 4.5 hour and 9 hour duration results were similar. This indicated that flooding at this location of the golf course would be the worst during this duration event due to the higher flood depths driving more stormwater through the outlets. This duration of storm event was convenient for adoption with the various modelling scenarios as 6 hours is also the approximately the time between a high and low tide. The list of scenarios modelled for the adopted peak storm were:

1. No downstream boundary conditions
2. Low tide at the beginning of the storm, no blockages (very close to peak flows reaching the golf course and tidal peaks coinciding)
3. High tide at the beginning of the storm, no blockages (very close to peak flows reaching the golf course and tidal lows coinciding)
4. Both sets of outlets from golf course 100% blocked

Flood mapping of these scenarios is provided in Appendix B.

There are many small farm dams in the upper catchment. Empty dams at the start of the simulations would act to detain runoff and reduce peak flows received by the lower catchment. The dams were therefore ‘pre-filled’ before the start of the simulations.

3.3 RAINFALL & CLIMATE CHANGE

Climate change related sea level rise will reduce the effectiveness of existing drainage infrastructure in this area. The Tasmanian Local Council Sea Level Rise Planning Allowances (refer to https://www.dpac.tas.gov.au/__data/assets/pdf_file/0017/64421/Local_Council_Sea_Level_Rise_Planning_Allowances_derived_from_RCP_8.5.pdf) give a planning allowance of 0.82m for George Town, and 0.81m for Devonport. This indicates that by the year 2100 the MSL may cover half the height of the outlets to Freers Beach. The Tasmanian Coastal Adaptation Pathways Project – Port Sorell Report (SGS Economics & Planning, 2012) provides mapping of the likely inundation for a 1% AEP storm surge event and a 0.9m sea level rise, refer to Figure 4:

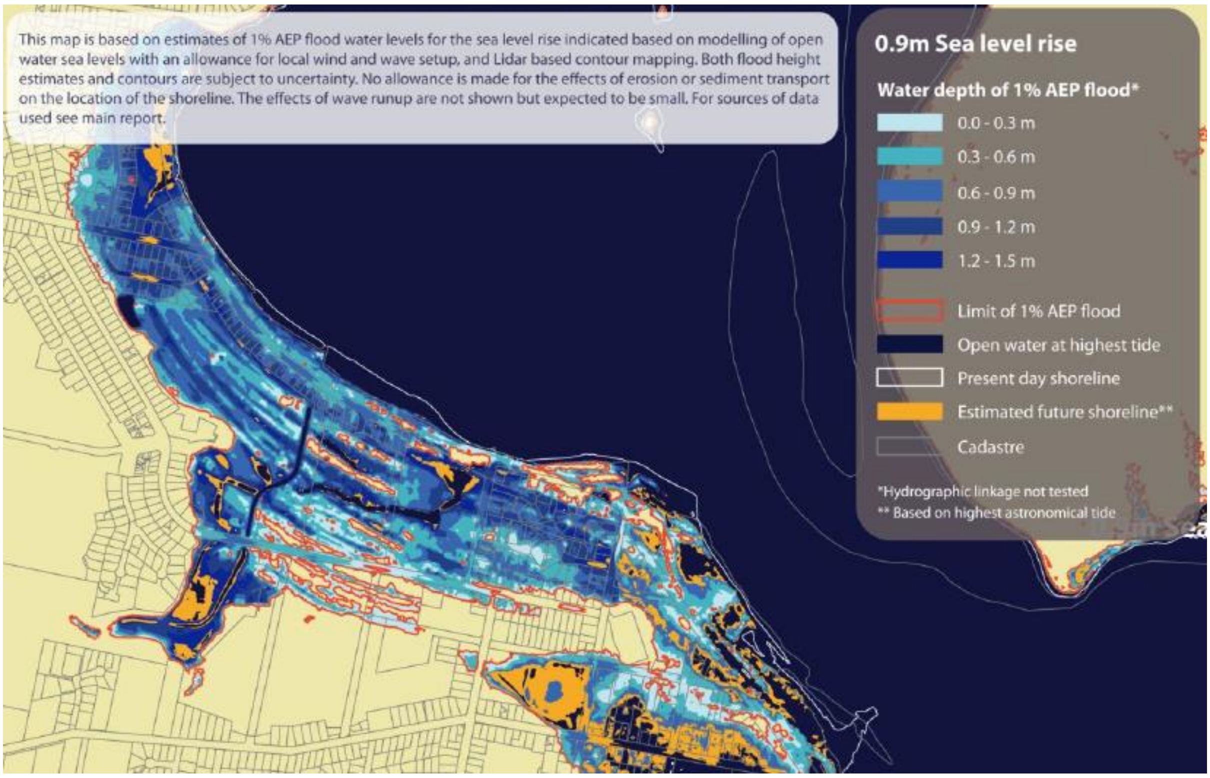


Figure 4. Likely inundation for an extreme (1% AEP) storm surge event inclusive of 0.9m sea level rise

The Tasmanian Coastal Adaptation Pathways Project Report gives indicative inundation levels of 3.25m, 3.7m and 3.8m for 10 the year, 50 year and 100 year ARIs as a result of coastal inundation. It is noted that these forecasts are from coastal inundation alone, and they have not considered any coinciding contributions to flooding from runoff generated in the upstream catchment.

In addition to sea level rise and storm surge, increasing rainfall intensities due to climate change are predicted. Increased rainfall intensities will further increase the risks of inundation for all low lying areas into the future.

In order to provide a base line of information that will be of benefit to Latrobe Council and the local community at the present day, it was agreed that climate change rainfall, sea level rise, and storm surge would not be included in the Stage 1 modelling, as this would not provide a determination of the current performance of the existing stormwater system below the Poyston Creek crossing of Pitcairn Street. Stage 2 modelling and solutions will consider the impacts of climate change rainfall, sea level rise, and storm surge in order to further assess and future proof the infrastructure projects which are further investigated.

Stage 1 modelling was undertaken using the 2016 Bureau of Meteorology (BOM) Design Rainfall System (and ARR Data Hub temporal pattern ensembles. Due to the size of the model and lengthy run times, half of the recommended 10 temporal patterns which made up the ensembles were run. These include a range of front loaded, mid-loaded and back-loaded patterns. Median pre-burst rainfall depths were applied to the storm bursts.

4. DISPLAYED MODELLING RESULTS INFORMATION

The modelling results presented in this document show the 1D (pipe and node) and 2D (above ground) flooding and surcharge. Displayed 1D pipe results are colour coded as follows:

- **Green** pipes have not surcharged. The water level is below the soffit level at both ends of the pipe;
- **Yellow** pipes have surcharged. The water level at the upstream and/or downstream end of the pipe is above the soffit level. The flow is less than or equal to the pipe's full capacity;
- **Magenta** pipes have surcharged. The water level at the upstream and/or downstream end of the pipe is above the soffit level. The flow is greater than or equal to the pipe's full capacity;

Longer duration storms lead to a greater inundation of downstream catchment than do short duration events, which are likely more critical for adjacent pipe and pit infrastructure. Given the focus is peak inundation in the downstream catchment flood mapping does not show peak surcharge states for all pipes and pits.

Displayed 2D flood depth results are generally categorised as per Table 3:

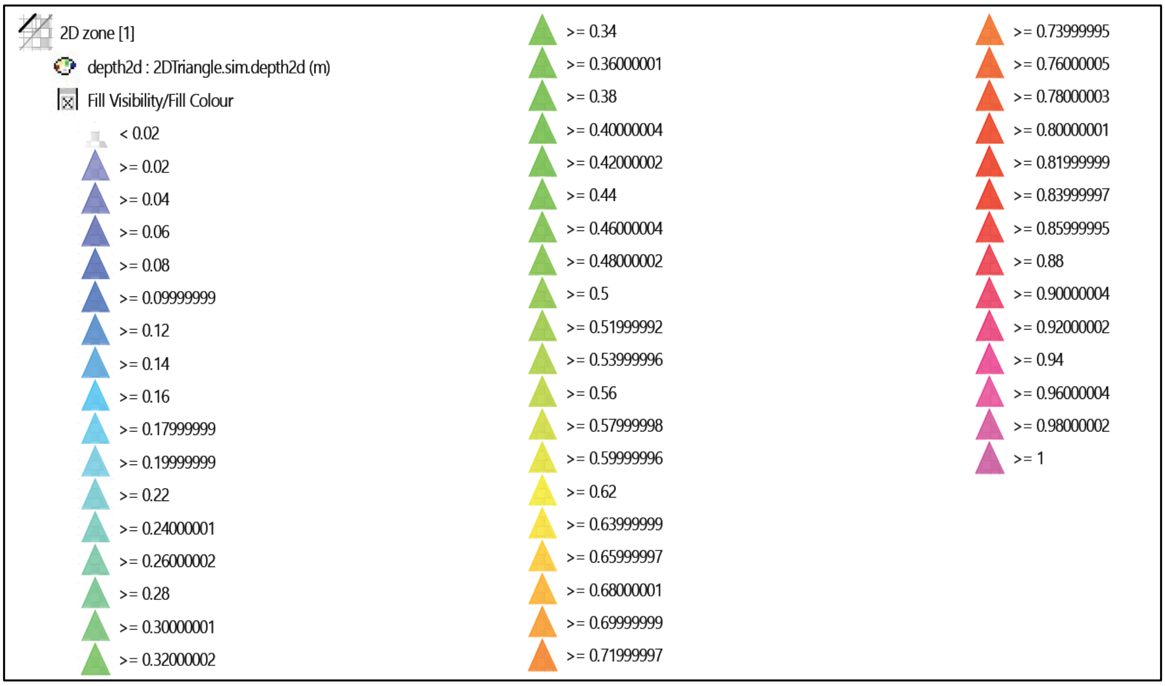


Table 3. 2D (overland) flood depth results categories (metres)

Hazard Vulnerability Categories, which relate to the risk of flooding and are a function of depth and velocity, are defined in Australian Rainfall and Runoff 2019 (ARR 2019) and have been reproduced in Figure 5.

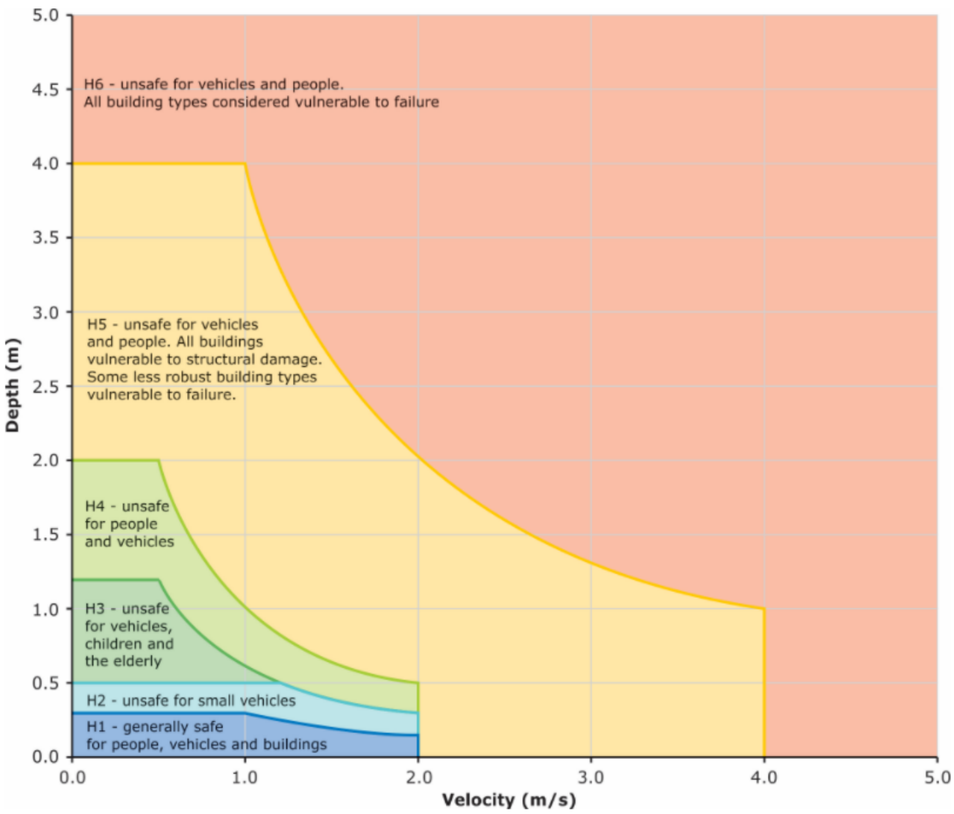


Figure 5. ARR 2019 Flood Hazard Categories (ref. ARR 2019 Book 6, Chapter 7)

The ARR 2019 hazard vulnerability classifications are a function of velocity and depth. Refer to Table 4:

Hazard Vulnerability Classification	Classification Limit (D and V in combination)	Limiting Still Water Depth (D)	Limiting Velocity (V)
H1	D*V ≤ 0.3	0.3	2.0
H2	D*V ≤ 0.6	0.5	2.0
H3	D*V ≤ 0.6	1.2	2.0
H4	D*V ≤ 1.0	2.0	2.0
H5	D*V ≤ 4.0	4.0	4.0
H6	D*V > 4.0	-	-

Table 4. Hazard Vulnerability Limits (from *ARR 2019*)

In this report the *ARR 2019* hazard vulnerability classifications have been displayed as follows:

H1
H2
H3
H4
H5
H6

Table 5. Displayed Flood Hazard Vulnerability Categories

5. STORMWATER SYSTEM ASSESSMENT

This document of should be read while viewing the 1% AEP flood maps provided in Appendix B. These maps show the following modelled scenarios:

1. No downstream boundary conditions
2. Low tide at the beginning of the storm, no blockages
3. High tide at the beginning of the storm, no blockages
4. Both sets of outlets from golf course 100% blocked

The maps have a high resolution and best viewed in pdf format.

5.1 PORT SORELL GOLF CLUB

As expected, Scenario 1 (no downstream boundary conditions) shows the least inundation within the golf course. Even in this ‘best case’ scenario, however, there is extensive inundation of the golf course during the 1% AEP event. A 10% AEP flood map has also been provided for this scenario for comparison in Appendix B.

Flooding tends to accumulate naturally into the undulations or gullies both west and east of the main drainage line and parallel to the beach. These dips and ridges may be remnants of a dune system. Before the main drainage line fills some shallow flooding in these gullies occurs. Once the main drainage line fills, however, significant volumes of floodwater spills into these lateral gullies. The gullies carry runoff which is not captured by the twin DN900 pipes. The flow arrows in Figure 6 show the direction of some of these flows, the most significant lateral flow paths are labelled.

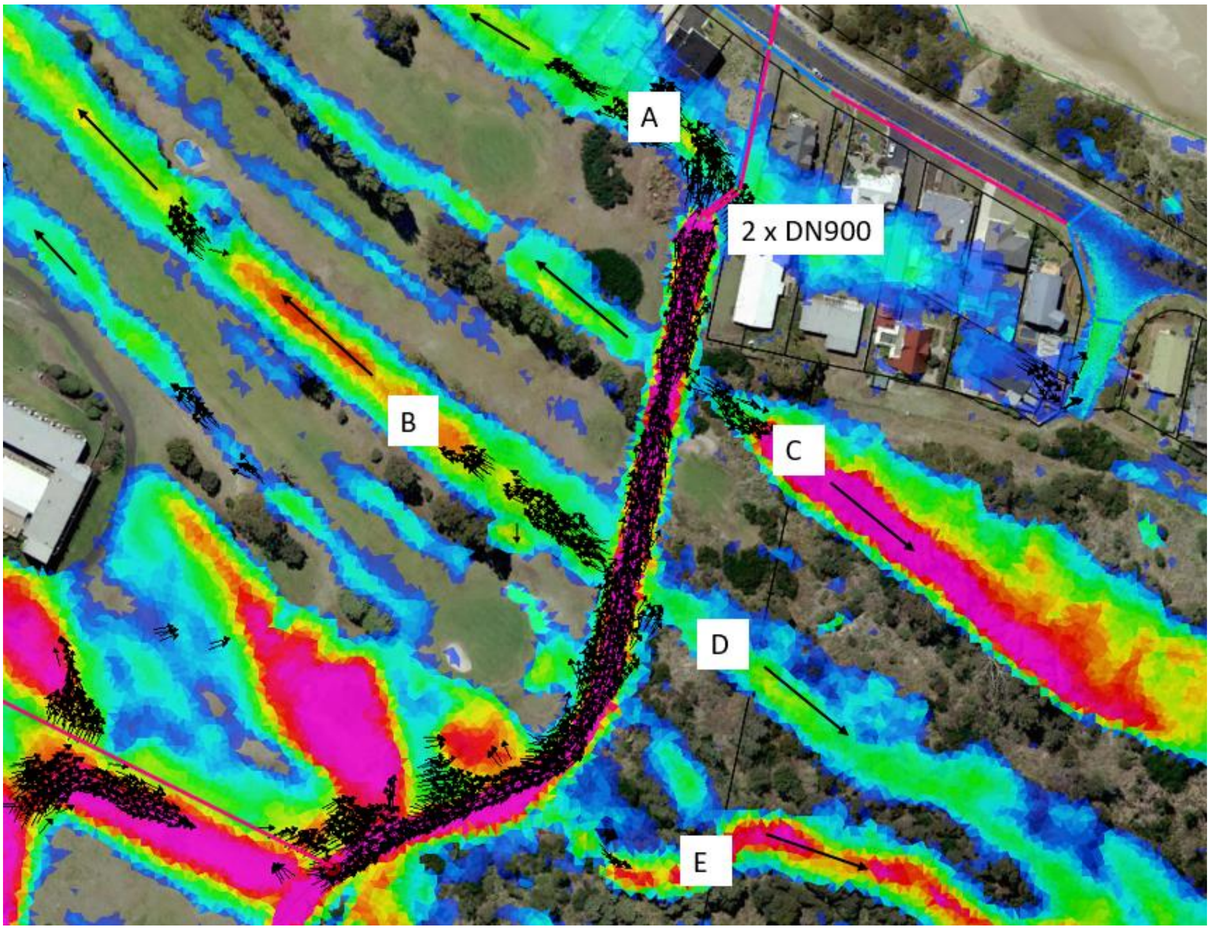


Figure 6. Lateral flooding from main drainage channel

Flow paths A and B move north-west and floods Seabreeze Avenue and Freer Street. Over time the road pits in these two streets removes the floodwater, with the remainder of overland flooding moving north-west over the edge of the model. According to recent survey there are no Council owned inlets or pits within this north-western section of the golf course, apart from a small detention system at the very north-western corner of the golf course.

Flow paths C, D and E similarly pass significant flows eastwards to Camp Banksia where they accumulate at the northern end of that property before spilling to Anderson Street and off the edge of the model.

North of Pitcairn Street Scenario 2 flooding (low tide at the beginning of the storm, no blockages) is slightly worse than Scenario 3 (high tide at the beginning of the storm, no blockages). This is due to the timing of the high-tide relative to the peak flows reaching the golf course. Examining at the peak flows and volumes through the dual DN900 outlet pipes to the beach during Scenarios 2 and 3, there is a reduction in Scenario 2 due to the timing of the tide coinciding with the high tide.

The outflow hydrographs presented in Figure 7 are for the dual DN900 outlets discharging to Freers Beach during Scenarios 1 (no tidal influence), Scenario 2 (low tide at t=0) and Scenario 3 (high tide at t=0) respectively.

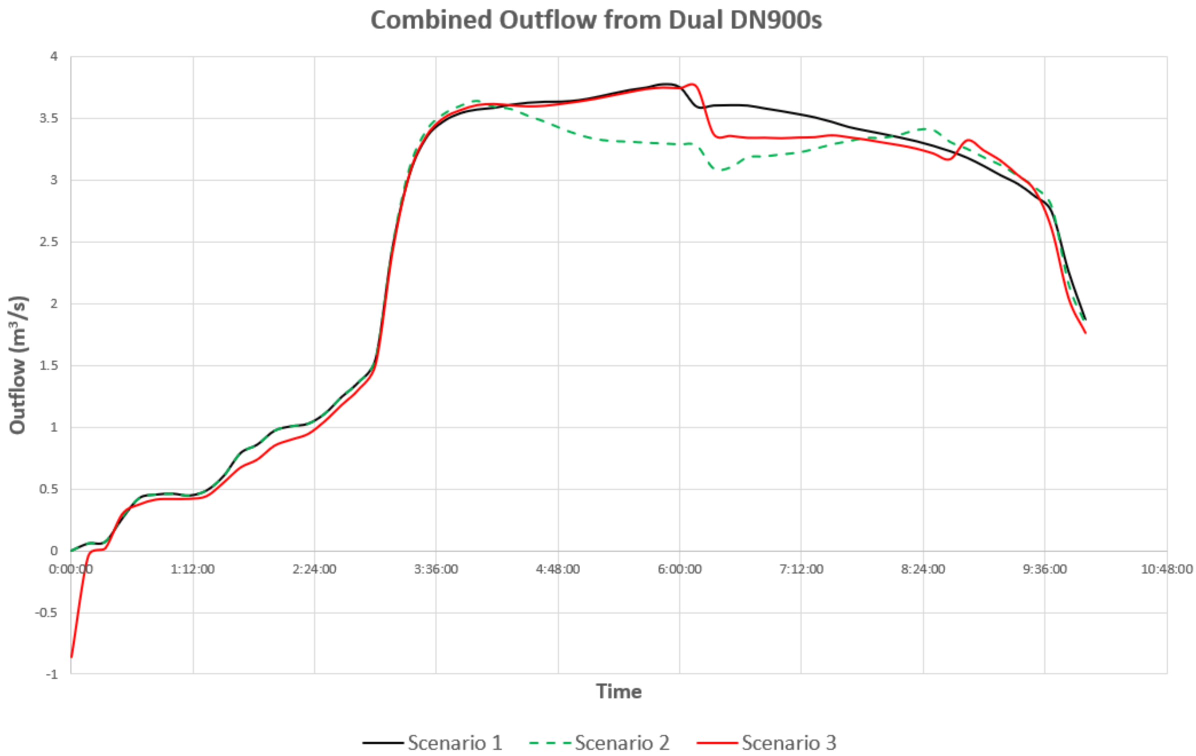


Figure 7. Outflow hydrographs from dual DN900 outlets

Over the duration of the storm event this decrease in flow equates to a reduction of 1.43 ML outflowing to the beach in Scenario 2 compared to Scenario 3. It is noted there is a backflow volume up and out of the pipes and into the golf course at the start of Scenario 3 due to the initial high tide.

The greatest changes in flood depth can be seen at the north-western end of the golf course. Surcharge escaping the drainage line fills the pond associated with the detention arrangement south of no. 22 Seabreeze Avenue, and in Seabreeze Avenue itself.

Scenario 4 (high tide, both sets of outlets 100% blocked) is obviously the worst of all the scenarios though interesting not to an extreme level. Assuming the golf course outlets have no capacity means that all flows must pass laterally from the main drainage line before escaping to the edges of the model.

All results show significant flooding upstream of Pitcairn Street, eventually leading to overtopping of the road itself. The capacity of the 4 no. DN600 pipes under the road is not fully utilised, probably due to submerged tailwater conditions. Based on the survey information for these pipes, the model suggests they have a total 'just full' capacity of 1.88 m^3/s. During the simulations a peak flow of only 1.21 m^3/s is achieved. This compares to peak over road flooding of between 8.5 and 8.8 m^3/s in the four scenarios. In comparison the twin DN900 outlet pipes have 'just full' capacities of just over 1 m^3/s each, but due to the headwater acting on the system they pass about 1.9 m^3/s each when there are no blockages and they can discharge freely to the beach without tidal influence.

Appendix B provides a Flood Hazard map for Scenario 2. Although there is considerable inundation of the golf course the most significant areas in terms of risk are immediately up and downstream of Pitcairn Street. These areas are generally categories H3 and H4, refer to Section 4 for descriptions, which is flooding between 0.5m and 2m deep. The Shearwater Resort and Port Sorrell Golf Club rooms appear safe from flooding from within the golf course during the 1% AEP. The club rooms may, however, be impacted by shallow flooding caused by overtopping of the road in Shearwater Boulevard in all scenarios.

The most severe risk in terms of people and property occurs along Shearwater Esplanade and Freer Street residences due to lateral flooding out of the main drainage channel. The associated risks range from H1 to H3 and affects all properties in Shearwater Esplanade, from Princess Court and Freer Street, and in Freer Street, from the Shearwater Esplanade to Seabreeze Avenue. Latrobe Council staff suggest that anecdotally real outcomes may not be as severe as the model predicts.

5.2 BURGESS DRIVE

The flood maps clearly show a natural overland flow path leading to a small dam adjacent to the north-eastern boundary of Lot 19 Burgess Street. Refer to Figure 8. When the dam overflows flooding moves through Lot 1 Burgess Drive.

Modelling shows overland flows passing into nos. 44 and 48 Burgess Drive and OneCare Limited Rubicon Grove. There is currently no formal connectivity from the natural overland flow path and dam to the public stormwater system. When the flows reach Burgess Drive some inlet capacity is provided by the sag pits in the road, but when these capacities are exceeded, the kerb overtops and there is no formal adjacent overland flowpath.

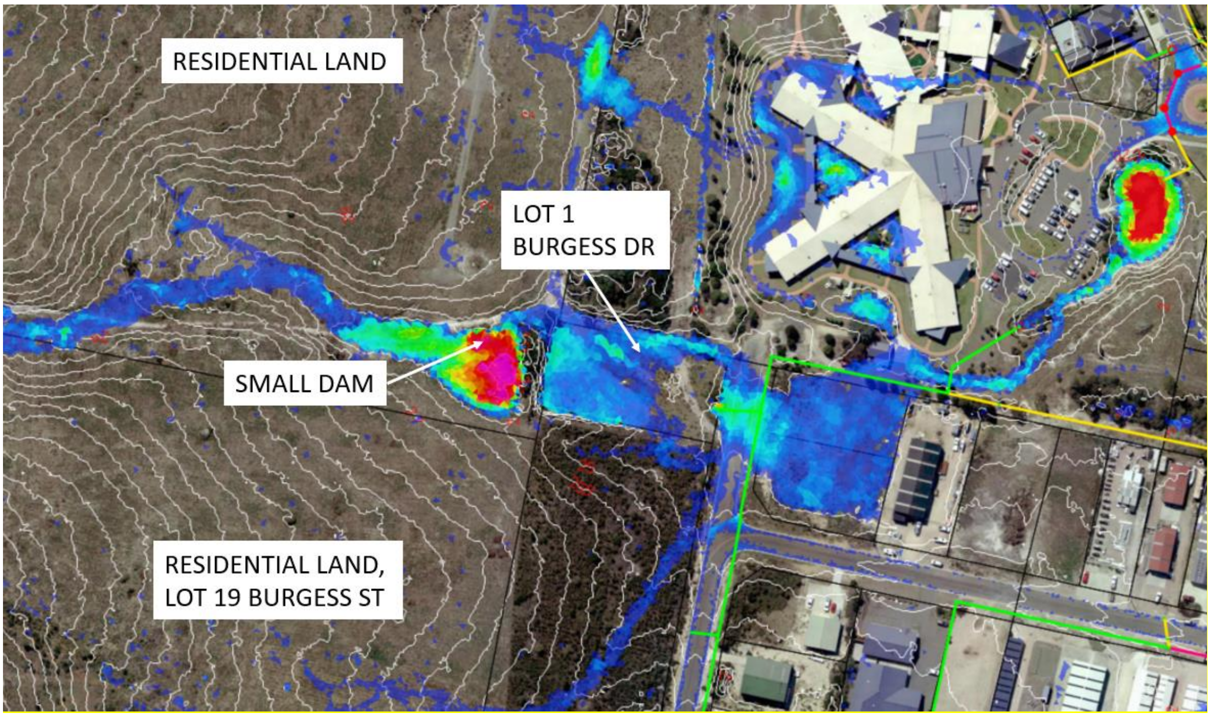


Figure 8. 5% AEP overland flow at Burgess Drive

Some of the capacity of the Burgess Drive minor stormwater system is already utilised by the developed industrial properties on Burgess Drive and Burgess Way, and further development within the industrial area is expected. Unrestricted post-development flows from proposed residential subdivision upstream of Burgess Drive will place additional pressure on this minor drainage system and, in turn, contribute additional flows into the golf course.

Initial modelling suggests that pre-developed current-day 10% and 5% AEPs flows concentrated and discharged by the dam would exceed the available capacity of the minor stormwater system immediately downstream of lot 1 Burgess Drive. Post-development flows may therefore need to be reduced to less that these pre-development levels.

6. DISCUSSION & RECOMMENDATIONS

Given the extent of the existing flood risks in the lower catchment, including to the golf club and adjacent residential properties, the potential impacts of any upstream development should be carefully considered. All development, whether it be a small subdivision, unit development, or a larger development such as the proposed subdivision west of Burgess Drive, has the potential to increase flooding downstream discreetly and cumulatively.

As time progresses the risks in the lower catchment are likely to worsen without any development, both as a result of sea level rise and as a result of increased rainfall intensity due to climate change. Any additional increase in the outlet capacity of the stormwater system through to the ocean may become counterproductive into future, allowing more backflows into the lower catchment. Stage 2 modelling and investigations will consider how projected sea level rise and increased rainfall intensities will impact the catchment and any proposed mitigation measures.

Overall, the modelled results are considered consistent with other urban areas which have expanded over decades and now rely on systems installed prior to growth and climate change predictions.

6.1 DOWNSTREAM CONTROLS

Although new or upsized outlets may help, the downstream constraints including tidal levels, projected sea level rise, blockages of outlets due to deposition of sand, and the location of properties on Shearwater Esplanade make it difficult 'solve' lower catchment flooding simply by adding extra pipes to cater for the full 1% AEP peak flows.

Larger culverts under Pitcairn Street alone would help reduce the flood footprint on the top side of the road, but that would cause greater flooding of the lower part of the golf course and present additional risk to downstream residential properties. Modelling shows that Pitcairn Street currently provides some protection of lower golf course through detention of the large flood upstream. More benefit of this mechanism appears to occur during 10% AEP, as shown in the Appendix B flood maps.

It may be possible to construct an overland flow bypass of the main drainage line eastwards, through Camp Banksia, and out to the sea. Historical photos provided by Council show that the original flow path was eastwards towards Anderson Street. Figure 9 shows the outfall in circa 1946. Some ponding within the dunes is also evident. The circa 1966 photo shows realignment of the open drain coinciding with the development of Shearwater Resort and the golf club.

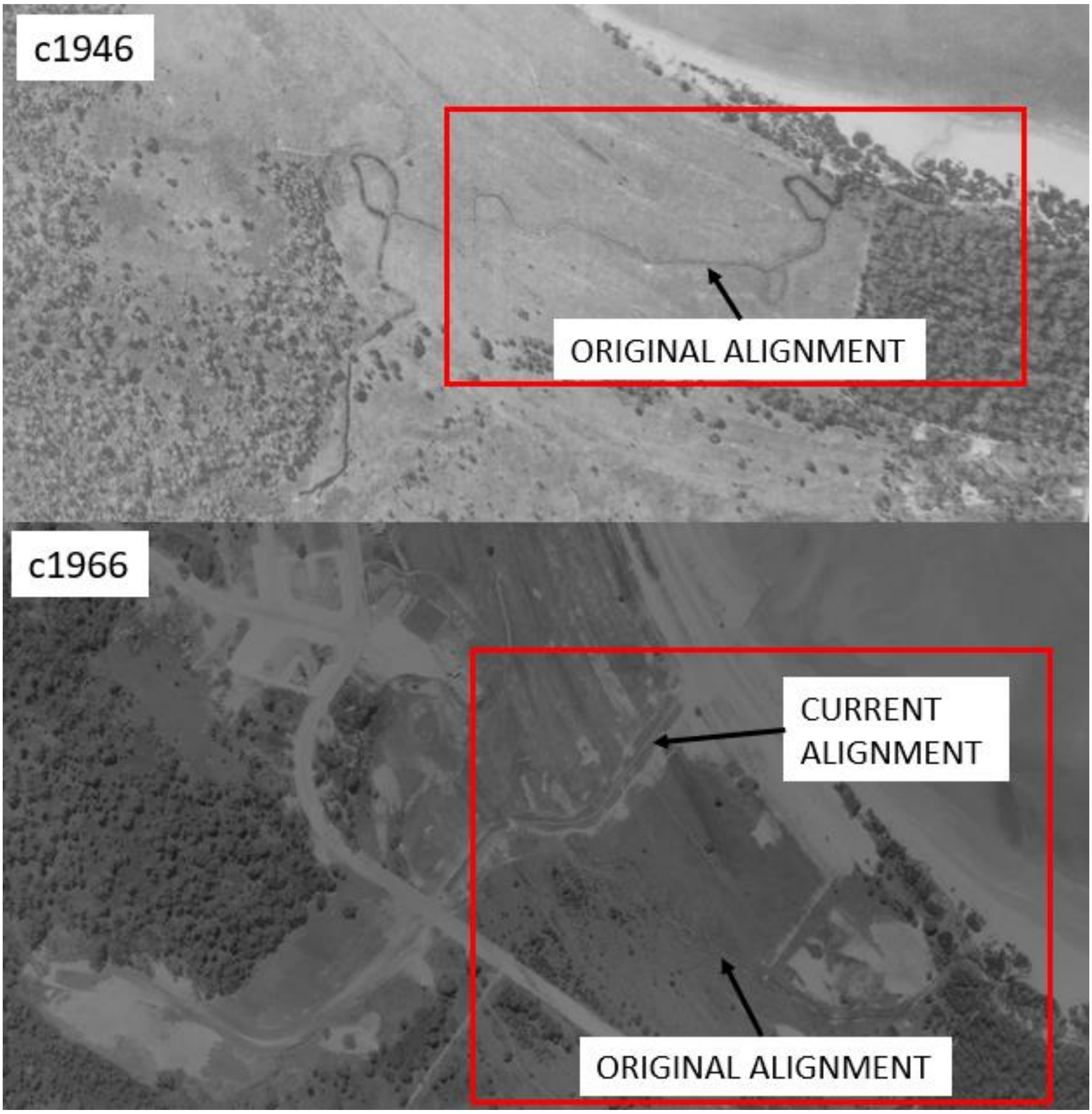


Figure 9. Historical and current alignment of drain and outfall to Freers Beach

Realignment of the drain would allow the DN900's will be free to service local rainfall and drainage within the lower golf course and service flooding when the capacity of the bypass is exceeded. Such a bypass may still be subject to some of the constraints of a piped outlet, such as tidal influence and blockage. It is much easier, however, to provide for the magnitude of 1% AEP flows in flow buffering areas such as ponds, open drains and channels as opposed to in pipes and culverts.

Even if a bypass was installed there may be a residual need to ensure any flooding along the rear of the properties on Shearwater Esplanade to is able to be captured by the existing Freer Street stormwater system, or via a new connection through to Freers Beach. See Figure 10.

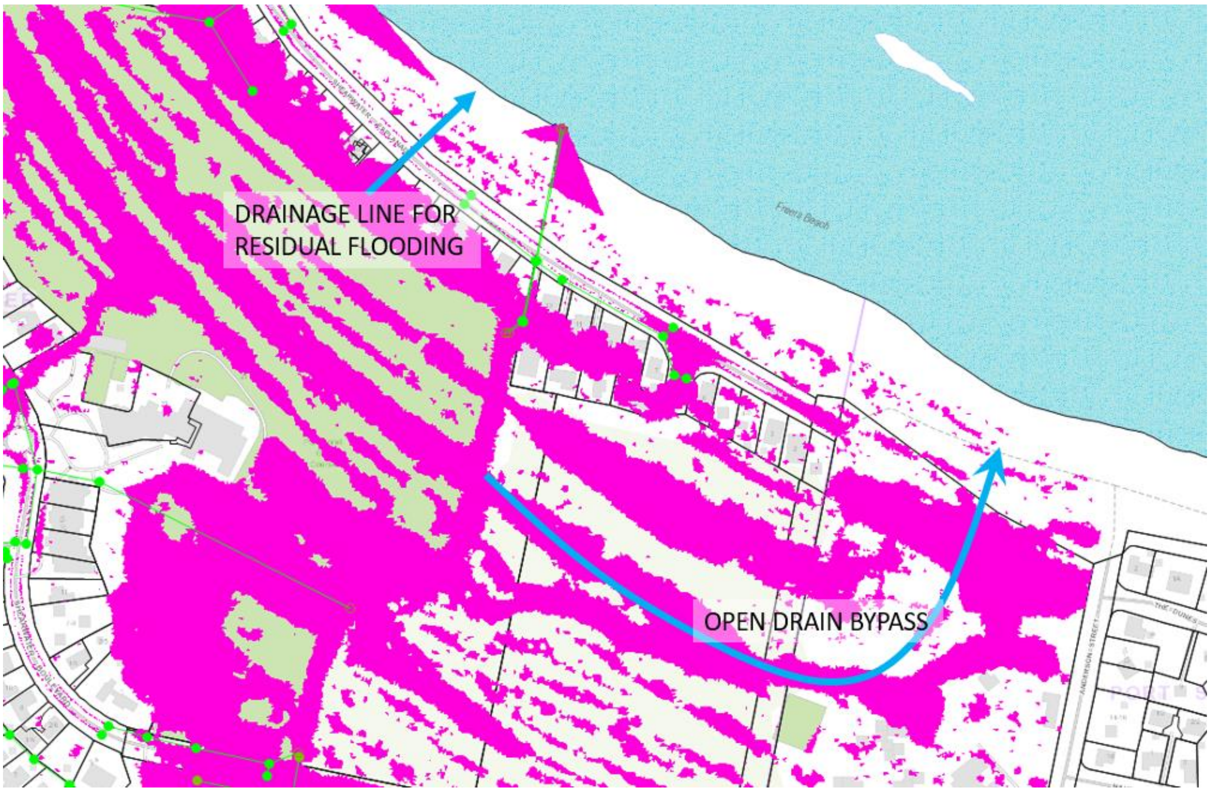


Figure 10. Suggested main channel diversion through to Camp Banksia, and new connection to Freers Beach.

As a last resort Council could consider a pumped system to help reduce flooding in the golf course. The pump-out rates required to provide a tangible benefit during the 1% AEP would be impractical, however such a system may increase recovery times after a flood event.

Immediate and relatively low-cost solutions to blockage issue affecting the outlets to Freers Beach would be for installation of removable drop boards or weirs in the manholes immediately downstream of the twin DN900 inlets at the golf course. These can be used to help prevent blockage of the outlets by driving stormwater low flows through each pipe alternately, while allowing high flows to overtop if not removed prior to a high-flow event.

Another opportunity is for the installation of duckbill check valves on the beach outlets. These operate by differential pressure and can prevent seawater infiltration into the upstream network. These can operate well in areas prone to sand build-up and not prone to jamming or corrosion as are standard flap valves.

6.2 UPSTREAM CONTROLS

Consideration may also be given to the provision of detention basins in the upper catchment which can reduce peak flows entering the lower catchment. These should be considered, if necessary, as an addition to the suggested lower catchment works.

It will be difficult and expensive to retrofit public detention basins into the developed urban residential area in the lower catchment. Unfortunately, the upper catchment is large and has multiple natural flow paths, and therefore multiple detention basins might be necessary, potentially in series, rather than a single more cost effective and efficient basin. Multiple basins would likely not remove the need for downstream works.

Figure 11 shows some indicative locations for potential detention basins. A large detention basin at location A would probably provide the most benefit. It is understood, however, that the streams upstream of the golf course are home to the freshwater burrowing crayfish. The construction of detention basins in some areas, including in location A, may therefore be problematic.

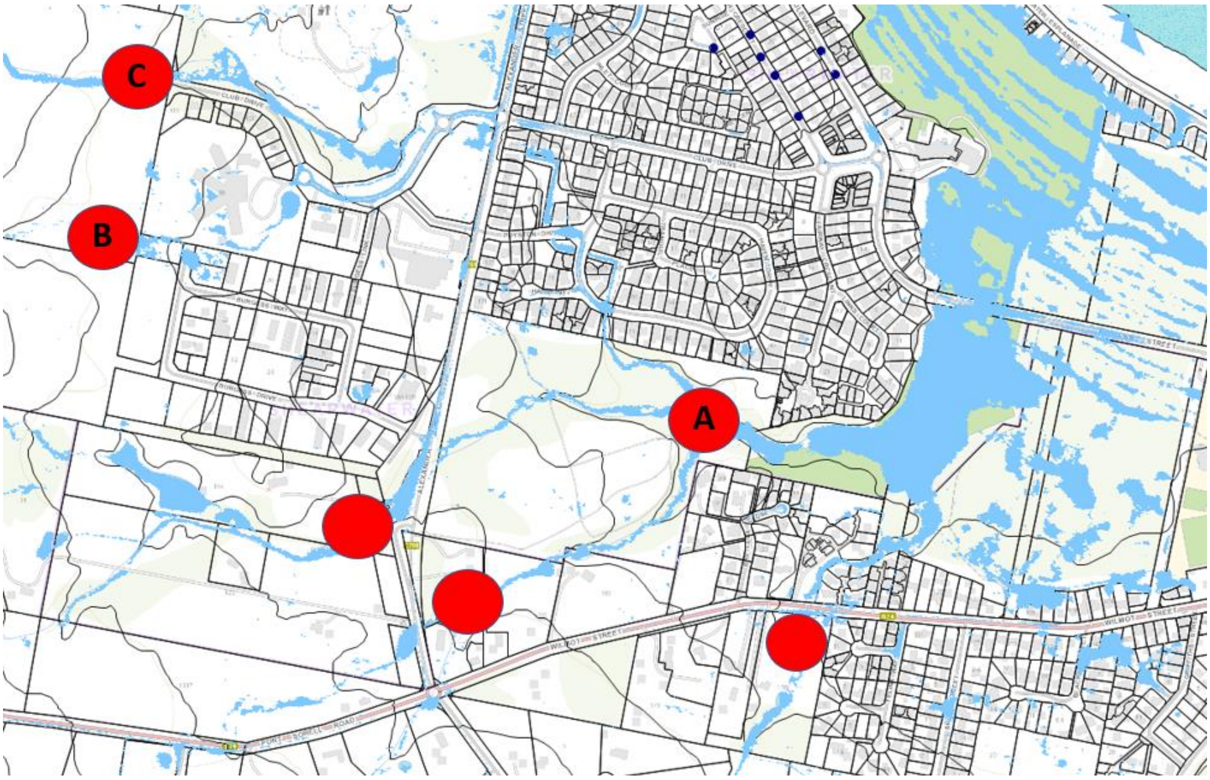


Figure 11. Potential detention basin locations

Wherever possible Council should require new developments to control their post-development flows to a predevelopment level for storm events up to and including the 1% AEP inclusive of climate change. Detention basins at locations B and/or C shown in Figure 11 should be constructed in association with developments in those areas. It is noted that the subcatchment sizes draining to these locations are modest in comparison to the remaining subcatchments which contribute flooding to the lower catchment, refer to Figure 12. Therefore, detention in these areas will only modestly decrease impacts at the bottom of the catchment.

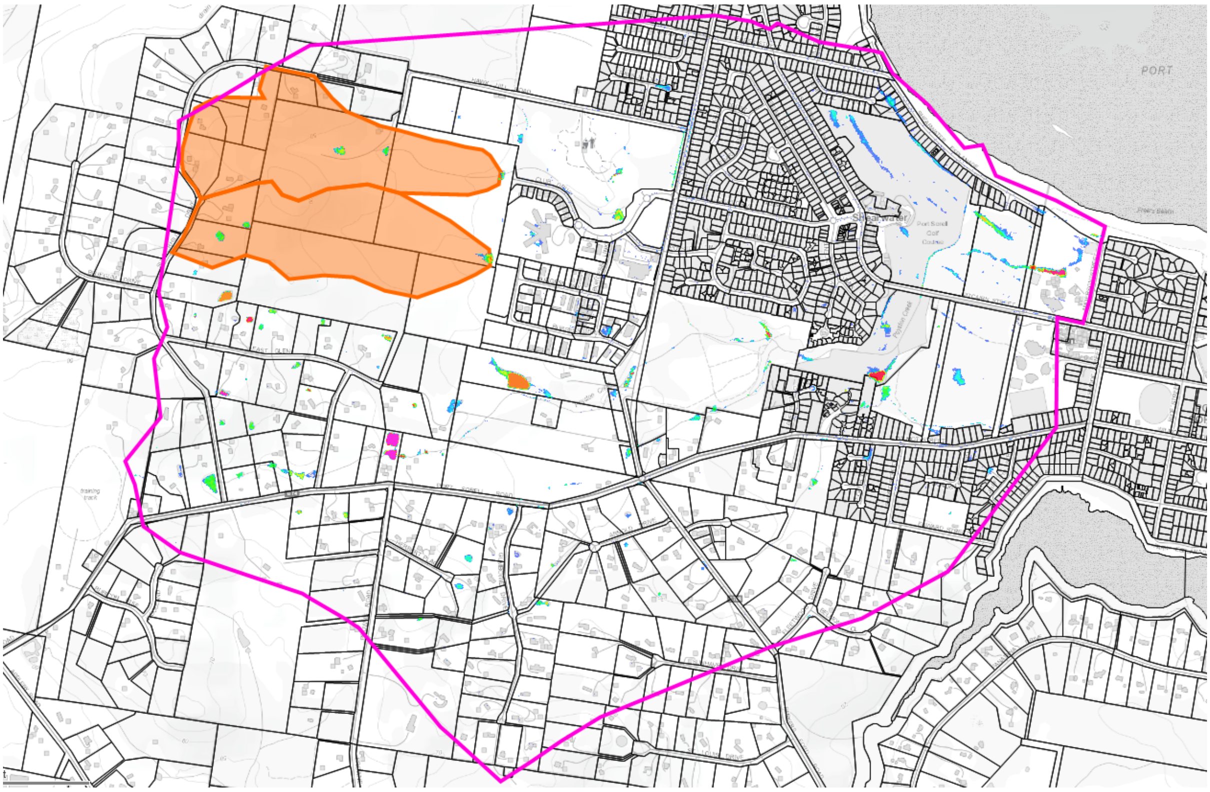


Figure 12. Relative size of subcatchments draining to potential detention basins B and C

It is recommended that Council consider augmenting the size of any basins in locations B and/or C over and above the requirement placed on the developers. The primary benefits of such detention basins will be better controlling the interface between the major overland flow paths upstream and the existing downstream minor system, as well as ensuring the immediate downstream piped system is not overloaded. The secondary benefit will be a modest contribution in the reduction in peak flows discharged to the golf course.

It was noted in Section 5.2 of this document that the Burgess Drive minor stormwater has limited capacity to accept the full amount of unrestricted pre-development flows from the upstream undeveloped subcatchment. This will be further exacerbated following development unless strict detention storage requirements are enforced. According to the *LGAT Subdivision Guidelines 2013*, industrial areas, such as that contained by Burgess Drive and Burgess Way, should have minor drainage systems which are fully able to service the 20 year ARI (5% AEP) with 100 year (1% AEP) flows serviced by open drains or roadways. This level of service is not provided by the existing drainage system.

Figure 13 shows a hydraulic grade line (HGL) of the Burgess Drive minor system from the north-western corner of no. 48 Burgess Drive down to Alexander Street for the 5% AEP. The location of this pipeline is presented in Figure 14 in red outline. A reduction in pipe size from DN525 to DN375 can be seen at Burgess Link. It is understood that the lateral link which discharges upstream into OneCare caters for surcharging into the original overland flow path.

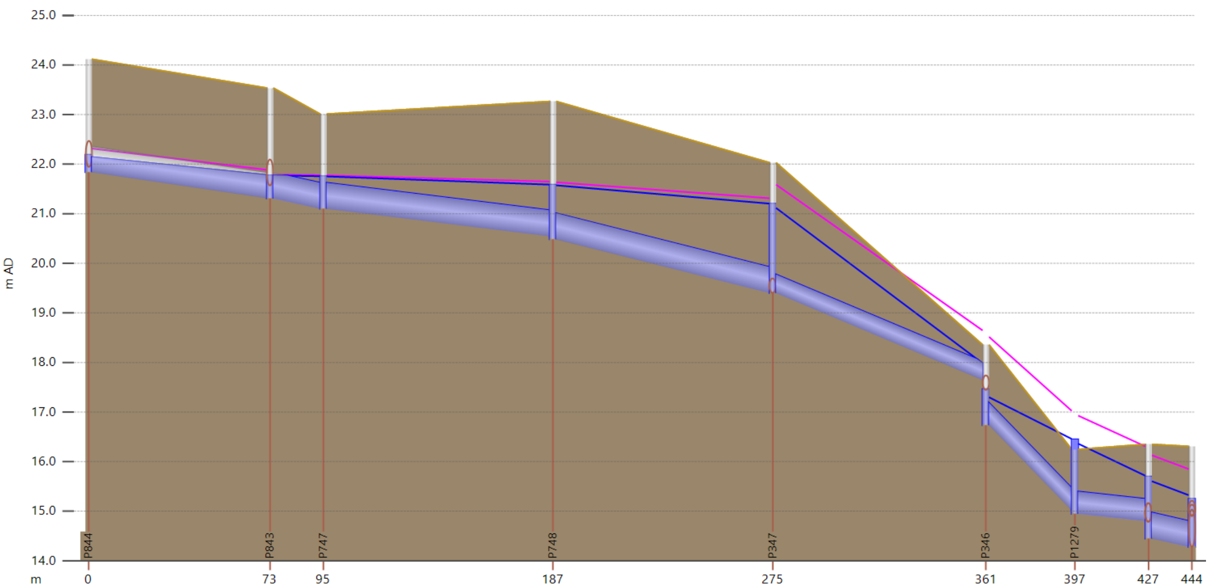


Figure 13. Burgess Drive HGL

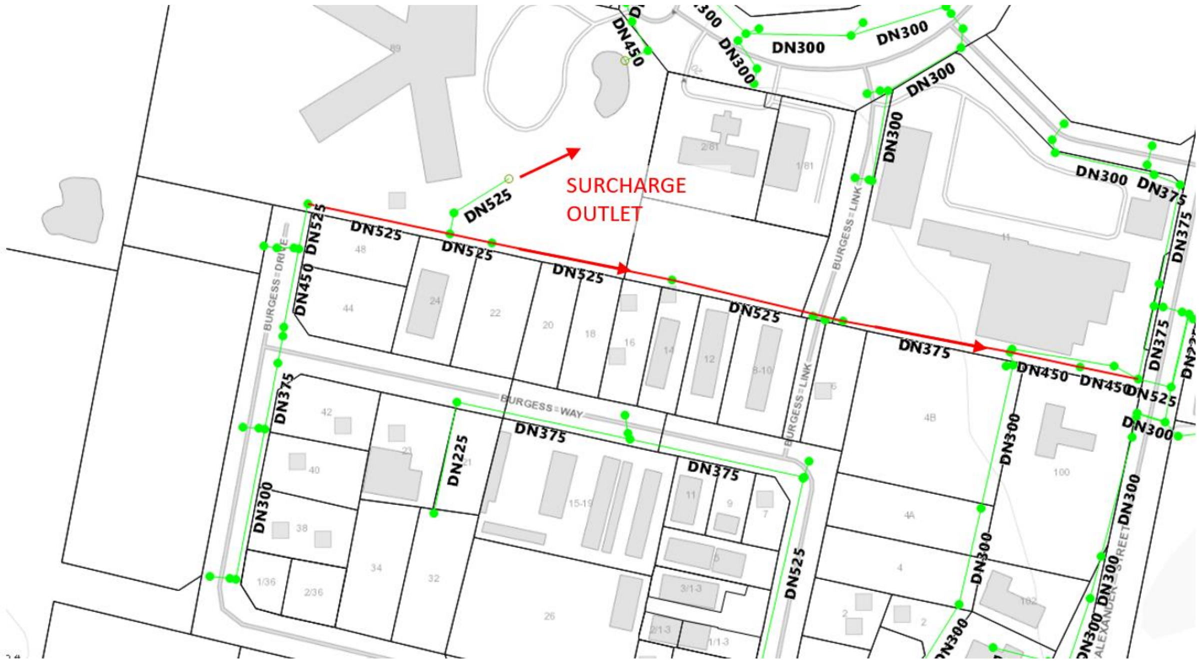


Figure 14. Burgess Drive pipeline

The results presented in Figure 13 are exclusive of any future development from industrial lots 1 and 2 Burgess Drive and exclusive of development of the general residential zoned land. The stormwater main is flowing full and a small amount of surcharge from a manhole within the shopping centre at no. 11 Poyston Drive is predicted.

It appears that the available capacity of the Burgess Drive system is currently marginally exceeded, and may be worsened even if post-development flows controlled to pre-development levels were allowed to be concentrated into the system above. The sizing and outflow rates of detention will need to be designed to account for current runoff levels, capacity levels in the receiving system, restrictions on outflows at the outfall and 1% AEP climate change volumes. Dynamic modelling of subdivision development, detention and fully developed downstream system should be undertaken in order to ensure detention basin outflow rates and capacity is correctly sized.

7. SUMMARY

The lower catchment which includes the golf course is caught between climate change impacts, which affect it from below, and increased runoff which affects it from above. The Tasmanian Coastal Adaptation Pathways Project clearly shows the forecast risks to the lower catchment, for which adaptation may be appropriate rather than solely relying on engineering solutions.

Present day tides, the flatness of the lower catchment, downstream development, and natural sand deposition present constraints which limit Council's ability to reliably increase outflows through the golf course. It is clear, however, that while Council has a responsibility to ensure additional development upstream does not incrementally increase flooding in the area. This responsibility also provides opportunities to reduce current-day downstream impacts.

A summary of this report's recommendations is as follows:

1. Adaptation and engineered mitigation need to be considered in the context of longer term cost verses benefit of mitigating current issues and providing for future development.
2. In order to improve modelling of the existing catchment and potential solutions going forward, it is recommended that data be collected to further validate and calibrate the model. There are several ways in which data can be collected in order to refine the model:
    a. Through flow loggers installed on strategic pipelines such as at Pitcairn Street. A formal flow monitoring site including weir and telemetry could be considered, however this would be significantly more expensive
    b. Installation of a local rain gauge. The nearest BOM weather stations are at Narawntapu National Park (station no. 91349) and at Northdown (station no. 91039), which are respectively 5.4km and 5.5km away from Shearwater. These stations also only provide daily rainfall totals. A local gauge operated by Council staff can provide rainfall depths in 1 minute increments.
    c. Collection of tidal data. This could be achieved simply though survey of the high and low tides once every week or two over the space of two months. This information will help confirm that the modelled tidal boundary conditions are reasonable.
    d. Examination of seasonal infiltration rates and groundwater levels in the golf course. According to the NRE Tasmania's Groundwater Information

Access Portal (https://wrt.tas.gov.au/groundwater-info/) there are two functioning bores within the golf course. These were installed in September 1974 (bore 1606) and April 2009 (bore 41372) and encountered standing water levels at 1.5m and 3m deep at the time. It may be possible to use these bores collect standing water levels in different seasons in order to better understand how infiltration rates within the golf course are affected by seasonal rainfall. Constant head permeameter tests can also easily be carried out in order to determine how infiltration rates and saturated hydraulic conductivity (K_{sat}) rates seasonally.

3. To help prevent and clear sand blockages which impact the twin DN900s at Freers Beach drop boards/weirs in the manholes immediately downstream of the twin DN900 inlets at the golf course can be trialled. These can be used to help prevent blockage of the outlets by driving stormwater low flows through each pipe alternately, while allowing high flows to overtop if not removed prior to a high-flow event.
4. An alternative to the drop boards (recommendation 3) is for the installation of 'Tideflex' or similar duckbill check valves on the beach outlets. These would prevent tidal backflow, are not impacted by corrosive environments, and are claimed to be self-cleaning. Check valves may, however reduce the carrying capacity of the outlets due to headloss.
5. Assess the topographic history of the area and the feasibility of installing a diversion of the main drainage channel, north of Pitcairn Street, through to the Freers Beach via Camp Banksia. An indicative route is shown in Figure 10. Refer to Section 8.
6. If recommendation 5 is feasible this would allow for the Pitcairn culverts to be upgraded and for the flood footprint south of the road to be reduced.
7. If an overflow route is not possible (recommendation 5) Council may consider upgrading or installing new piped links through to Freers Beach. Such links would be prone to blockage with sand as are those existing, and as such they should be accompanied by a robust operation and maintenance regime and consideration should be given to the installation of the duckbill check valves as per recommendation 4.
8. Consideration should be given to drainage of the golf course at its northern extent. There is currently no ability for the existing drainage to intercept and collect flooding which escapes the main drainage channel, or which accumulates naturally within the troughs in the golf course which run parallel to the beach. It appears that this has not historically impacted residential properties, however.
9. An assessment should be made of the potential for installation of new detention basins on greenfield sites. Constraints such as the presence of burrowing crayfish and the multiple natural overland flow paths requires means several potential detention basins require investigation, rather than a single large basin. A basic assessment could be conducted quickly by simply removing single or multiple

overland flow paths from the model, and checking the impacts of the reduced downstream flows. Refer to Section 8.

10. Council should review its stormwater detention policies to ensure that all new developments within the catchment, where possible, are required to limit the post-development flows to a predevelopment level. Novel policies, such as the installation detention or rainwater harvesting tanks on new and existing properties can also be considered.
11. Ensure the future development of residential subdivision west of Burgess Drive incorporates detention that restricts outflows from the fully developed site to pre-development levels, or to capacity levels in the receiving system (whichever is less), and is sized to hold 1% AEP climate change volumes. Dynamic modelling of the subdivision, proposed detention, and fully developed downstream system should be undertaken in order to ensure it is correctly sized. Refer to Section 8.

Additional investigations and modelling of mitigation measures is further discussed in Section 8.

8. STAGE 2 – SCENARIO MODELLING OF MITIGATION MEASURES

Due to the number of individual subcatchments contributing to stormwater to the lower catchment mitigation will not able to be managed by a single intervention. Interventions, such as detention basins, need to be considered within each of the contributing subcatchments.

There is currently a modest restriction in outflows to Freers Beach in a coinciding high tide, assuming there is no blockage of the outfalls. Restrictions on outflows from the outfalls will increase as sea level rises with climate change. These restrictions, in addition to the inherent limitations of the existing public stormwater system to accept additional stormwater inputs, may require outflow rates from future detention systems to be reduced to rates smaller than would otherwise be expected. Detention will need to consider the impacts of climate change induced changes in rainfall intensity, and peak events coinciding with rising sea level and storm surge.

It has been agreed that Stage 2 modelling will investigate:

- The potential sites for, and provision of, detention basins above Alexander Street and the effect on lower catchment flooding;
- Overland flow path linkages from the upper catchment to the lower catchment;
- Lower catchment (below Alexander Street) volumes and the effect of an eastern diversion to additional detention facility on Camp Banksia land including a new outfall to Freers Beach;
- Reprofiling of the drainage line through the golf course to provide additional storage, when the outfall is restricted by a high tide (assumes Tideflex or similar check valve installed); and

- A combination of upper and lower catchment interventions described above.

When Stage 2 modelling is completed, it is foreseen that a plan for stormwater management, which will set thresholds and locations for private and public management of future stormwater loads, will be produced.

9. REFERENCES

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.

Coombes P, and Roso S, (Editors), *2019 Runoff in Urban Areas, Book 9 in Australian Rainfall and Runoff - A Guide to Flood Estimation*, Commonwealth of Australia, © Commonwealth of Australia (Geoscience Australia), 2019.

Department of State Growth, *A preliminary Mannings-n layer to support regional flood modelling in Tasmania*, 2020.

Infoworks ICM Help File.

Latrobe Council Stormwater System Management Plan V0.1, 2020.

Latrobe Interim Planning Scheme 2013.

LGAT Subdivision Guidelines, 2013.

Martin C, Port Sorell, *Shearwater and Hawley Stormwater Assessment*, CSE Tasmania Pty Ltd, 2017.

O'Loughlin G., and Stack, B., *DRAINS User Manual*, Watercom, 2017.

SGS Economics & Planning, *Tasmanian Coastal Adaptation Pathways Project – Port Sorell Report*, 2012.

Tasmanian Local Council Sea Level Rise Planning Allowances, Department of Premier and Cabinet, Tasmania.

SGS Economics & Planning, *Tasmanian Coastal Adaptation Pathways Project – Port Sorell Report*, 2012.

APPENDIX A – MODEL SETTINGS

A.1 VALIDATION OF PERVIOUS HYDROLOGY

In the absence of actual rainfall and flow response data with which to calibrate the model it was deemed appropriate to adopt the "Fixed" and "Horton" runoff models to represent the percentage runoff from impervious and pervious surfaces respectively for the purposes of this assessment. The "Horton" model is described in ARR 2019 Book 5, Chapter 3.

To provide validation of the pervious hydrological settings in the 1D-2D rain-on-grid model a separate 1D WBMN model of an upper subcatchment was produced. This 91 hectare subcatchment is shown in Figure A1.1.

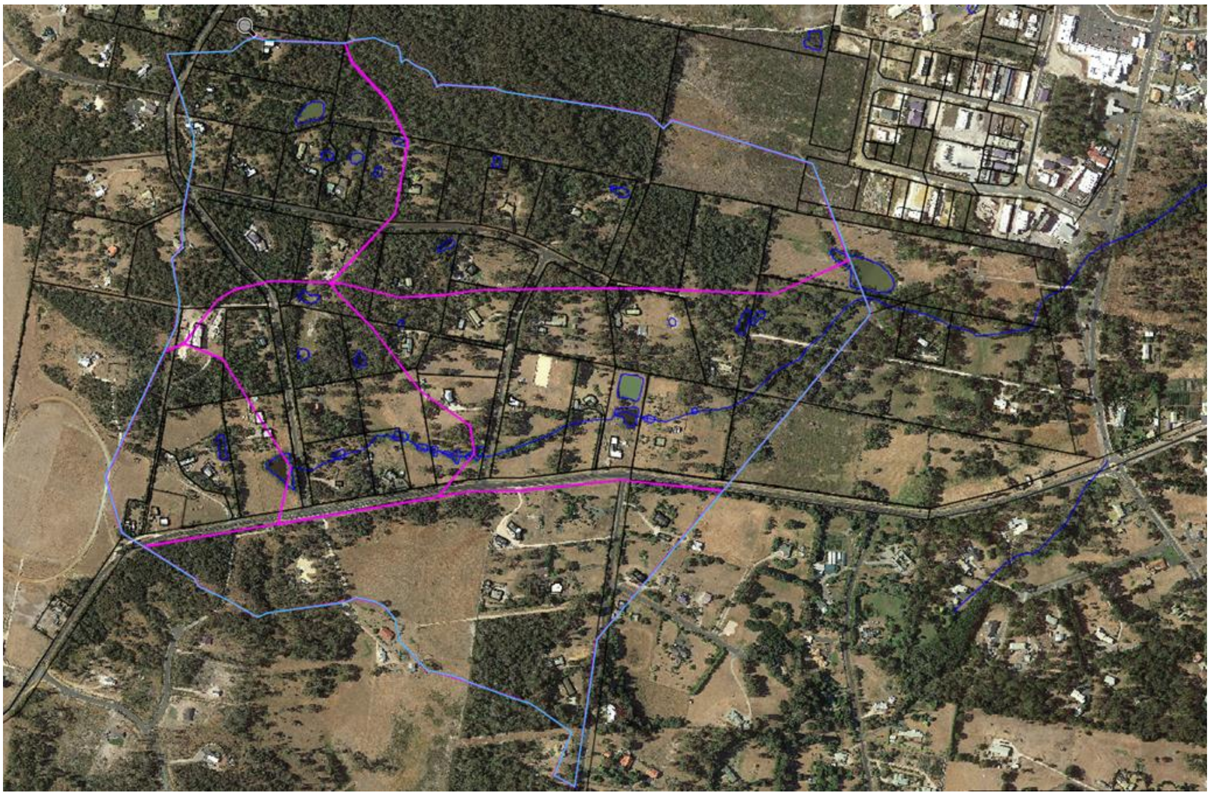


Figure A1.1 Upper catchment used for validation

The ARR Data Hub provides rural initial loss (IL) of 14.0mm and continuing losses (CL) of 4.5mm/hr. The Data Hub values are derived primarily from pasture and paddock coverage, while most of this subcatchment is forest. The loss values are therefore likely to be conservative for this specific subcatchment but representative of the overall Shearwater catchment's pervious surfaces. The small dams located throughout the subcatchment were ignored in the 1D validation model.

The 2D model used Horton Infiltration model. The final (limiting) infiltration rate was set to 4.5mm/hr, i.e., the ARR CL rate, and the initial infiltration rate was adjusted to replicate the flow rates found in the 1D WBNM model. Given the DEM picked up the dam depressions, an initial model was run to fill the dams and the Horton infiltration rate was allowed to recover.

Table A1.1 provides the peak mean flow rates for the 45 minute to 3 hour storm ensembles using an initial Horton infiltration rate of 50mm/hr. The WBMN results were on average 6.8% higher in the 1% AEP and 11.6% lower in the 1% AEP inclusive of the https://data.arr-software.org/ RCP 8.5 climate change loading factor of 16.3%.

1%	**WBNM**	**ICM**	**Difference (%)**
45min	5.55	4.77	16.4
60min	5.75	5.33	7.9
90min	5.3	4.73	12.1
120min	5.65	5.6	0.9
180min	5.3	5.47	-3.1
1% CC	**WBNM**	**ICM**	**Difference (%)**
45min	7.1	7.99	-11.1
60min	7.3	8.43	-13.4
90min	6.7	7.4	-9.5
120min	7.1	8.15	-12.9
180min	6.6	7.44	-11.3

Table A1.1 Peak mean flow rates (m^3/s) for the sample catchment

These Horton parameters were therefore deemed suitable for adoption in the greater 1D-2D model.

An additional check of these Horton parameters was made using rainfall data from the nearby Northdown (Hamley) BOM station (no. 91039). Historical daily totals from the station were ranked and the 5-day antecedent rainfall totals were analysed. Once each day was identified as having rainfall, the preceding 5 days the data was ranked from the largest daily rainfall to the smallest, with the largest 100 daily events given an Antecedent Moisture Condition (AMC) rank based on Table A1.2:

Number	**Description**	**Total rainfall in 5 days preceding the storm** (mm)
1	Completely dry	0
2	Rather dry	0 to 12.5
3	Rather wet	12.5 to 25
4	Saturated	Over 25

Table A1.2 Antecedent moisture conditions (ref. DRAINS Manual)

The frequency of each AMC number was then tallied as per the chart below:

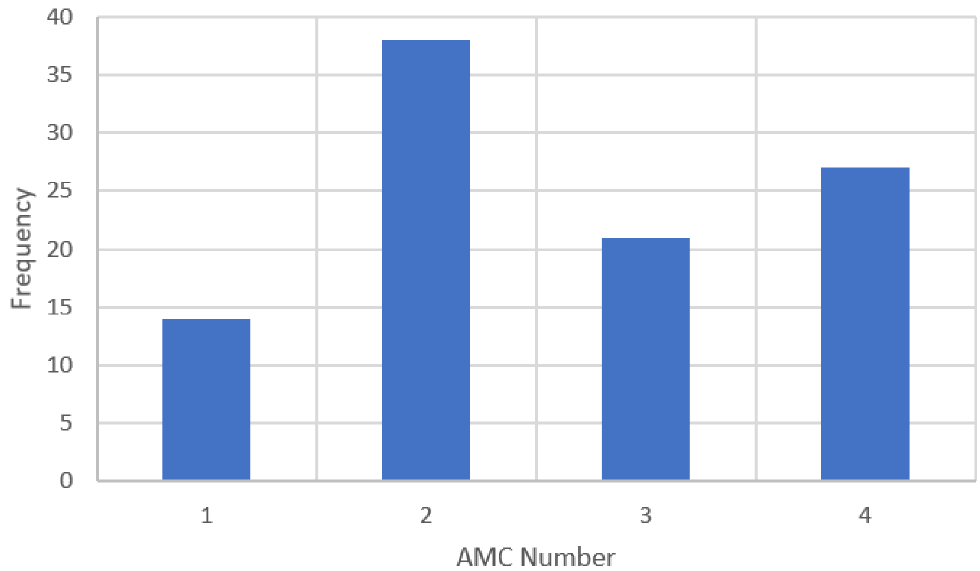


Figure A1.2 AMC Number Frequencies for Northdown

The mean of AMC number is 2.61, the median AMC number is 2, and the average 5-day preceding 5-day rainfall depth was 19.57 mm. The adopted Horton initial loss rate of 50 mm/hr lies halfway between the recommended values for type B and C soils with 'rather wet' antecedent conditions. It also lies closest to the 'rather wet' AMC for a type C soil, refer to Table A1.3:

Factor	Soil Type			
	A (or 1)	B (or 2)	C (or 3)	D (or 4)
Initial Rate, f_0 (mm/h)	250	200	125	75
Final Rate, f_c (mm/h)	25	13	6	3
Shape Factor, k (h^{-1})	2	2	2	2
Antecedent Rainfall Depths (mm) for AMCs:				
1	0	0	0	0
2	50	38	25	18
3	100	75	50	38
4	150	100	75	50
Initial Infiltration Rates (mm/h) for AMCs:				
1	250	200	125	75
2	162.3	130.1	78.0	40.9
3	83.6	66.3	33.7	7.4
4	33.1	30.7	6.6	3.0

Table A1.3. Horton infiltration model parameters (ref. DRAINS Manual)

Soil types B and C are defined as follows:

Group B is silt loam or loam. It has a moderate infiltration rate when thoroughly wetted and consists chiefly or moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C soils are sandy clay loam. They have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine structure.

Given that the 50mm/hr initial Horton infiltration rate led to results which were close to those given by the 1D model using default Australian Rainfall and Runoff hydrology, and because it aligns well with reasonable Horton infiltration parameters, the rate was adopted.

The ARR Data Hub values and the adopted Horton values *may* underestimate the infiltration rates in the lower golf course, where it is likely the soil profile is sandy. However sandy soils subject to seasonal or tidal high water tables may still have very low infiltration rates. As such the adopted upper catchment pervious parameters were used.

A.2 URBAN HYDROLOGY & BUILDINGS

To ensure modelled overland (2D) flow and in-pipe (1D) flows were well represented within the urban area in the lower half of the catchment a proportion of the subcatchment was directed directly into minor drainage system.

1D subcatchment polygons 60% of the urban residential lot sizes were created and were given a fixed runoff coefficient of 1. Subcatchments were directed to adjacent pipes and pits. The remaining areas, including roads and naturestrips, were modelled using direct rainfall.

Figure A.2.1 shows the 1D catchments in cyan.

On some occasions building footprints fell outside of the 1D subcatchments. In these instances, the overlap was also impervious but modelled in the 2D realm. Other urban land uses were modelled in a similar fashion but on a case by case basis.

Manholes were modelled as sealed.

Figure A2.1 Urban residential 1D subcatchments (Cyan)

A.3 2D ROUGHNESS COEFFICIENTS

Depth-varying Manning's n runoff coefficients were used. These were generally in alignment with industry standard values and the draft document *A preliminary Manning's-n layer to support regional flood modelling in Tasmania* (Department of State Growth, 2020).

In the 2D realm building were modelled with a Manning's n=1.

A.4 PIPE ROUGHNESS

In the absence of specific data or CCTV pipe roughness was set to the following Colebrook-White values for different materials:

PP BlackMax/Stormpro:	0.1
PVC:	0.1
Concrete/VC:	0.6
Unknown:	0.6

A.5 MANHOLE/ENERGY LOSS DUE TO TURBULENCE

Infoworks ICM calculates headloss at the top of each conduit, to represent the energy lost due to turbulence at the transition between a manhole and a conduit. It is also calculated at the bottom of each conduit to represent the loss at transition from conduit to manhole. There is greater turbulence, and therefore headloss, at the top of a conduit.

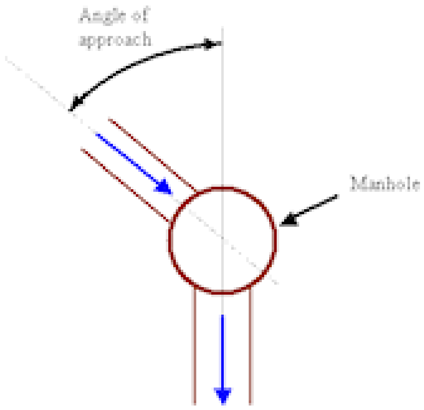
The headloss equation is presented below:

$$\Delta h = ku * ks * kv * (v^2/2g) \quad (1)$$

where:

Δh = headloss
ku = user defined headloss factor
ks = surcharge ratio coefficient
kv = velocity coefficient
v = flow velocity (m/s)
g = acceleration due to gravity (m/s^2)

The headloss factor (Ku) was defined as follows by the angle of approach to the manhole:



Angle of Approach	Ku
30	3.3
60	6.0
90	6.6
>90	8.0

Figure A5.1 Manhole headloss factors (Ku)

For the normal and high headloss types the surcharge ratio coefficient (ks) and the velocity coefficient are hard coded into Infoworks ICM, as shown in the following figure:

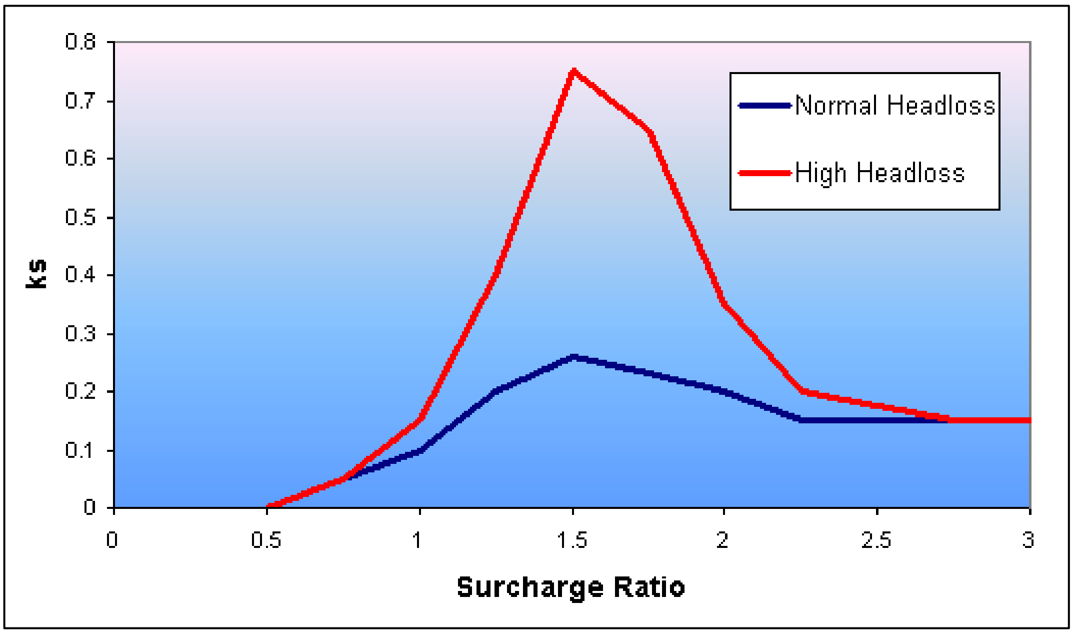


Figure A5.2 Surcharge ratio coefficients (Ks)

The internal condition of individual manholes and pits within the stormwater system is unknown. 'High' headloss is appropriate for badly constructed manholes that are benched only to half pipe height. 'Normal' head losses were assumed.

A.6 GULLY PIT INLET PARAMETERS

For simplicity it has been assumed that all pits have a hydraulic capacity as per a 1220mm side entry pit in sag conditions. Refer to LGAT Standard Drawing TSD-RF03-v3:

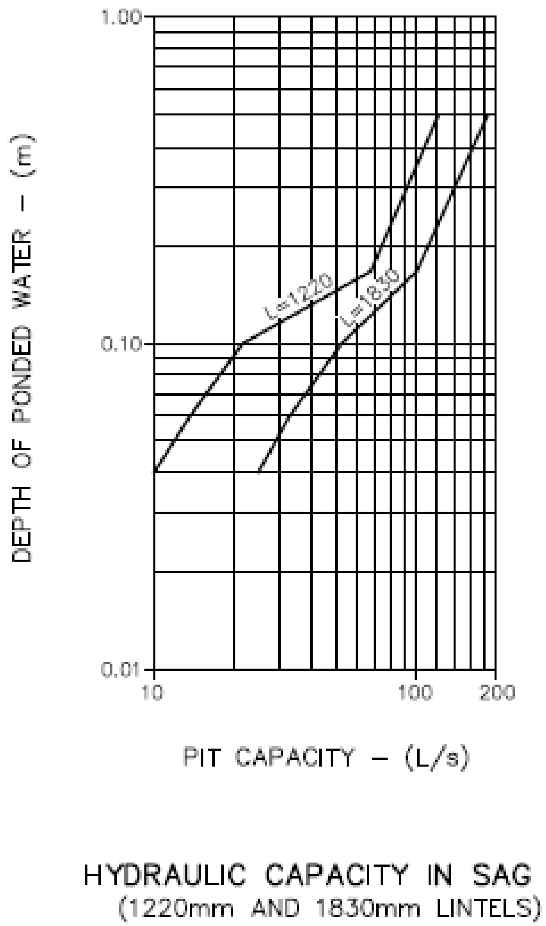


Figure A6.1 Pit hydraulic capacity

No blockages were assumed.

APPENDIX B – FLOOD MAPS