# Stormwater Management Plan
# Coastal Catchments Between Glenelg and Marino

## Cities of Holdfast Bay and Marion

July 2014

Ref No. 20100878RA7F

*The Cities of Holdfast Bay and Marion have an overarching objective of progressing towards becoming "Water Sensitive Cities" and to minimise flooding and harness the potential of stormwater to overcome water shortages, reduce urban temperatures , and improve waterway health and the landscape of their cities. Water Sensitive Urban Design is the process that will lead to Water Sensitive Cities.*

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# Document History and Status

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| F | Final Version | KSS | KSS | KSS | 22 July 2014 |

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# 1 Introduction

## 1.1 Intent of the Plan

The Cities of Marion and Holdfast Bay have agreed to a coordinated approach to the management of stormwater across both local government areas for the coastal catchments between Glenelg and Marino. The area of land generally within the scope of this Stormwater Management Plan is shown on Figure 1.1.

The intent of the Plan is to set out strategies, actions and programs that can be implemented jointly by the Cities of Marion and Holdfast Bay so that progress will be made towards the overarching objective of both Councils which is:

**The Cities of Holdfast Bay and Marion have an overarching objective of progressing towards becoming "Water Sensitive Cities" and to minimise flooding and harness the potential of stormwater to overcome water shortages, reduce urban temperatures , and improve waterway health and the landscape of their cities. Water Sensitive Urban Design is the process that will lead to Water Sensitive Cities.**

The Plan applies to the coastal catchment which is the area generally west of the Sturt River from the Patawalonga Lake in the north to the hills in the south. The catchment is described in more detail in Section 2.

## 1.2 Contents of the Plan

This Stormwater Management Plan describes the agreed approach and has been prepared to comply with the requirements set out in the Stormwater Management Authority's "Stormwater Management Planning Guidelines", (Stormwater Management Authority, July 2007).

This plan contains:

- a description of the catchment
- a description of all known stormwater assets
- the identification of problems and opportunities
- stormwater management objectives
- identification of strategies and outcomes
- costs, benefits and funding arrangements
- priorities and timeframes
- responsibilities
- consultation

## 1.3 Investigations and Studies Underpinning this Plan

To support this Plan a number of investigations and studies have been undertaken and are summarised in this document but are reported in standalone documents. These include the following:

1. **Jensen Planning and Design** (April 2011), Stormwater Management Plan: Cities of Holdfast Bay and Marion Development Potential within the Catchment.

   This report considers factors impacting on stormwater management and particular changes that could occur into the future by considering the:

   - strategic and policy context effecting development
   - zoning provisions

- 30 Year Plan for Greater Adelaide
- current growth trends
- dwelling density and site coverage

2. **Australian Groundwater Technologies** (February 2011) Managed Aquifer Recharge as a Stormwater Management Option, Cities of Holdfast Bay and Marion.

   This report provides an overview of the hydrogeology of the region, the potential for managed aquifer recharge and a summary of schemes operating in the region.

3. **Tonkin Consulting** (January 2013) Marion and Holdfast Bay, Floodplain Mapping and Drainage Capacity Assessment Report.

   This report sets out the underlying assumptions and inputs into the computer modelling that was undertaken to produce floodplain maps of the catchment both for the current situation and for a future scenario which allows for increased urban density, and increased rainfall intensity and sea levels due to a changing climate. The report includes:

   - assessment of the current imperviousness of the catchment
   - assumptions about the future imperviousness of the catchments based on development trends
   - a review of climate change predictions particularly in relation to future rainfall intensity
   - a summary of the sea levels adopted for the modelling both for the current situation and a future scenario that allows for sea level rise
   - an assessment of the financial costs (flood loss damages) that will occur as a result of flooding
   - the results of floodplain mapping
   - the results of the assessment of the capacity of the current drainage network

4. **Tonkin Consulting** (January 2013) Discussion Paper – Water Quality Data

   This report summarises the changes that have occurred to the catchment since European settlement, a summary of the available water quality data for stormwater discharges and the marine environment and a summary of the key water quality issues for stormwater management.

5. **Tonkin Consulting** (January 2013) Discussion Paper – Water Quality Model

   This report summarises modelling work undertaken to understand the pollution generation from the catchment, opportunities for water quality improvement and assessment of the effectiveness of various Water Sensitive Urban Design (WSUD) initiatives.

6. **Tonkin Consulting** (September 2012) Discussion Paper – Retention Storage Systems

   This report summarises the result of modelling to synthesise the effect of providing on site retention storages within new development.


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**Cities of Marion and Holdfast Bay**

**STORMWATER MANAGEMENT PLAN STUDY AREA**

**Figure 1.1**

# 2 Catchment Description

## 2.1 Catchment Extent

The catchment is shown on Figure 2.1. It comprises a number of sub-catchments that are logically grouped and encompass all of the land discharging to Gulf St Vincent (the Gulf) between Glenelg and the cliffs at Marino, as well as some land that discharges directly into either the Sturt River or the Patawalonga Lake. The Sturt River, Brown Hill Creek and the other creeks discharging into the northern end of the Patawalonga Lake are not covered by this plan. The eastern and northern boundary of the catchment is generally the Sturt River, or the boundary of the Sturt River catchment. The Sturt River is a logical catchment boundary since its capacity effectively isolates the catchment from flooding in the Sturt River catchment. Whilst it is conceivable that an extreme rainfall event could cause flooding from the Sturt River to impact on the catchment, this will not occur for the range of storm events considered as part of a Stormwater Management Plan, that is, up to a 100 year Average Recurrence Interval (ARI) event.

The catchment area is approximately 35 $km^2$ of which 13 $km^2$ is in the City of Holdfast Bay and 22 $km^2$ is in the City of Marion. Approximately 30 $km^2$ is urban and the remaining 5 $km^2$ is the rural open spaces of the escarpment at the southern end of the catchment.

The area varies in topography, including steeper zones in the south and the undulating sand dunes running along the coastline. The natural grade is toward the north-west, stormwater from the City of Marion area generally discharges to the Gulf through the City of Holdfast Bay. The urban areas are typically flat to moderately sloped with grades in the order of 1%.

The topography of the catchment is shown graphically in Figure 2.2.

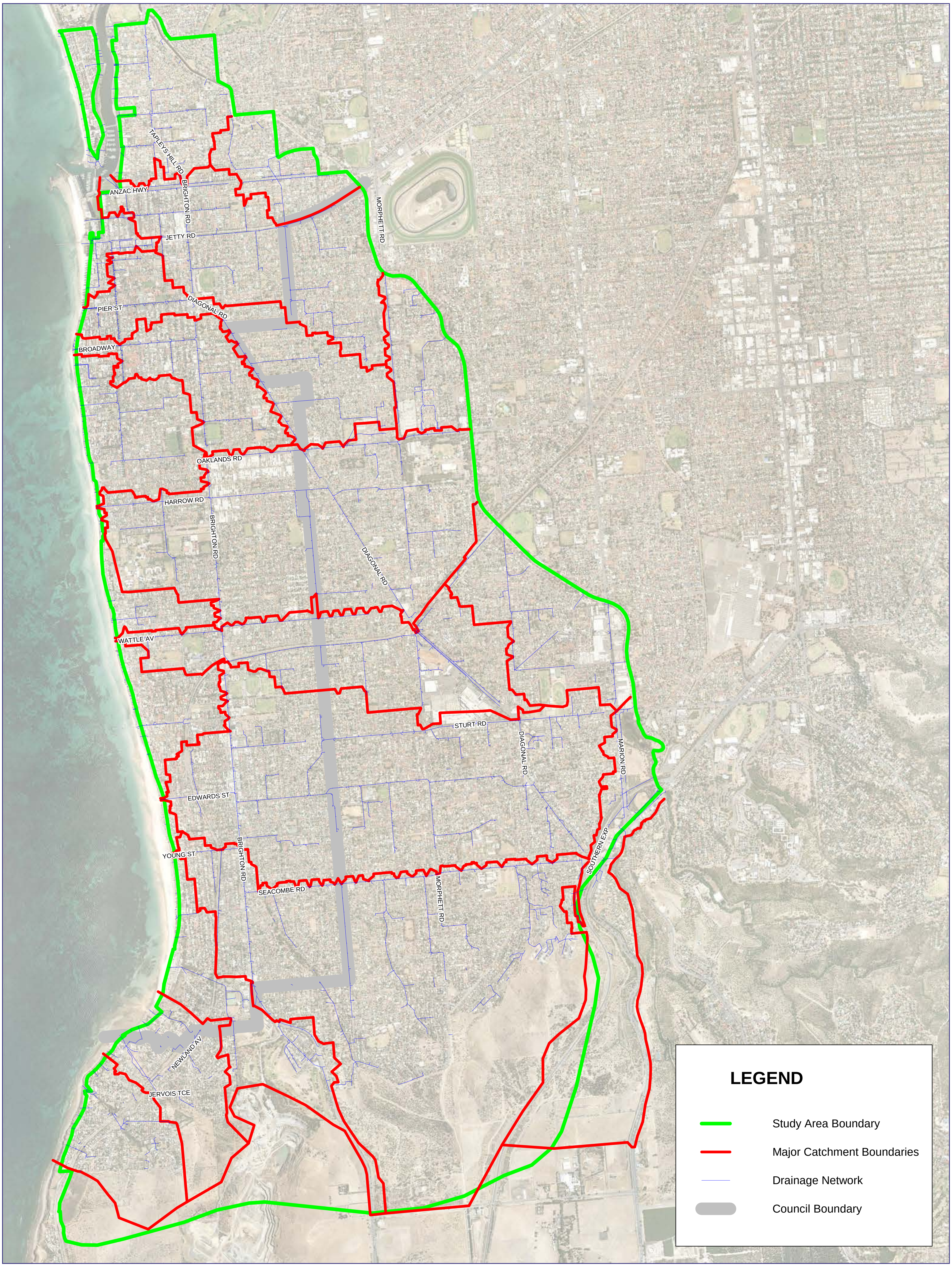
## 2.2 Historical Context

Prior to European settlement, the hydrology of the Adelaide Plains was quite different to that which exists today. Figure 2.3, which is based on surveys undertaken by Colonel Light, shows far less connection between the rivers and creeks of the plains and Gulf St Vincent. In 1839 the only outlet to the sea between Marino and Outer Harbor was the Patawalonga. The only defined watercourses in the catchment are the gullies in the hills at the southern part, and the Sturt River on the eastern side.

In terms of the catchment, apart from the hills in the southern part, the rainfall from the study area would have been trapped behind the coastal dune where it would have infiltrated into the ground or, during periods of heavy rainfall, moved slowly northwards towards the Patawalonga. The pattern of catchment outflows would have been dominated by winter runoff which would have been passed through broad floodplains and reed beds prior to discharge to the marine environment. Loads of nutrients and turbidity would have been small as a result of sedimentation and other processes that occurred on these floodplains and in the reed beds.

Significant changes have occurred during the development of the catchment; notable amongst these was the construction of major drainage infrastructure which responded to the demand to protect property from flooding, particularly:

- the works undertaken during the 1930s as part of the Metropolitan Drainage Works. These included work in the Sturt River to increase its capacity;
- South Western Suburbs Drainage Scheme in the late 1960s and 1970s that included:
  - channelisation and concrete lining of the Sturt River from Sturt Road to the Patawalonga
  - the construction of underground drains directly connecting the catchment to the Gulf by piping stormwater beneath the dunes.



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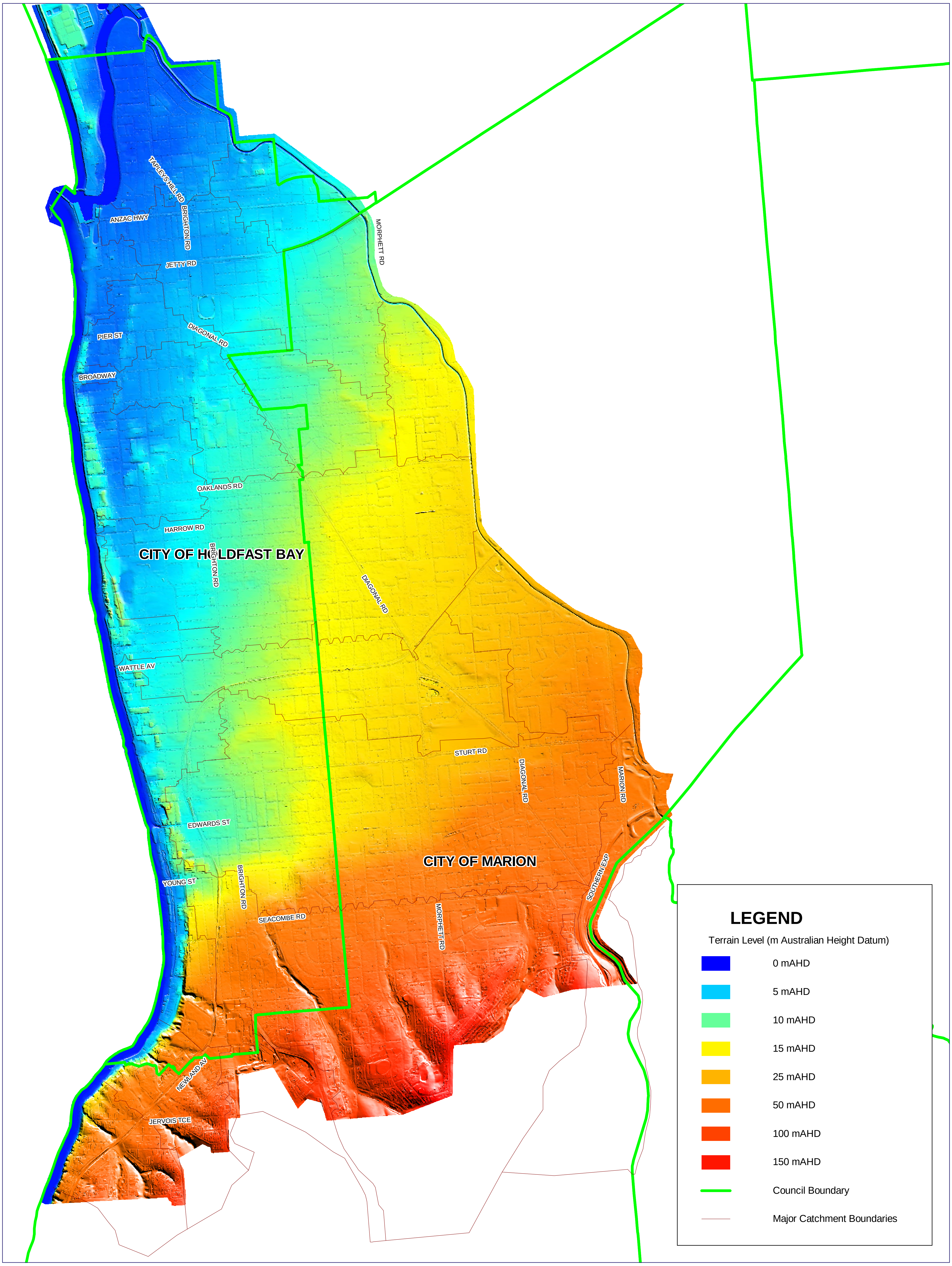
Cities of Marion and Holdfast Bay

## MAJOR CATCHMENT BOUNDARIES

Figure 2.1



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**Cities of Marion and Holdfast Bay**

## DIGITAL TERRAIN MODEL

Figure 2.2

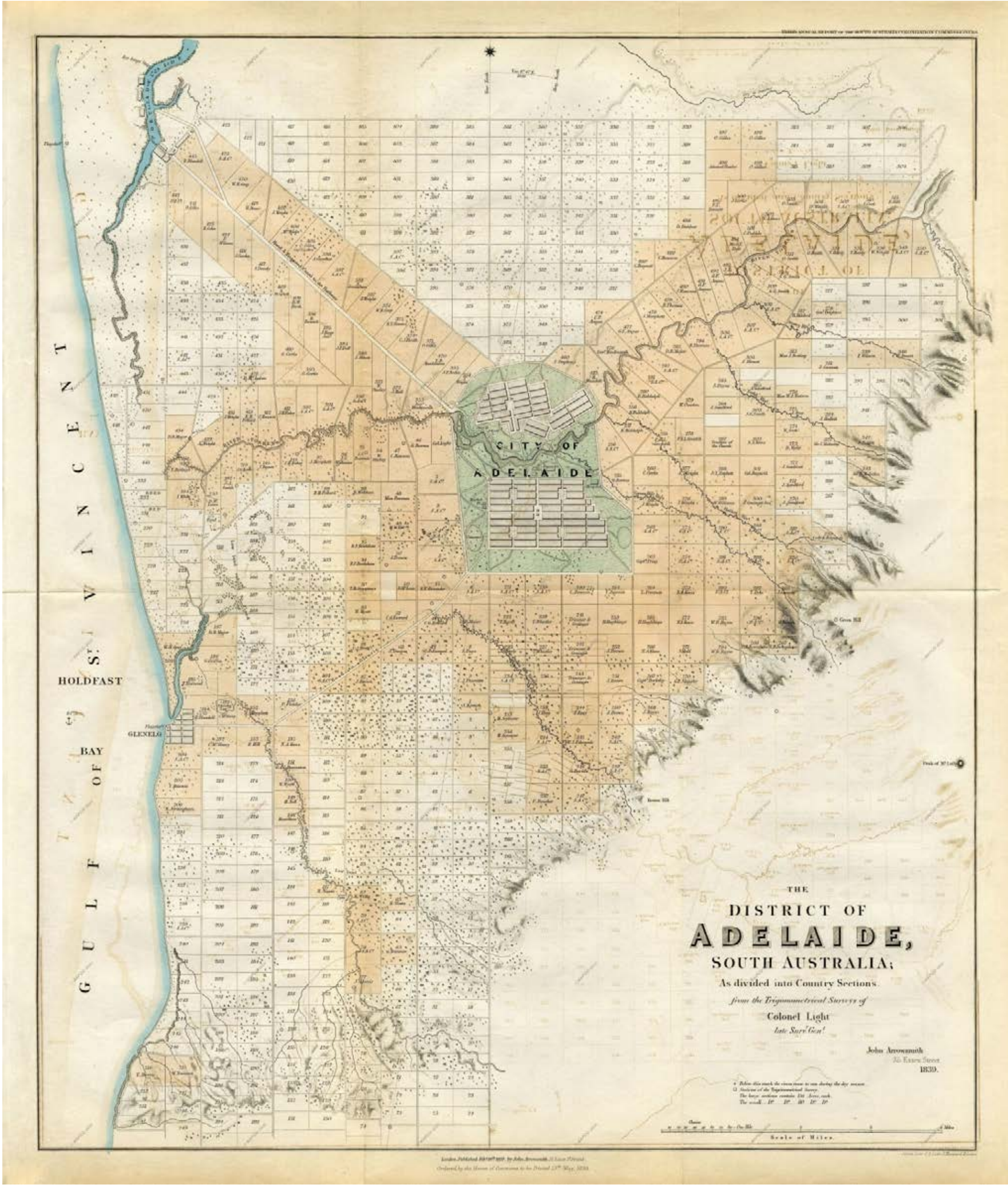



*Figure 2.3 District of Adelaide 1839*

## 2.3 Current Development in the Catchment

The greater part of the catchment is developed as residential housing. Notable exceptions include:

- Hills face area at the southern end of the catchment
- Marion Shopping Centre
- Warradale Army Barracks
- Schools including Brighton Secondary School and Sacred Heart College
- Commercial and shopping precincts along arterial roads
- Shopping precincts along Jetty Road Brighton and Jetty Road Glenelg
- Ovals and other open spaces

The allotments and the ranges of sizes of existing allotments are shown on Figure 2.4.

## 2.4 Development Trends

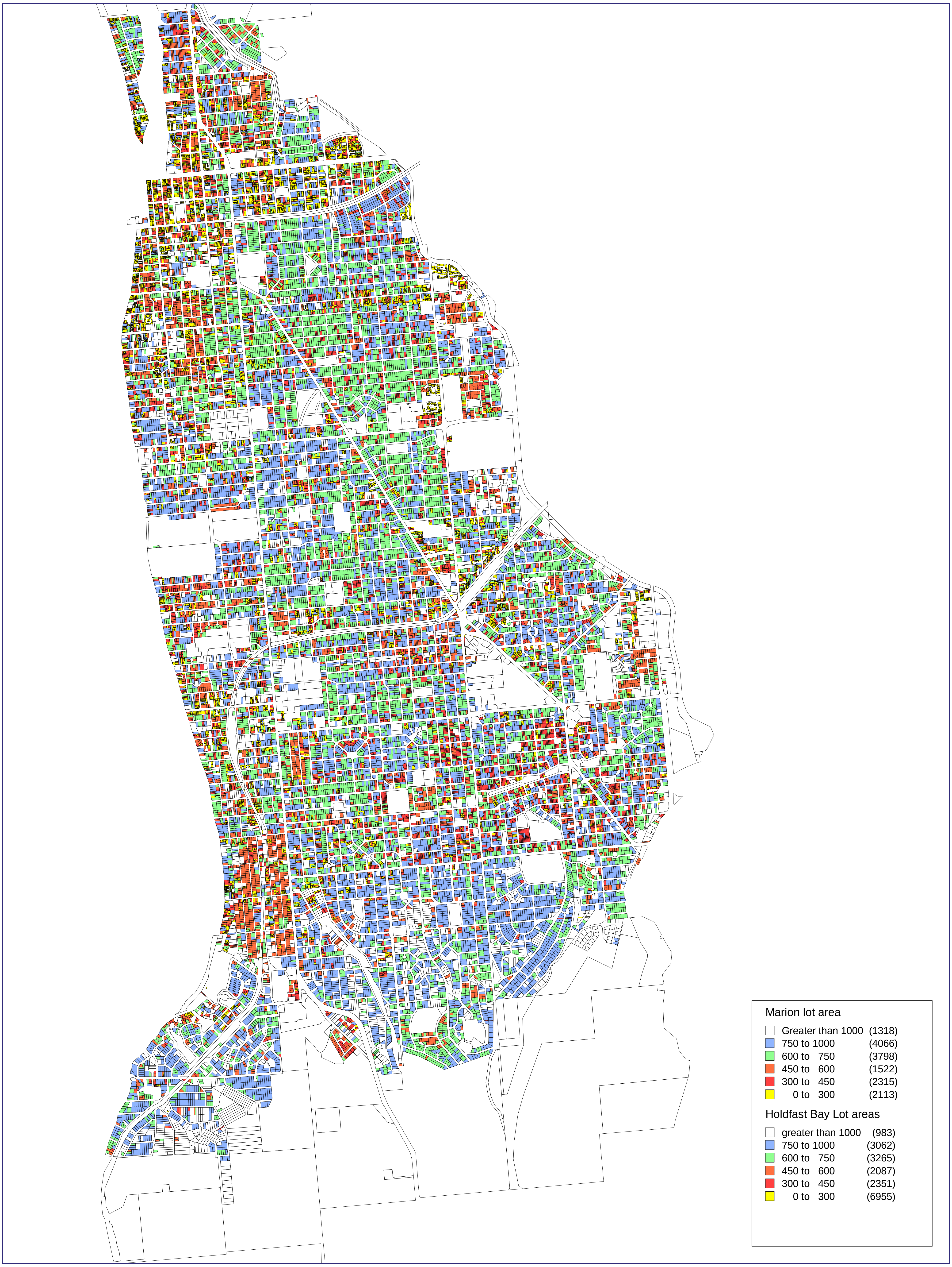
In relation to the detached residential component of the catchment, it is apparent that there is considerable variation in the density of development. This has been explored in the report by Jensen Planning and Design (Jensen Planning and Design, 2011) prepared as a part of this Study. There has been a trend over the last decade for subdivision of land and construction of two or more dwellings on allotments previously containing only one dwelling. This trend is evident across the whole study area, but some areas, particularly the area in Marion north of Seacombe Road appears to have undergone significant redevelopment.

Figure 2.5 and Figure 2.6 demonstrate the increase in impervious areas as a result of the infill development.

As part of the assessment of the hydrology of the catchment, particular examples of development were measured to correlate imperviousness to allotment area, refer to Appendix A of the Floodplain Mapping and Drainage Capacity Assessment Report (Tonkin Consulting, January 2013). The results clearly demonstrate that as allotment sizes reduce, the percentage of the allotment covered in impervious surfaces increases dramatically, and the area of impervious surfaces often changes from less than 50% prior to redevelopment to over 90% post redevelopment. The results are set out in Table 2.1.

*Table 2.1 Residential Impervious Area Percentages*

| Lot Size ($m^2$) | Directly Connected Impervious Area (%) | Indirectly Connected Impervious Area (%) | Total Impervious Area (%) |
|---|---|---|---|
| 175 | 90 | 0 | 90 |
| 200 | 90 | 0 | 90 |
| 300 | 78 | 0 | 78 |
| 400 | 50 | 8 | 58 |
| 500 | 36 | 20 | 56 |
| 600 | 27 | 26 | 53 |
| 700 | 22 | 30 | 52 |
| 800 | 18 | 32 | 50 |
| 900 | 15 | 34 | 49 |

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**Cities of Marion and Holdfast Bay**

## ALLOTMENT SIZES

**Figure 2.4**

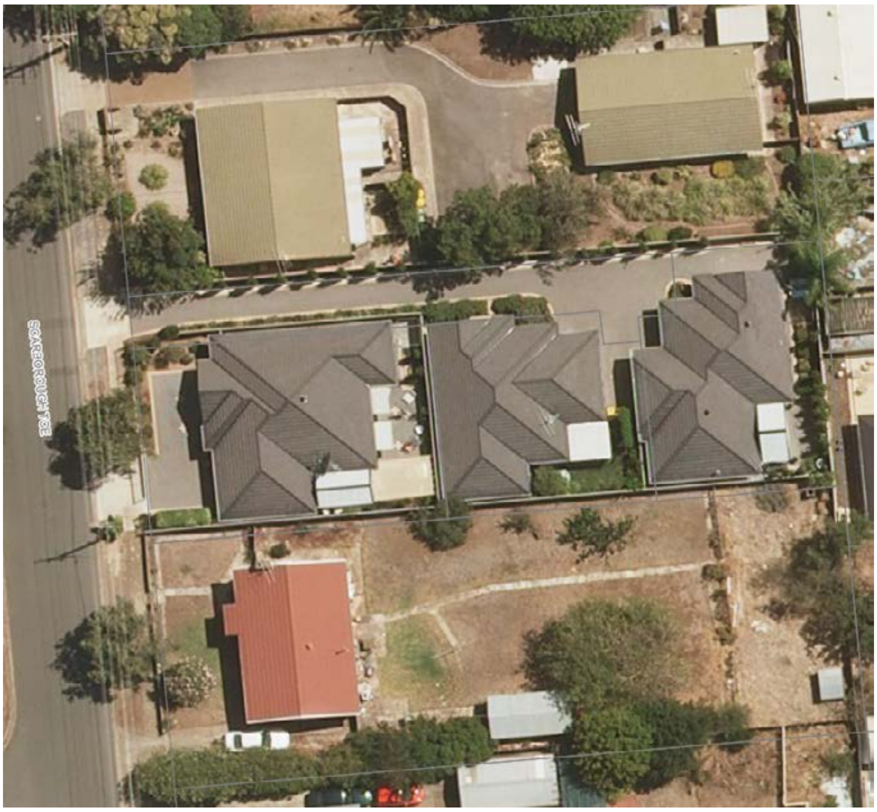


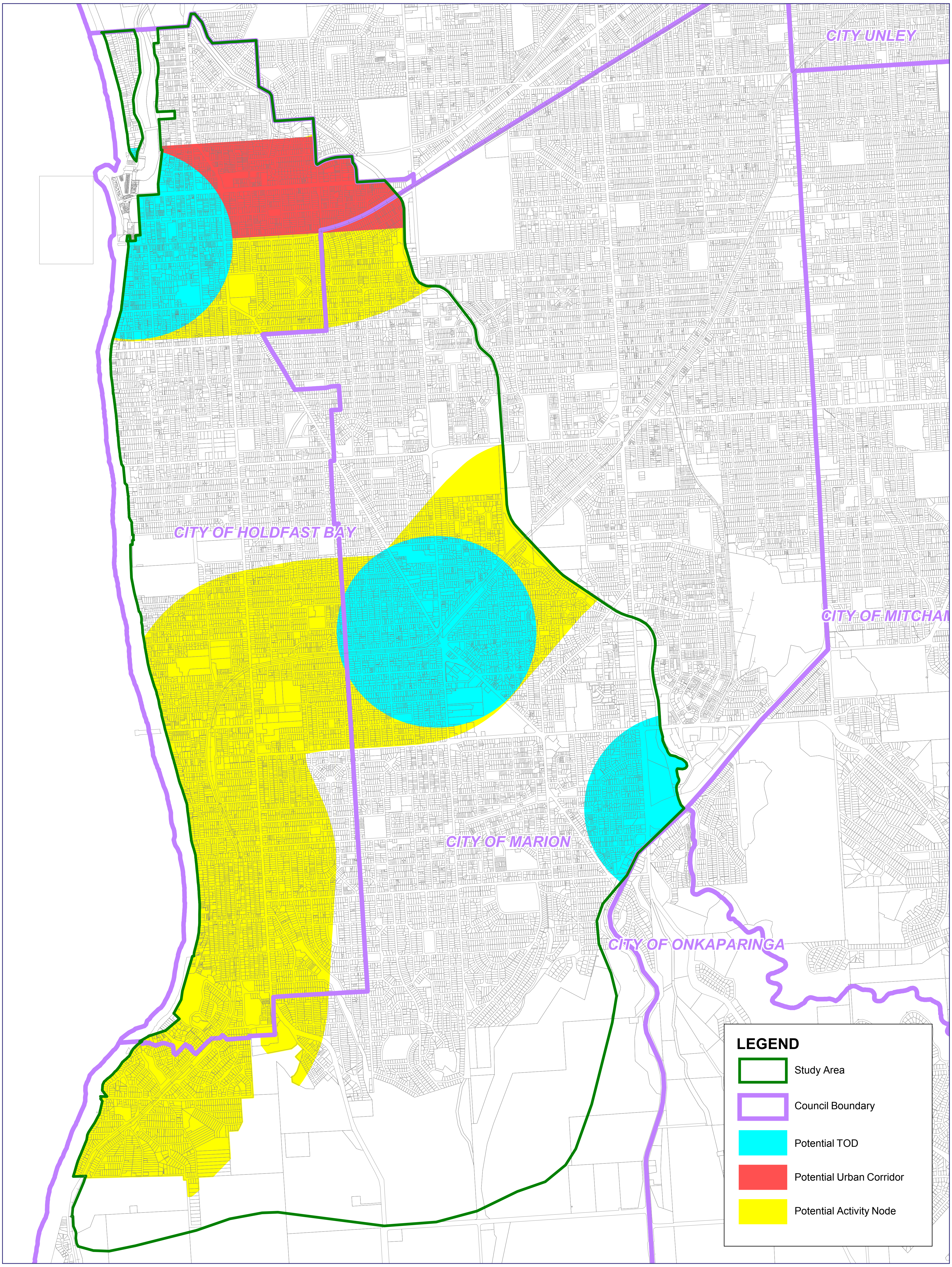

*Figure 2.5 Development Example Dover Gardens*

*Figure 2.6 Development Example Mitchell Park*

Ongoing development within the catchment is impacting on its imperviousness. To understand current and future development trends, Jensen Planning and Design (Jensen Planning and Design, 2011) undertook a comprehensive assessment of past and future development trends. Key findings of this assessment include:

- The growth rate in the number of dwellings in the Marion Council area was 4.3% over the period 2001 to 2006 (0.85% per annum)
- In Marion Council in 2006, over 90% of dwellings were separate houses, semi detached, row or terrace houses, and less than 10% were flats, units or apartments
- The growth rate in the number of dwellings in the Holdfast Bay Council area was 4.5% over the period 2001 to 2006 (0.88% per annum)
- In Holdfast Bay Council in 2006, around 70% of dwellings were separate houses, semi detached, row or terrace houses and around 30% were flats, units or apartments
- Current housing densities are around 15 dwellings/ha
- The current growth rate will yield approximately 14,500 additional dwellings in the Cities of Holdfast Bay and Marion in the next 30 years. As part of the City of Marion lies outside the catchment covered by this Stormwater Management Plan, approximately 9500 additional dwellings will be created within the Study Area at this growth rate (refer Section 2.5 below).
- Current subdivision of existing allotments and redevelopment with single or double story dwellings is yielding around 30 to 40 dwellings/ha, an increase of approximately 20 dwellings/ha
- The State Government's 30 year Plan for Greater Adelaide:
  - allows for an additional 40,500 dwellings in the Cities of Marion, Holdfast Bay, Mitcham and Onkaparinga by 2036
  - aims to increase the ratio of infill to greenfield development from the current 50:50 ratio to 70:30
  - aims to focus redevelopment around major transit routes. Within the study area these are the railway line to Noarlunga, the Glenelg tramline and Anzac Highway
  - promotes transit oriented developments (TODs). In the study area these are at Glenelg, Oaklands Park and Tonsley/Bedford Park
- In the new TODs and other redevelopment zones centred on transit corridors, the 30 year Plan provides for up to 60 to 120 dwellings/ha
- A fully developed TOD would yield 11,250 dwellings on a footprint of approximately 200 ha. If this replaced dwellings currently at 15 dwellings/ha, this represents an additional 8,250 dwellings
- Examples of interstate high density (>100 dwellings/ha) precinct scale redevelopments have been identified which have only approximately 40% impervious surfaces
- Some current policy provisions allow residential allotments to be covered by up to 92% impervious surfaces
- It is likely that on average only 15% of the site area in future residential developments will be permeable

TODs, Potential Urban Corridor Zones and Potential Suburban Activity Zones as envisioned by the State Government's 30 year Plan for Greater Adelaide are shown on Figure 2.7.

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30 YEAR PLAN DEVELOPMENT POTENTIAL

Figure 2.7

The manner in which these trends and planning ambitions for future growth translate into a likely catchment condition in a 30 to 50 year time horizon is uncertain. For example, if all the additional dwellings are contained within TODs (net dwelling increase say 40 dwellings/ha) the 30 year Plan targets for growth in the Cities of Marion, Holdfast Bay, Mitcham and Onkaparinga (40,500 new dwellings) would be achieved within a footprint of around 1,000 ha. Conversely, if the increase in dwelling numbers is achieved through a continuation of the subdivision of lots and the construction of one or two storey dwellings (net increase of 20 dwellings/ha) then over 2,000 ha of land will be required to be redeveloped.

In addition to this, a planned precinct scale redevelopment within a TOD will present opportunities to incorporate open space permeable areas, areas for deep rooted plants and other WSUD initiatives because the housing density is achieved in multi-storey buildings. It will be possible that a TOD could be developed with no net increase in the amount of impermeable surfaces. By contrast, the current development trend of one or two storey development is yielding very little space for deep rooted plants, no WSUD and impermeable surfaces covering 90% or more of allotments. This compounds the impact of the current form of development and in fact may result in not only twice the required area, but perhaps fourfold the area of impervious surfaces.

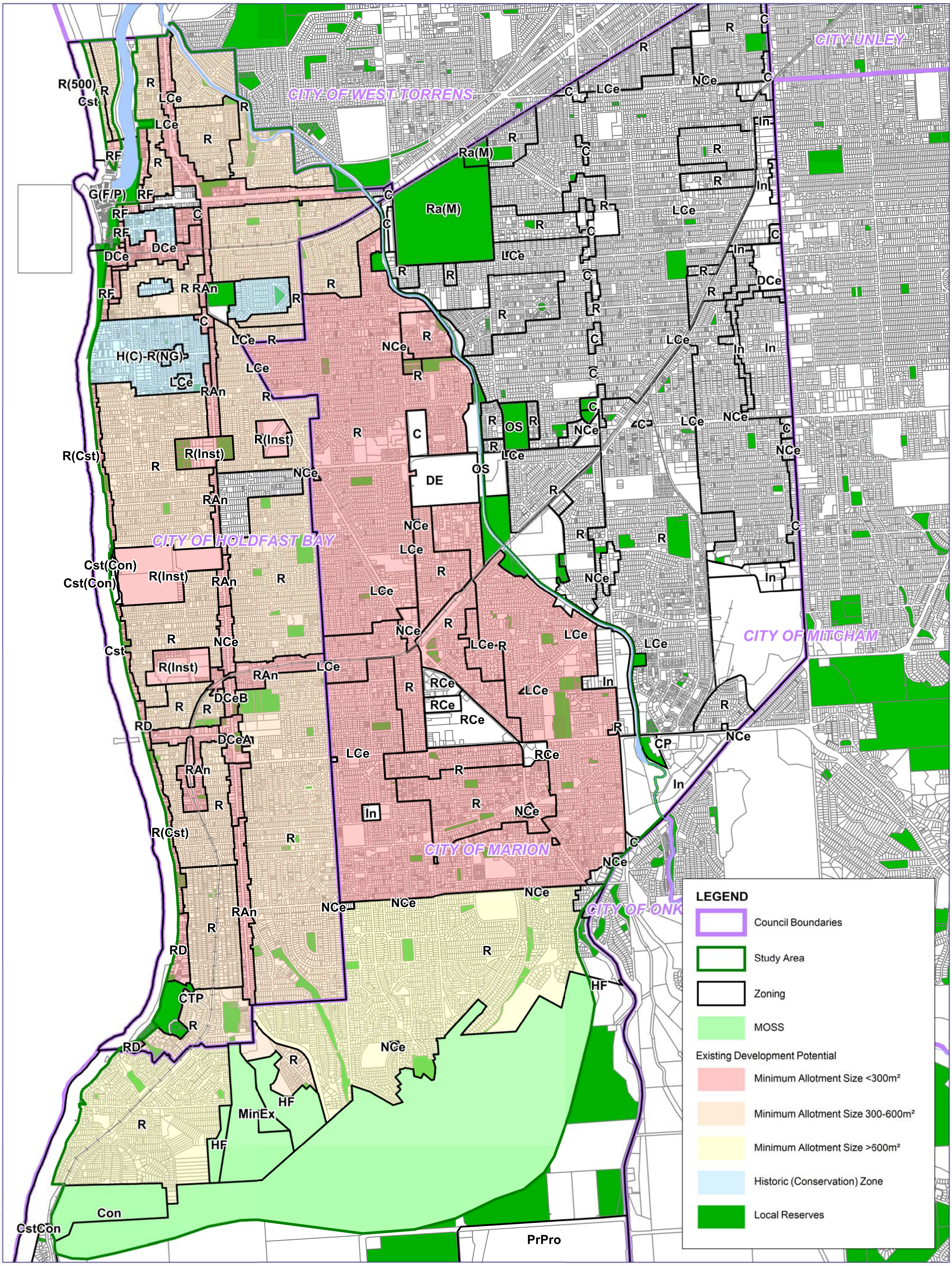
The relative contributions that TOD style redevelopment and traditional allotment subdivision will make to future dwelling growth is impossible to determine with certainty however current planning rules allow for the continuing subdivision of land. Figure 2.8 shows the minimum allotment sizes allowed for in current planning provisions in the catchment. Figure 2.9 shows the areas where subdivision of land can occur. It is clear that there are still a great number of large allotments in the catchment that could be redeveloped.

## 2.5 Possible Future Impact of Development on the Catchment

As part of the development of this Plan, it was required that a hydrological analysis be undertaken for a “future scenario”. This requires an assessment of the imperviousness for the future catchment. As discussed previously, there are a number of possibilities for the future style of development, particularly in relation to the mix between redevelopment on a lot by lot basis or the development on a precinct scale, for example TODs. Additionally, there are a range of WSUD controls that could be implemented, including widespread adoption of detention or retention, on-site infiltration and possibly other measures that will affect the hydrologic performance of the catchment.

Rather than speculate on these possibilities, the hydrology for the future catchment has been based on a projection assuming that current trends continue. This provides a baseline or “business as usual” scenario that can be used to assess the changes to flood risk and other impacts that will occur if no mitigation measures are put in place. To assess future catchment imperviousness, the following assumptions have been made:

- The current growth rate in dwellings, say 0.85% per annum, continues and that this growth rate is achieved through the subdivision of the bigger, say 600 $m^2$ and larger (up to 1000 $m^2$) , residential allotments.
- Any growth rate over and above what occurs through land subdivision is within TODs or other high density corridor zones, but that this is in precinct scale developments that include open space, other permeable surfaces and WSUD initiatives so that the development has no net increase in the percentage of impermeable surfaces when compared to the predevelopment state. For the purposes of this exercise, there is no net change to the imperviousness and these developments can be ignored.



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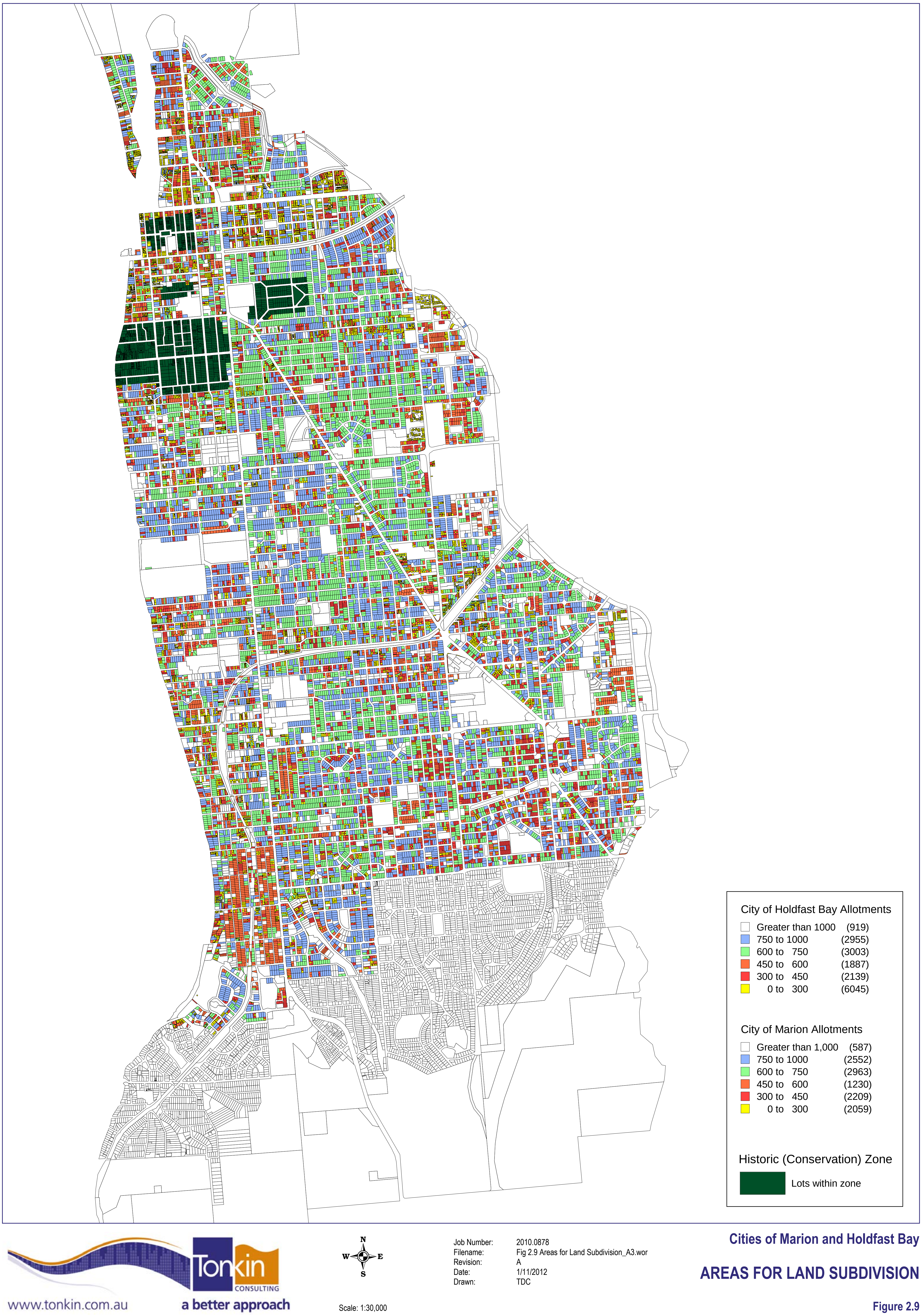
**DEVELOPMENT PLAN PROVISION & MINIMUM ALLOTMENT SIZES**

Figure 2.8

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**Cities of Marion and Holdfast Bay**

**AREAS FOR LAND SUBDIVISION**

**Figure 2.9**

- The current planning provisions that require a minimum allotment size of 600 $m^2$ in the City of Marion south of Seacombe Road remain in force and therefore this area is not redeveloped.
- The current planning provisions in Holdfast Bay in relation to the Historic (Preservation) Zone continue to prevent redevelopment within that zone.
- There is no development in land zoned as MOSS or that is currently reserve.
- There are no requirements for on-site detention or retention.

The allotments that lie within areas that can be redeveloped within the catchment are shown on Figure 2.9. The numbers of allotments that are eligible for redevelopment within these areas (based on their size) are summarised in Table 2.2 and have been categorised into two ranges, these being allotments between 600 and 750 $m^2$ in size and between 750 and 1000 $m^2$ in size.

*Table 2.2 Allotments Eligible for Redevelopment*

| Allotment Size ($m^2$) | City of Holdfast Bay | City of Marion | Total |
|---|---|---|---|
| 750 to 1000 | 2,955 | 2,552 | 5,507 |
| 600 to 750 | 3,003 | 2,963 | 5,966 |
| **Total** | **5,958** | **5,515** | **11,473** |

The current annual growth rate of dwellings within the Study Area has been determined to be 0.85%. This growth rate is based on the total number of dwellings within the catchment which has been determined to be 33,500 using data provided by the Cities of Holdfast Bay and Marion. Over the 30 year planning horizon, at an annual growth rate of 0.85%, this number will increase to about 43,000 (an increase of 9500 dwellings).

For the purposes of this exercise, it has been assumed that 3000 allotments in the 600 to 750 $m^2$ range are redeveloped and each allotment is divided into two. This will yield an additional 3000 dwellings. Additionally, it has been assumed that 3000 allotments in the range 750 to 1000 $m^2$ are divided into three allotments. This will yield an additional 6000 dwellings, and therefore collectively an additional 9000 dwellings are created in the study area (cf 9,500 at an annual growth rate of 0.85%).

The method of synthesising the effects of this redevelopment on the hydrology of the catchment is described in detail in The Floodplain Mapping and Drainage Capacity Assessment Report (Tonkin Consulting, January 2013). The approximate net effect of this however is to increase the imperviousness of the residential areas in the catchment from approximately 800 ha to 1000 ha, an increase of 25%. Note that this additional 200 ha of impermeable area equates to 220 $m^2$ for each of 9000 additional dwellings which is a reasonable estimate considering the additional roof area and paving associated with each new dwelling.

## 2.6 Impacts of Climate Change on the Catchment

### 2.6.1 Rainfall and Evapotranspiration

A summary of the current predictions for climate changes that potentially impact on the catchment is included in Appendix B of the Floodplain Mapping and Drainage Assessment Report (Tonkin Consulting, January 2013). All of the climate change models are driven by an increase in carbon dioxide levels in the atmosphere and the impact that this has on warming of the global climate. There are principally two areas of uncertainty in the predictions from the models. The first stems from the uncertainty about future levels of carbon dioxide which depend on population, economic development, and adoption of alternative energy technologies. The second uncertainty relates to the scientific uncertainty because different global climate models predict different outcomes. On balance however, predictions indicate a warmer and drier future.

For the purposes of this Plan, the following changes between now and 2050 have been adopted:

- Annual rainfall -10% compared to recent averages
- Potential evapotranspiration +3% compared to recent averages

There is also potentially an impact on rainfall intensity, and it is possible that even with an overall drier climate the incidence of high intensity rainfall may increase. Current predictions in relation to this are less certain than for other climate parameters, and indications for southern South Australia are that rainfall intensity may not significantly change or may even decrease. For the purposes of this Plan, an increase in rainfall intensity of +3% has been adopted which is the figure published in the LGA's Local Government Climate Adaption Program Interim Report, (Local Government Association of South Australia, July 2010)

### 2.6.2 Sea Level

The Brief for the preparation of this Plan called for the modelling of future scenarios to adopt a sea level rise of 0.5 m which is consistent with predictions for the next 50 or so years.

## 2.7 Water Levels in Receiving Waters (Gulf St Vincent, Patawalonga Lake and Sturt River)

The drainage systems from the catchment discharge into Gulf St Vincent, Patawalonga Lake or the Sturt River. The level that these water bodies are at may impact on flooding and drainage performance within the catchment and act as boundary conditions for the hydraulic models that assess drainage system performance. A discussion of these is included in Appendix C of the Floodplain Mapping and Drainage Capacity Assessment Report (Tonkin Consulting, January 2013) and are summarised below.

### 2.7.1 Gulf St Vincent Current Conditions

Sea levels in the gulf are driven by two effects. The first of these is the astronomical tide and the second is due to meteorological effects.

The astronomical tide can be predicted based on the position of the sun and the moon. Astronomical tide levels for Port Adelaide (Outer Harbor) are summarised in Table 2.3.

*Table 2.3 Astronomical Tide Levels for Port Adelaide (Outer Harbor)*

| | Tide level to Port datum (1.452 m below AHD Datum) | Tide level to AHD |
|---|---|---|
| Highest astronomical tide | 2.847 | 1.395 |
| Mean high water springs | 2.356 | 0.904 |
| Mean high water neaps | 1.344 | -0.108 |
| Mean sea level | 1.342 | -0.110 |
| Mean low water neaps | 1.340 | -0.112 |
| Mean low water springs | 0.328 | -1.124 |
| Indian spring low water | 0.095 | -1.357 |
| Lowest astronomical tide | 0.037 | -1.415 |

Weather conditions affect actual tide levels due to wind and atmospheric pressure. The difference between the predicted astronomical tide and the actual tide is referred to as the tidal anomaly. Tidal anomalies in excess of plus 1.5 m have been measured at Outer Harbour. A high astronomical tide coincident with a high tidal anomaly causes extreme tides.

An analysis of tide level frequency undertaken as a part of the Port Adelaide Seawater Stormwater Flood Study (Tonkin Consulting, 2005) developed level frequency curves based on 60 years of record from Port Adelaide (Outer Harbor). These levels are summarised in Table 2.4.

*Table 2.4 Tide Level Frequency Analysis for Port Adelaide (Outer Harbor)*

| Average recurrence interval (years) | Tide level (m AHD) |
|---|---|
| 1 | 1.70 |
| 2 | 1.80 |
| 5 | 1.95 |
| 10 | 2.05 |
| 20 | 2.13 |
| 50 | 2.24 |
| 100 | 2.38 |

During the period investigated by the study the highest tide recorded was on 3 July 1981 and reached 2.50 m AHD.

Work undertaken to assess whether there is a relationship between tidal anomalies and significant rainfall in the Adelaide region has failed to determine a strong correlation, and it is generally accepted that tidal anomalies and rainfall are independent events. This means that during a severe rainfall event the tide could be at any level.

Stormwater drainage design and analysis for local drainage systems has generally adopted mean high water springs as a boundary condition for systems that drain into the Gulf. This caters for a storm event of a duration about the same duration as a tidal cycle (a few hours) occurring coincidentally with a period of spring tides. This is considered a reasonable (and slightly conservative) approach for catchments with a similar critical duration and so the boundary condition adopted for the hydraulic models of systems draining to the Gulf for the current condition has assumed a tide level of 0.90 m AHD. This has been applied to all recurrence interval storms.

### 2.7.2 Gulf St Vincent Future Conditions

The Brief for this project requires that a future scenario that includes a 0.5 m sea level rise be modelled. This amount of sea level rise is consistent with predictions for the amount of rise that could occur over the next 50 or so years. This will translate to the astronomical tides listed in Table 2.5.

*Table 2.5 Astronomical Tide Levels for Port Adelaide (Outer Harbor) – With Sea Level Rise*

| | Tide level to AHD current | Tide level to AHD with 0.5 m sea level rise |
|---|---|---|
| Highest astronomical tide | 1.395 | 1.895 |
| Mean high water springs | 0.904 | 1.404 |
| Mean high water neaps | -0.108 | 0.392 |
| Mean sea level | -0.110 | 0.390 |
| Mean low water neaps | -0.112 | 0.388 |
| Mean low water springs | -1.124 | -0.624 |
| Indian spring low water | -1.357 | -0.857 |
| Lowest astronomical tide | -1.415 | -0.915 |

Similarly, adding 0.5 m to the tide level frequency undertaken for Outer Harbor (Tonkin Consulting, 2005) results in the levels listed in Table 2.6.

*Table 2.6 Tide Level Frequency for Port Adelaide (Outer Harbor) – With Sea Level Rise*

| Average recurrence interval (years) | Tide level current (m AHD) | Tide level to AHD with 0.5 m sea level rise added |
|---|---|---|
| 1 | 1.70 | 2.20 |
| 2 | 1.80 | 2.30 |
| 5 | 1.95 | 2.45 |
| 10 | 2.05 | 2.55 |
| 20 | 2.13 | 2.63 |
| 50 | 2.24 | 2.74 |
| 100 | 2.38 | 2.88 |

What is currently a 100 year event will become a 2 to 5 year event after a 0.5 m sea level rise.

Using the same logic as set out for the current situation and adopting the mean high water springs, it is considered appropriate to adopt a level of 1.40 m AHD as the appropriate boundary condition for 1 in 5 and 1 in 100 year flow modelling with climate change.

### 2.7.3 Patawalonga Lake Current Conditions

Under dry weather conditions, the Patawalonga Lake operates as a tidal flushed lake with water entering the southern end of the lake through the Glenelg Gates on a rising tide, and draining via the Barcoo Outlet at the northern end of the lake on a falling tide. The level of the lake under these conditions is maintained between a high water level of approximately 0.6 m AHD, and a low water level of approximately 0.1 m AHD.

Low flows of stormwater arriving at the northern end of the lake from Brown Hill Creek, Sturt River and Patawalonga Creek are diverted directly out to sea through the Barcoo Outlet. When flows exceed the capacity of the Barcoo Outlet either because the flow rate is too great for the outlet to convey, or if the tide level in the Gulf is so high as to reduce the capacity of the outlet, then stormwater is allowed to flow into the lake through the butterfly gates at Weir 2. Under these conditions, the operation of the lake switches from its normal south to north flushing mode to a flood operating mode. Under the flood operating mode, the lake acts as a detention basin storing stormwater until such time as the lake level exceeds the tide level and water is released out of the lake to the Gulf through the Glenelg Gates. Notwithstanding this, the available flood storage at the lake is extremely small relative to the inflows that can occur from the upstream catchments.

Excepting malfunction of the system, when the lake is operating in its flood mode it should never be significantly above the level in the Gulf, the only difference being any hydraulic losses through the lake and the gates themselves.

In 2006 a study was undertaken for the Patawalonga Catchment Water Management Board (Australian Water Environments, January 2006) that simulated the behaviour of the lake over a 100 year period using actual tide levels and simulated stormwater inflows based on rainfall records for the same period. A lake level frequency curve was developed from this analysis which is given in Table 2.7.

*Table 2.7 Lake Level Frequency Curve Analysis Results*

| Average recurrence interval (years) | Lake level based on frequency distribution (mAHD) | AWE's recommended adopted level (m AHD) |
|---|---|---|
| 1 | 1.21 | 1.21 |
| 2 | 1.52 | 1.5 |
| 5 | 1.75 | 1.7 |
| 10 | 1.83 | 1.8 |
| 20 | 1.88 | 1.9 |
| 50 | 1.92 | 2.0 |
| 100 | 1.93 | 2.1 |
| 200 | 1.94 | 2.2 |
| 500 | 1.95 | 2.3 |

The level for each return period is lower than for the Outer Harbor tide frequency analysis referred to earlier. This difference can be explained by the fact that the Patawalonga Lake requires rainfall to fill it to at least the level of the tide, and that there will be some periods of very high tide with little or no rainfall, and so any given tide level in the Gulf is a more frequent event than the same level in the lake.

Because the lake is very small by comparison with its upstream catchment, it requires only modest rainfall to fill it (the AWE report referred to earlier gives a catchment area of 20,835 ha of which 4,721 ha is impervious and a lake area of 15 ha). Such rainfall events will occur on a number of occasions each year and whether or not they convert to an extreme level in the lake will be determined by the state of the tide.

Considering the conclusion set out in the previous section that it is appropriate to use 0.9 m AHD as the level for the tide in the Gulf during assessment of rainfall events of 5 or 100 year average recurrence interval and accepting that the lake level is never significantly greater than the level in the Gulf, it would appear that a level of 0.9 m AHD ought to be adopted for the lake level for hydraulic modelling of rainfall events with a recurrence interval of 5 or 100 years. This can be justified on the basis that, notwithstanding that there needs to be sufficient rainfall to fill the lake, the level of the lake is primarily driven by tides, and that there is no co-relation between rainfall events and tide levels.

It is conceivable that the critical combination of tide and rainfall for some very low-lying parts of the study area which drain to the Patawalonga Lake may be produced by a minor rain event that is sufficient to fill the Lake in combination with an extreme tide. To assess the flooding impacts associated with this type of event, a 1 year ARI rainfall event was also modelled in combination with a 1 in 100 year ARI tide in the low lying areas of the catchment.

### 2.7.4 Patawalonga Lake Future Conditions

On the basis that the level in the Patawalonga is largely driven by tides, it appears reasonable to add 0.5 m to the predicted levels set out earlier, given that only modest rainfall is required to fill the lake to match the downstream tide level. Therefore for the future scenario, a lake level of 1.4 m AHD was assumed.

Adding 0.5 m to the recommended levels results in the level frequency set out below.

*Table 2.8 Level Frequency Recommended Results*

| Average recurrence interval (years) | AWE's recommended adopted level (m AHD) | Recommended levels plus 0.5 m |
|---|---|---|
| 1 | 1.21 | 1.71 |
| 2 | 1.5 | 2.0 |
| 5 | 1.7 | 2.2 |
| 10 | 1.8 | 2.3 |
| 20 | 1.9 | 2.4 |
| 50 | 2.0 | 2.5 |
| 100 | 2.1 | 2.6 |
| 200 | 2.2 | 2.7 |
| 500 | 2.3 | 2.8 |

The consequences of a 0.5 m sea level rise on the Patawalonga Lake are that the frequency of extreme levels in the lake will increase significantly. For example, a level of 2.1 m which is currently a 1 in 100 year event will become a 1 in 2–5 year occurrence, and a level of 2.3 m which is currently a 1 in 500 year event will become a 1 in 10 year event.

Again following the logic set out previously, it is conceivable that for low-lying areas of the study area, tides could influence flooding more so than rainfall and a minor rain event sufficient to fill the lake up to a 1 in 100 year tide level in the Gulf may result in greater flooding than a 1 in 100 year rainfall event if it is coincident with the mean high water springs tide (1.4 m AHD).

### 2.7.5 Sturt River

Flows in the Sturt River will correlate strongly with rainfall over the study area, and notwithstanding a possible lag between the peaks in the local systems and the Sturt River, it is considered appropriate to adopt the 1 in 5 year and 1 in 100 year ARI levels in the Sturt River as boundary conditions for the corresponding hydraulic models.

Levels have been based on the work undertaken by Tonkin Consulting for the City of West Torrens in 2003. This work accounted for the current normal operation of the Barcoo Outlet that is, for a maximum operating level in the diversion pond at the downstream end of the river of EL2.0 m.

The same levels were used for the future scenario as for the current condition since the operation of the Patawalonga Lake is likely to be unchanged.

## 2.8 Soil Types

The soil association map of the Adelaide region has been used to assess soil types in the study area (Figure 2.10). The map shows approximately 50% of the study area is a red brown clay soil with granular structure over clay with variable lime (medium clay). The other 50% of the soils over the site are varied, consisting of sands, alluvial soils and other types of clay.

## 2.9 Soil Permeability

Constant head permeability tests were carried out in reserves at 14 locations across the catchment. Detailed test results are contained in the Discussion Paper on Retention Storage Systems (Tonkin Consulting 2012) prepared as part of the investigations for this Plan.

The test results indicate that within the red brown clay soils that occur across most of the catchment, permeabilities are generally in the range between 1.0 x $10^{-7}$ and 2.5 x $10^{-7}$ m/s. Some higher permeabilities (up to 1.5 x $10^{-5}$ m/s) were recorded in these soils at isolated locations, but the limited number of higher permeability results appears to be indicate that these higher permeabilities were not typical of these soils.

A single test was carried out within the sandy soils of the coastal zone and as expected, this yielded a significantly higher permeability (greater than 3 x $10^{-4}$ m/s).

## 2.10 Hydrogeology

The catchments are located within the Adelaide Plains Sub-Basin, and as shown in Figure 2.11, it spans over four hydrogeological zones which have previously been broadly characterised as follows:

- Zone 1 Covers the basement rocks of the Adelaide Hills and, where fractured, the bedrock can yield useful supplies of groundwater.
- Zone 2 Contains two to four shallow Quaternary aquifers, and two to four Tertiary aquifers.
- Zone 2A Similar to Zone 2 however it occurs in a structurally complex area being bounded by the North and South Splinters of the main Para Fault.
- Zone 3 Contains five to six Quaternary aquifers and three to four, almost flat-lying Tertiary aquifers.

The hydrogeological characteristics of each zone are presented in more detail below.

### 2.10.1 Zone 1

As indicated, bedrock can yield useful supplies where fractured, and for small-scale operations (generally less than 100 ML/yr), Managed Aquifer Recharge (MAR) operations can be viable, as demonstrated by numerous existing small scale schemes in the Adelaide Hills.

However, for large-scale operations there are more risks to the viability of MAR schemes due to:

- generally more limited storage capacity of fractured rock aquifers (as compared to the Tertiary limestones within the Adelaide Plains sub-basin);
- very variable well yield at any site due to the heterogeneous nature of fractured rock aquifers;
- low recovery efficiencies, particularly if the natural groundwater is brackish, and typically compartmentalised, resulting in discrete plumes around each well, rather than amalgamation of plumes into a large plume, as would be expected in a porous limestone aquifer;

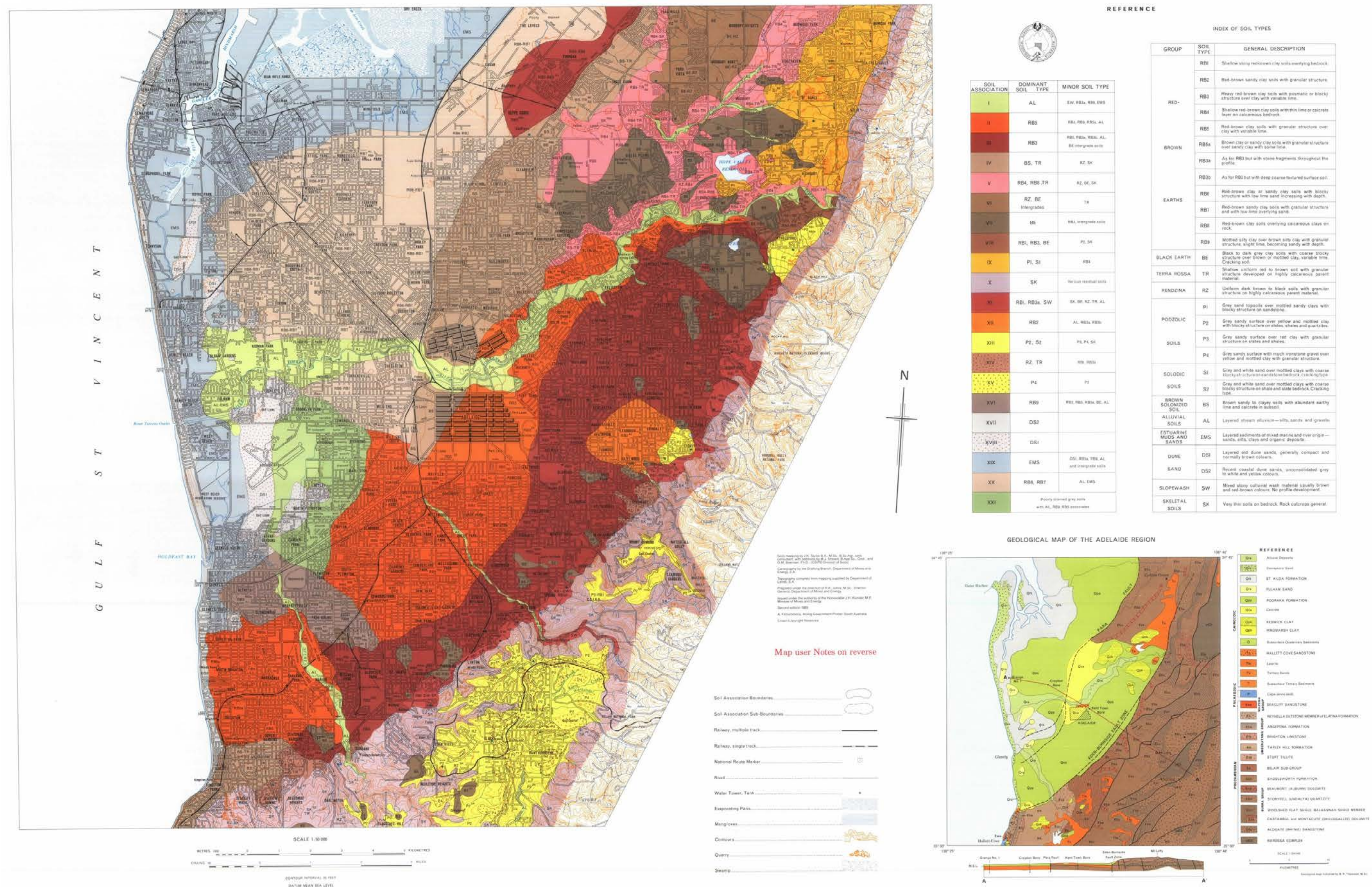
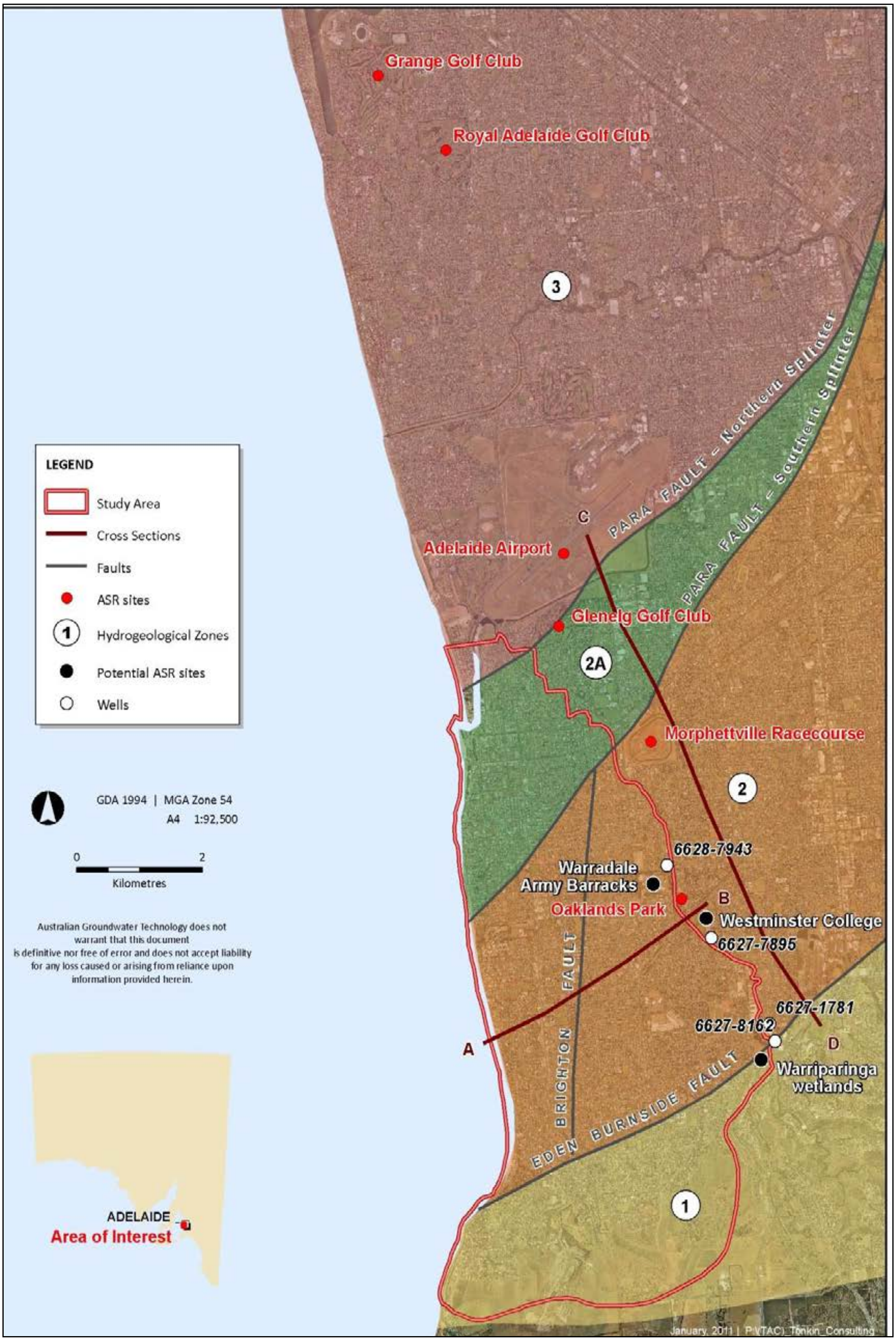



***Figure 2.10 Soil Association Map of the Study Area***




Note: A description of each Hydrological Zone is provided in Section 2.10.

***Figure 2.11 Hydrological Zones***

- higher level of treatment of the injected water required by the Environment Protection Authority (EPA).

The Department for Water drill hole data base was interrogated to locate and derive relevant hydrogeological data from wells drilled within Zone 1 and completed within the bedrock aquifer.

Results are summarised below:

- Salinity is expected to range between 1,500 to 2,500 mg/L.
- Well yields are generally expected to be less than 10 L/s/well, except possibly 10–20 L/s in the vicinity of the Eden Fault, which is conducive to the generation of fractures in the bedrock.
- Injection rates are likely to be less than 5 L/s generally except possibly in wells drilled near the Eden Burnside Fault.
- MAR potential is moderate for small-scale MAR schemes (generally less than 100 ML/yr) and low for large-scale MAR schemes.

### 2.10.2 Zone 2

The shallow Quaternary aquifers are generally thin and low yielding (generally less than 4 L/s) and are not suitable for large-scale MAR.

The upper Tertiary aquifers, commonly known as the T1 (Upper Port Willunga Formation) and the T2 (Lower Port Willunga Formation), offer the best prospects for MAR. Both the existing Morphettville Racecourse ASR scheme and the Oaklands Park ASR scheme occur in this zone.

At the Morphettville Racecourse, the two ASR wells are completed in the T1 aquifer. Injection rates in excess of 25 L/s have been achieved, but this is attributed to the intersection of karstic features. These karstic features (large cavities in the limestone) are probably localised, possibly associated with the adjoining fault.

Further to the south, near the Westminster/Oaklands Park localities, the T1 aquifer is not as well developed and may in fact be absent locally. The Lower Port Willunga (T2) aquifer is the main aquifer in this area, with a minimum thickness of approximately 40 m (well 6627-7895). The influence of the T1 aquifer, where present, is likely negligible.

In the western part of the zone, both the T1 and T2 aquifers are well developed.

The properties of the undifferentiated Tertiary aquifer are summarised below:

- Salinity generally ranges between 1500–3000 mg/L.
- Well yields typically range between 5–10 L/s
- Well injection rates are likely to range between 5–10 L/s
- MAR potential is good

### 2.10.3 Zone 2A

This zone exhibits some geological complexity due to presence of splinter faults associated with the main Para Fault. Aquifers have been displaced vertically through this faulting, resulting in a series of blocks, which however are still hydraulically connected (within the zone).

Both the T1 and T2 aquifers are well developed. In fact, the Glenelg Golf Club ASR scheme has ASR wells completed in both the T1 and T2 aquifers.

Table 2.9 summarises the hydrogeological properties for both aquifers (for the T2 aquifer based on only 1 well located at the Glenelg Golf Club).

*Table 2.9 Hydrogeological Properties for T1 and T2 Aquifers*

| Parameter | Aquifer T1 | Aquifer T2 |
|---|---|---|
| Salinity | 1200–1500 mg/L | 1500 mg/L |
| Well yield | 5–10 L/s | 20–25 L/s |
| Likely well injection rates | 8 L/s | 13–15 L/s |
| MAR potential | Good | Good |

### 2.10.4 Zone 3

This zone is bounded to the south by the North Splinter of the Para Fault. As for the other zones, the shallow Quaternary aquifers are not suitable for MAR.

Both the T1 and T2 aquifers are extensive, however the T2 aquifer is not widely used because the shallower (hence more accessible) T1 aquifer yields adequate supplies of low salinity water and is generally accessed by most wells. The nearby Kooyonga, Grange and Royal Adelaide golf clubs utilise wells completed in the T1 aquifer for watering of the greens/fairways.

In the T2 aquifer, there is evidence of salinity stratification as has been observed at North Glenelg (located some 3 km north-west of the Adelaide Airport), and further north at the Coopers Brewery and Wingfield. At the North Glenelg well, the upper T2 aquifer exhibited a salinity of 1290–1480 mg/L Total Dissolved Solids (TDS), whilst the lower strata yielded more saline water at 6,700–8,000 mg/L TDS.

Hydrogeological properties are summarised in Table 2.10 for each aquifer.

*Table 2.10 Hydrogeological Properties*

| Parameter | Aquifer T1 | Aquifer T2 |
|---|---|---|
| Salinity | 1000–1200 mg/L | 1200 mg/L (upper part)<br>+6,000 mg/L (lower part) |
| Well yield | 10–15 L/s | 15–25 L/s |
| Likely well injection rates | 8 L/s | 13 –15 L/s |
| MAR potential | Good | Good |

# 3 Description of All Known Existing Stormwater Assets

## 3.1 South Western Drainage Scheme

The South-Western Drainage Scheme was designed in the 1960s and has progressively been installed across the Cities of Marion and Holdfast Bay since that time. The scheme consists of over 30 km of underground collector and lateral drainage pipes, with several large outfalls to the coast.

Both Councils have mapped the location, size and condition of their drainage assets in their respective GIS databases.

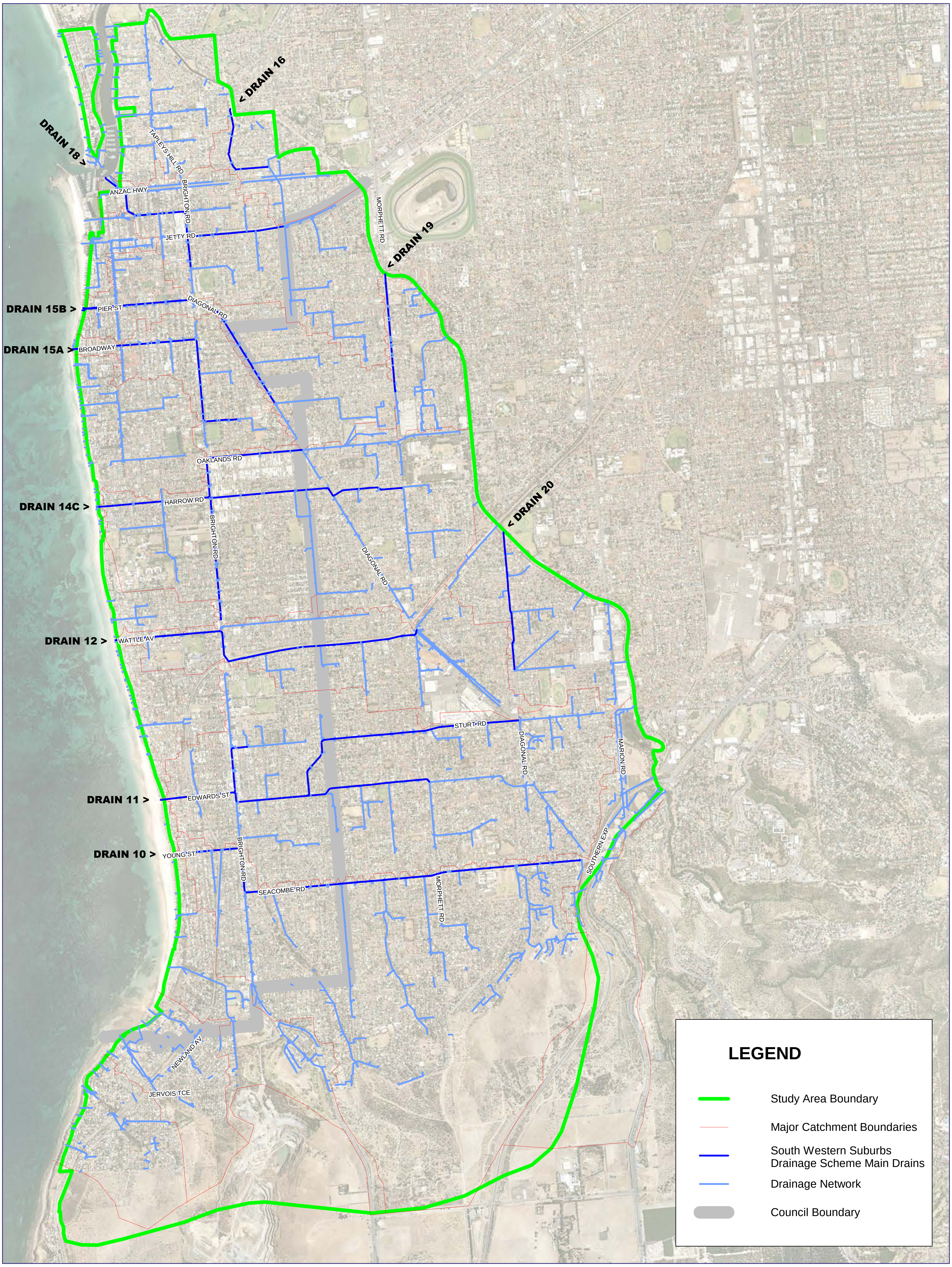
Originally, the design ARI for this system was a 1 in 5 year storm event, however over time the construction of infill and other development has reduced the standard of the existing pipe system.

The existing drainage assets are shown on Figure 3.1.

## 3.2 Water Quality Devices

The existing drainage scheme includes over twenty gross pollutant trap (GPT) devices of various sizes with a range of contributing catchment areas. There are also a small number of other minor water quality devices (i.e. ASR pits, oil and grease separators) serving small sub-catchments.

Locations of the existing water quality infrastructure are shown on Figure 3.2.



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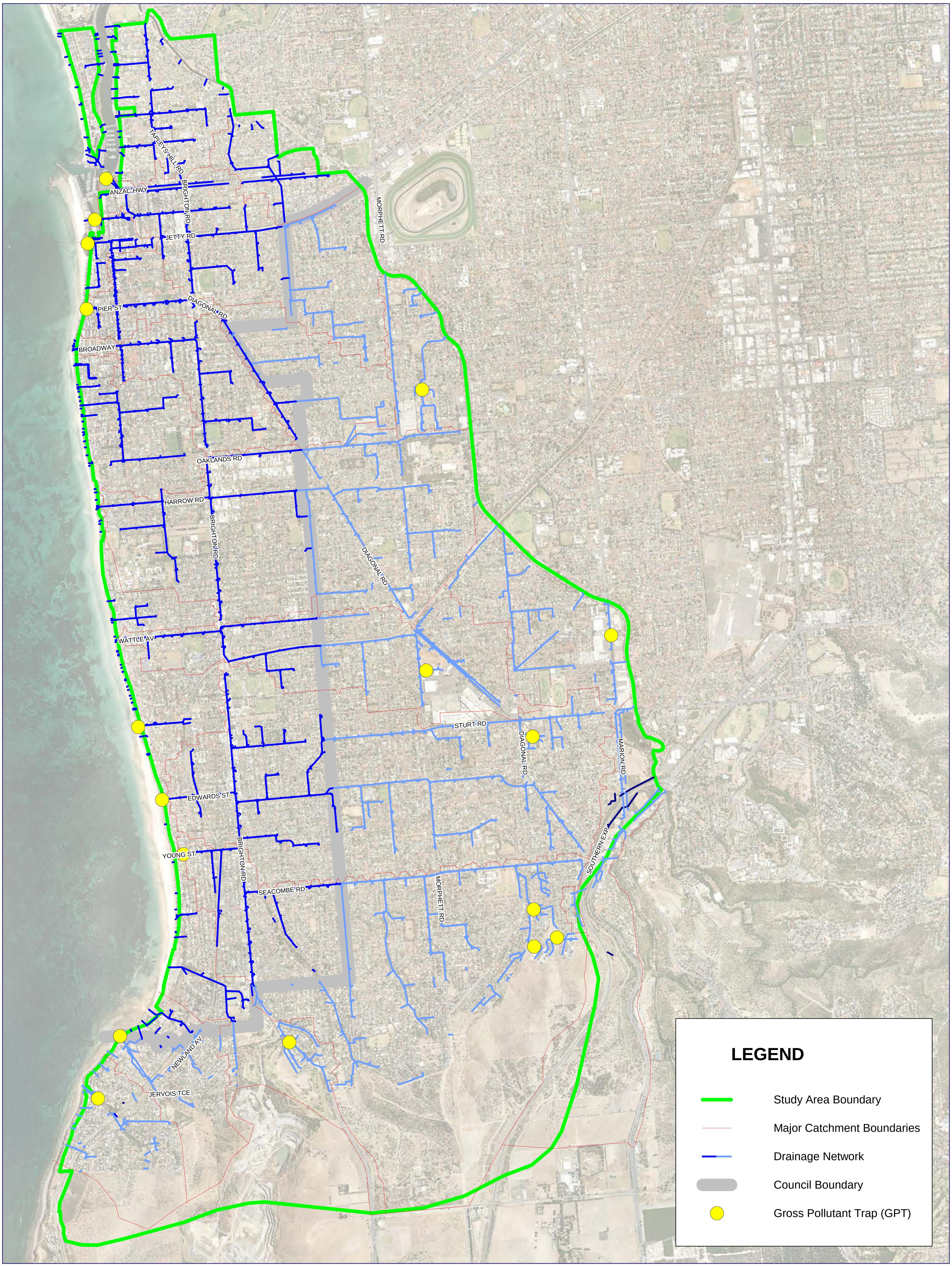
**Cities of Marion and Holdfast Bay**

**EXISTING DRAINAGE NETWORK**

**Figure 3.1**



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**Cities of Marion and Holdfast Bay**

**GROSS POLLUTANT TRAPS**

**Figure 3.2**

# 4 Stormwater Runoff – Quantity

To investigate issues associated with the quantity of stormwater runoff, extensive hydrological and hydraulic modelling was undertaken. These investigations are set out in detail in the Floodplain Mapping and Drainage Capacity Assessment Report (Tonkin Consulting, January 2013) and the key inputs and outcomes are summarised in this section.

## 4.1 Input Data

### 4.1.1 Drainage Network

GIS data of the pipe network was obtained for the catchment from the Councils. This data, which comprised approximately 5,880 pipe and culvert elements, was comprehensively reviewed and updated to ensure its fidelity. The data did not include levels or grades and these were synthesised from terrain data.

### 4.1.2 Terrain Modelling

A digital terrain model was developed based on photography captured in January 2010.

### 4.1.3 Hydrology

#### Rainfall Parameters

The rainfall Intensity-Frequency-Duration (IFD) data were taken from the Australian Bureau of Meteorology for the catchment centroid. The parameters used to generate the IFD data are shown in Table 4.1 below.

*Table 4.1 Rainfall Intensity Frequency Duration Parameters*

| Parameter | 2 year average recurrence interval | 50 year average recurrence interval |
|---|---|---|
| 1 hour Rainfall Intensity (mm/hr) | 17.04 | 34.99 |
| 12 hour Rainfall Intensity (mm/hr) | 3.37 | 6.60 |
| 72 hour Rainfall Intensity (mm/hr) | 0.87 | 1.47 |
| Average Skew Coefficient | 0.6 | |
| Short Duration Geographic Factor F2 | 4.47 | |
| Short Duration Geographic Factor F50 | 14.98 | |
| Latitude | -35.0188 | |
| Longitude | 138.5280 | |

As discussed in Section 2.6.1, rainfall intensities were increased by 3% for the future scenario.

#### Rainfall Loss Parameters

Hydrographs were created using an ILSAX routine (time-area method) for each sub-catchment, based on the rainfall temporal data, loss model and the time of concentration specific to that sub-catchment. The adopted rainfall loss parameters are presented in Table 4.2. These losses have been derived by calibration to recorded events in other catchments in and around Adelaide.

*Table 4.2 Rainfall Loss Parameters*

| Parameter | Unit | Value |
|---|---|---|
| Paved (impervious) area depression storage | mm | 1 |
| Supplementary area depression storage | mm | 1 |
| Pervious area depression storage for urban areas | mm | 45 |
| Pervious area depression storage for rural catchments | mm | 30 |
| Continuing loss (all pervious areas) | mm/hr | 3 |

### Impervious Area Percentages

The adopted impervious area percentages are outlined in Table 4.3, Table 4.4 and Table 4.5.

The values for the future scenario were increased to synthesise the effects of subdivision of the larger lots and the corresponding increase in imperviousness of the catchment. The figures are applied to the lots as they currently exist. This is explained in more detail in the Floodplain Mapping and Drainage Capacity Assessment Report.

*Table 4.3 Impervious Area Percentages – Residential Existing Scenario*

| Lot Size ($m^2$) | Directly Connected Impervious Area (%) | Indirectly Connected Impervious Area (%) | Total Impervious Area (%) |
|---|---|---|---|
| 175 | 90 | 0 | 90 |
| 200 | 90 | 0 | 90 |
| 300 | 78 | 0 | 78 |
| 400 | 50 | 8 | 58 |
| 500 | 36 | 20 | 56 |
| 600 | 27 | 26 | 53 |
| 700 | 22 | 30 | 52 |
| 800 | 18 | 32 | 50 |
| 900 | 15 | 34 | 49 |

*Table 4.4 Impervious Area Percentages – Residential Future Scenario*

| Lot Size ($m^2$) | Directly Connected Impervious Area (%) | Indirectly Connected Impervious Area (%) | Total Impervious Area (%) |
|---|---|---|---|
| 175 | 90 | 0 | 90 |
| 200 | 90 | 0 | 90 |
| 300 | 78 | 0 | 78 |
| 400 | 57 | 14 | 71 |
| 500 | 57 | 14 | 71 |
| 600 | 57 | 14 | 71 |
| 700 | 57 | 14 | 71 |
| 800 | 57 | 14 | 71 |
| 900 | 57 | 14 | 71 |

*Table 4.5 Impervious Area Percentages – Non-Residential*

| Land Use | Directly Connected Impervious Area (%) | Indirectly Connected Impervious Area (%) |
|---|---|---|
| Units and flats | 85 | 5 |
| Commercial and light industrial | 75 | 5 |
| Rural | 0 | 0 |
| Urban open space | 0 | 0 |
| Main roads | 85 | 0 |
| Local roads | 60 | 10 |

## 4.2 Floodplain mapping

### 4.2.1 TUFLOW Model

A detailed one dimensional and two dimensional (1D/2D) model was created using the Two-dimensional Unsteady FLOW (TUFLOW) software package to simulate storm events within the catchment and generate flood inundation maps. The model encompassed an area of 31 km$^2$. The extent of the model is shown in Figure 4.1.

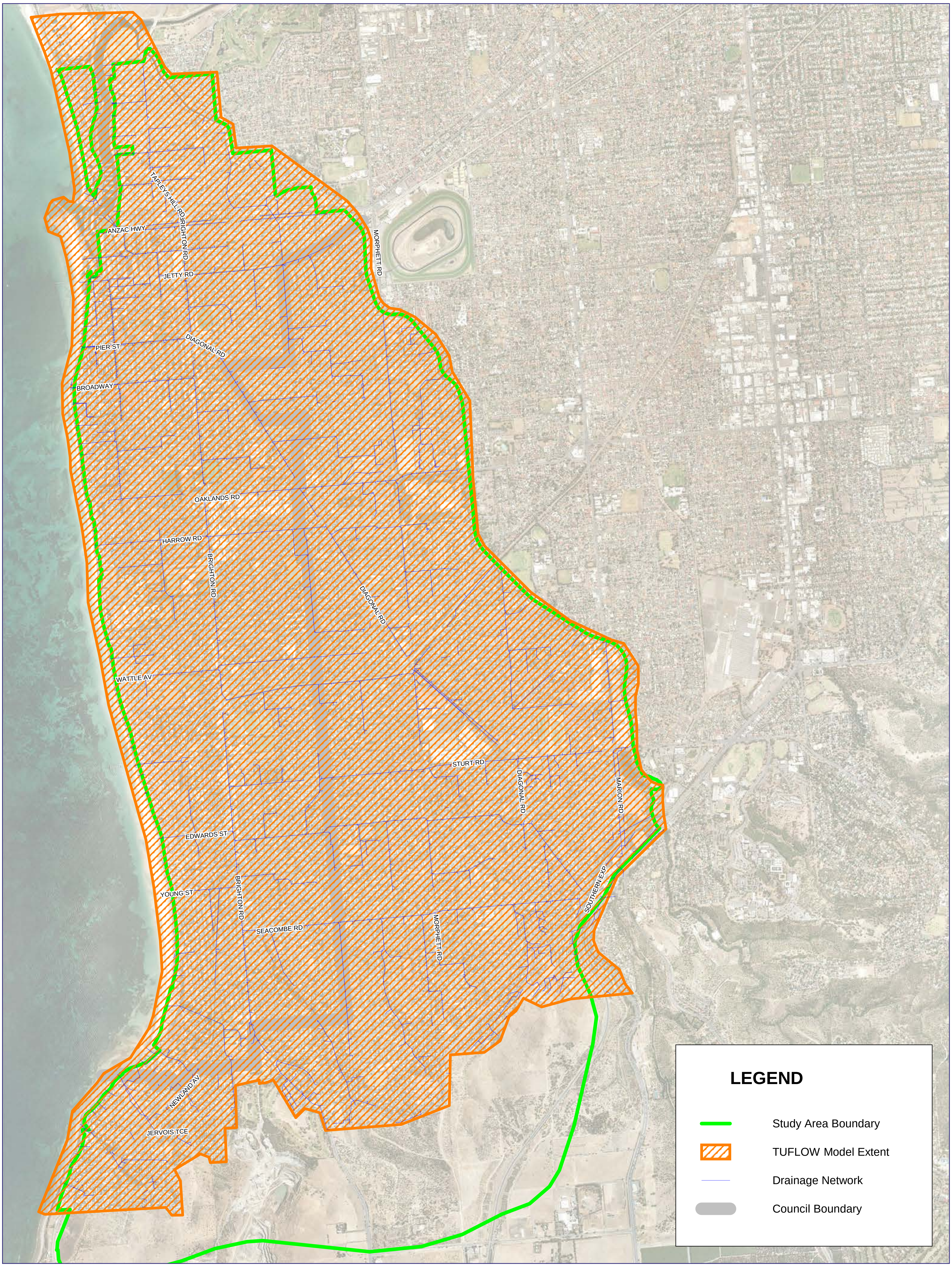
TUFLOW simulates depth averaged, two dimensional free surface flows such as those that occur from floods and tides (WBM Oceanics Australia Pty Ltd, 2010) and is able to dynamically link both 1D, i.e. pipes and channels, and 2D, i.e. surface flow, flow domains. The model was set up with 4 m x 4 m cells.

### 4.2.2 Roughness Coefficients

The TUFLOW model utilises a GIS layer of roughness coefficients (Manning's n values) to define the bed resistance used in calculating the flow and hence the water depth at any location within the model domain. The Manning's n roughness coefficients used in the models are shown in Table 4.6.

*Table 4.6 Adopted Manning's n Roughness Coefficients*

| Type of land use | Manning's n |
|---|---|
| Houses/Residential areas, obstructions to flow | 0.20 |
| Medium density residential and commercial | 0.30 |
| Parklands with scattered trees | 0.045 |
| Grassed areas and bare ground | 0.035 |
| Roads | 0.030 |
| Concrete channels, culverts and pipes | 0.013 |

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**Cities of Marion and Holdfast Bay**

**EXTENT OF TUFLOW MODEL**

**Figure 4.1**

### 4.2.3 Boundary Conditions

As discussed in Section 2.7, boundary conditions set for the model were as listed in Table 4.7.

*Table 4.7 Adopted Boundary Conditions*

| Event | Gulf St Vincent and Patawalonga Lake boundary | Sturt River boundary |
|---|---|---|
| Existing 5 year ARI | 0.9 m AHD | 5 year ARI level |
| Existing 100 year ARI | 0.9 m AHD | 100 year ARI level |
| Future 5 year ARI | 1.4 m AHD | 5 year ARI level |
| Future 100 year ARI | 1.4 m AHD | 100 year ARI level |
| Existing 100 year tide | 2.38 m AHD | – |
| Long-term 100 year tide | 2.88 m AHD | – |

## 4.3 Inundation and Hazard Maps

The TUFLOW model was run for twenty design rainfall events as set out in Table 4.8 below. An analysis of the flooding extents and peak flows at the downstream ends of the main drains confirmed that the critical durations had been identified for each storm event. The critical durations resulted in the maximum flood depth at the downstream end of the model (i.e. ponding locations behind the dunes and railway). The critical durations are also presented in Table 4.8.

*Table 4.8 Critical Durations*

| Event | Storm durations modelled | Critical duration |
|---|---|---|
| 5 year ARI existing | 1hour, 3hour, 6hour, 9hour, 12hour | 1 hour* |
| 5 year ARI future | 1 hour, 3 hour, 6 hour, 9 hour, 12 hour | 1 hour* |
| 100 year ARI existing | 1 hour, 3 hour, 6 hour, 9 hour, 12 hour | 6 hour* |
| 100 year ARI future | 1 hour, 3 hour, 6 hour, 9 hour, 12 hour | 6 hour* |

Output from the models was processed and two series of maps were produced:

- Inundation Maps which show the maximum depth at each point in the floodplain.
- Hazard Maps which show the hazard at each point on the floodplain.

The hazard has been classified in accordance with categories set out in SCARM Report 73 (CSIRO, 2000). These hazard classes are defined based on the maximum velocity and depth at each point in the floodplain, as shown in Figure 4.2. Hazard maps were generated for the 100 year ARI storm event for both the current and future cases.

The full series of maps is included in Appendices A, B, C and D.

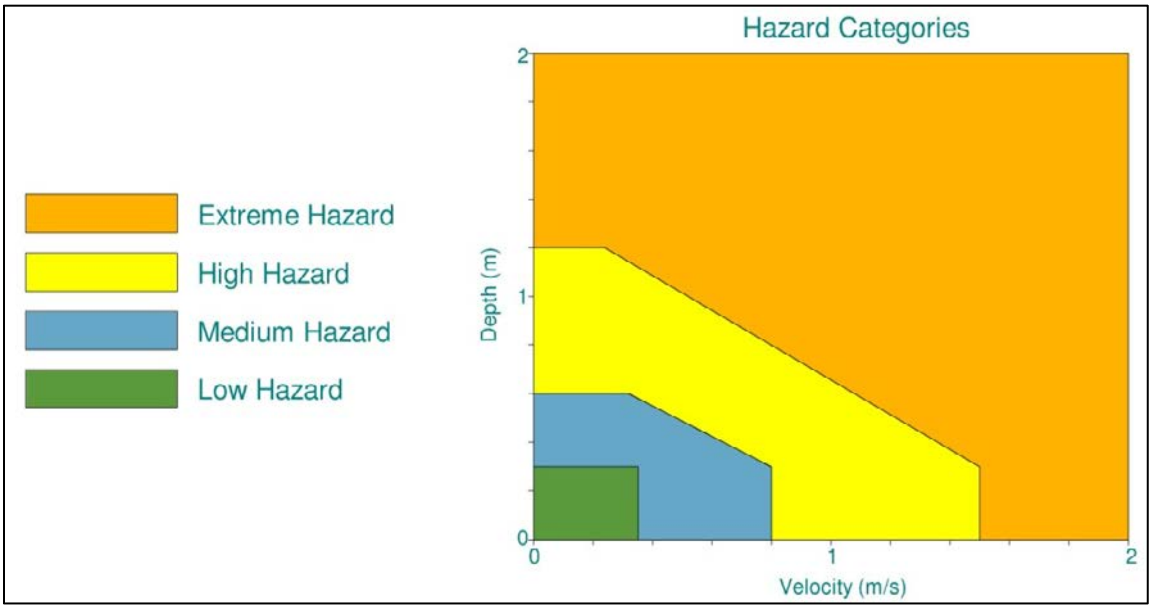



*Figure 4.2 Hazard Categories*

As noted in Section 2.7, high tides combined with storm surge could affect low-lying land behind the dunes and around the Patawalonga Lake. For flooding of the Patawalonga Lake to occur, coincident with a 100 year ARI tide, either of the following would have to occur:

- Failure of the Glenelg Gates to hold back the tide.
- Sufficient rainfall within the Brown Hill Creek or Sturt River catchments to cause the lake level to rise at the same time as the tide.

Consequently, the combined probability of this scenario occurring would be less likely than the 100 year ARI event, but notwithstanding this, the maps include an inset that shows the extent of inundation and the hazard if a 100 year ARI tide level were to occur in combination with a 1 year ARI rainfall.

## 4.4 Flood Damages Assessment

### 4.4.1 Introduction

Flood damage cost estimates are carried out to quantify the cost to the community of a flood event. Flood damages can be used to determine the cost effectiveness of drainage upgrade works.

As outlined in the SCARM report 73 (CSIRO, 2000), flood damages can be classified into two categories: tangible and intangible.

Tangible damages are the financial cost to recover from the flood event. These include direct damages such as the cost to repair or replace damaged property and possessions; and indirect damages such as the loss of wages and cost of the clean-up and recovery exercise.

Intangible damages are due to the physical and emotional stress resulting from the flood event. Intangible damages can be significant, but it is very difficult to meaningfully estimate these damages in financial terms, even though they can represent one of the most significant impacts of a flood.

The flood damage cost estimates undertaken for this study only seek to estimate the direct tangible damages and in that sense can be considered to be a lower bound estimate.

For the purposes of this study, the land that is at risk of flood damage within the catchment has been divided into two categories: Residential (Residential, Units & Historical) and Business (Commercial, Business, Industry & Institution). Reserves, Hills Face and public open space were not included in the damage cost estimates. The flood damage cost estimates are presented for each category and also classified as above or below floor level.

### 4.4.2 Methodology

There are approximately 36,000 parcels of land in the catchment. Flood depths were taken at the centroid of each parcel as a representation of the flood depth at the building location on each parcel. Approximately 5,000 parcel centroids were found to be inundated to a depth greater than 0.025 m during the 100 year ARI long-term storm event.

#### Finished Floor Levels

For each land use type, finished floor levels had to be assumed in order to estimate the depth of above floor and below floor flooding. No actual floor level information was available for this study, so floor levels were adopted based on experience from other studies (Tonkin Consulting, 2004) and a sensitivity analysis was carried out to assess the impact of the adopted levels. The adopted finished floor levels are shown in Table 4.9.

*Table 4.9 Adopted Finished Floor Levels*

| Land use category | Finished floor level (m above surrounding site) |
|---|---|
| Residential<br>(Residential, Units, Historical) | 0.2 |
| Business<br>(Commercial, Business, Industry, Institution) | 0.1 |

#### Damage Cost Estimating

For each inundated parcel, costs were assigned based on land use and the depth of above and below floor flooding to estimate the financial cost of flood damage.

To determine the cost of damages reference was made to work undertaken as a part of the Brown Hill Keswick Creek Stormwater Management Plan (Worley Parsons Services Pty Ltd, July 2011). As no actual capital value or improved value information was available, values were estimated for each land use. The adopted damage costs for each land category by depth of inundation are given in Table 4.10.

*Table 4.10 Adopted Damage Costs*

| Zone | Cost by flood depth | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 0.025-0.1 | 0.1-0.14 | 0.15-0.19 | 0.2-0.29 | 0.3-0.49 | 0.5-0.99 | 1.0-2.49 | 2.5-4.0 |
| Unit | $2,100 | $3,200 | $3,200 | $27,565 | $29,241 | $35,551 | $69,792 | $154,117 |
| Residential | $3,150 | $4,800 | $4,800 | $41,348 | $43,862 | $53,326 | $104,687 | $231,176 |
| Building | $3,150 | $4,800 | $4,800 | $41,348 | $43,862 | $53,326 | $104,687 | $231,176 |
| Historic | $3,150 | $4,800 | $4,800 | $41,348 | $43,862 | $53,326 | $104,687 | $231,176 |
| Commercial | $18,500 | $34,690 | $70,040 | $122,487 | $225,301 | $454,422 | $1,025,892 | $1,652,097 |
| Business | $18,500 | $34,690 | $70,040 | $122,487 | $225,301 | $454,422 | $1,025,892 | $1,652,097 |
| Institution | $10,500 | $26,221 | $30,671 | $37,366 | $50,833 | $82,644 | $176,536 | $325,719 |
| Industry | $73,500 | $215,008 | $284,114 | $386,619 | $587,482 | $1,034,649 | $2,146,322 | $3,352,862 |

The allotment damages were then summed across the catchment to yield the total damage cost estimates.

### 4.4.3 Damages Assessment

Damage cost estimates were prepared for the 5 year ARI and 100 year ARI storm events for both the existing and the long-term scenarios. A summary of the damage cost estimates is provided in Table 4.11.

*Table 4.11 Damage Cost Estimates*

| Council area | Number of affected properties | | | Damage cost estimates |
|---|---|---|---|---|
| | Total | Below floor flooding | Above floor flooding | |
| ***5 year ARI Existing Scenario*** | | | | |
| City of Marion | 56 | 52 | 4 | $500,000 |
| City of Holdfast Bay | 113 | 108 | 5 | $700,000 |
| **Total** | **169** | **160** | **9** | **$1,200,000** |
| ***5 year ARI Long-term Scenario*** | | | | |
| City of Marion | 248 | 237 | 11 | $1,400,000 |
| City of Holdfast Bay | 421 | 380 | 41 | $3,500,000 |
| **Total** | **669** | **617** | **52** | **$4,900,000** |
| ***100 year ARI Existing Scenario*** | | | | |
| City of Marion | 1470 | 1400 | 70 | $8,300,000 |
| City of Holdfast Bay | 2575 | 2070 | 505 | $35,800,000 |
| **Total** | **4045** | **3470** | **575** | **$44,100,000** |
| ***100 year ARI Long-term Scenario*** | | | | |
| City of Marion | 1836 | 1741 | 95 | $11,000,000 |
| City of Holdfast Bay | 3232 | 2555 | 677 | $46,300,000 |
| **Total** | **5068** | **4296** | **772** | **$57,300,000** |

The flood damage cost estimates suggest a current 5 year ARI cost of $1,200,000. If development takes place as expected based on the projected trends and urban development plan, this can be expected to increase by an additional $3,700,000, if no additional stormwater infrastructure works are carried out and no development controls are put in place.

The damage cost estimates for the 100 year ARI storm events demonstrate that the City of Holdfast Bay as the downstream catchment receives the majority of the flood damages. Works to reduce the deep ponding locations behind the dunes and rail line could greatly assist in reducing these damages.

These costs estimates are the cost to the community rather than the cost to the Council. They can be assumed to be lower bounds as they are only an estimate of the tangible damages, with intangible damages (impacts of physical and emotional stress) not included.

While there were insufficient ARIs analysed to be able to produce meaningful annual average damage estimates, this damage assessment can give an indication of the costs that can be expected to be incurred as a result of flood damage and the viability of drainage upgrade works.

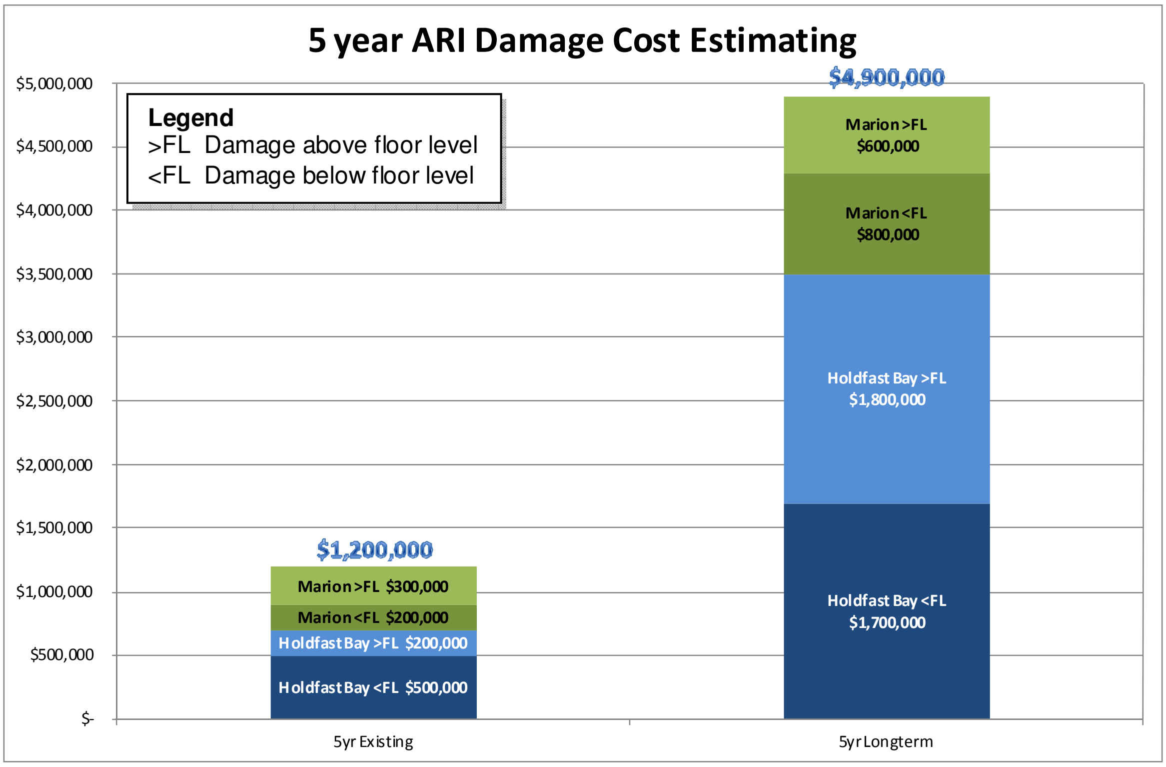



*Figure 4.3 5 year ARI Damage Cost Estimates*

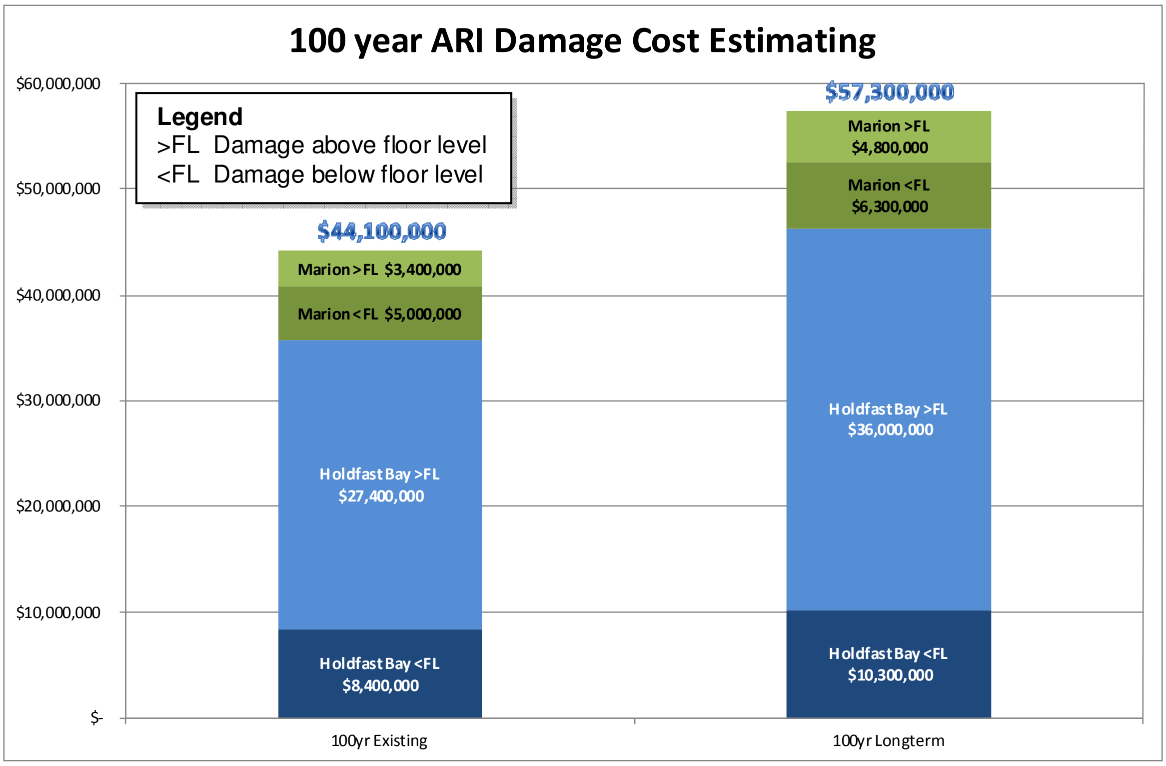



*Figure 4.4 100 year ARI Damage Cost Estimates*

## 4.5 Pipe capacity assessment

### 4.5.1 Introduction

An ILSAX hydrological model of each catchment was developed to assess the standard of the existing drainage systems and to investigate drainage upgrade proposals.

The pipe system used for the ILSAX modelling was the same as developed for the TUFLOW model and this has been described previously. Similarly the sub-catchments and hydrological analysis was as described in previous sections of this report.

Where the grades of drains were extreme (i.e. for the hills face catchments), the grades were limited in the model to a maximum slope of 5%. This was to maintain realistic estimates of pipe capacity in the ILSAX model. In addition to this, once formulation of the drain networks was complete, all drain grades were reduced to 75% of their actual value to account for minor losses through the drainage network (i.e. entry, exit, bend and box losses). As minor losses are not taken into account in the ILSAX model, this provides for a more accurate representation of the drainage network performance.

This modelling is described in more detail in the Floodplain Mapping and Drainage Capacity Assessment Report.

### 4.5.2 Model Assumptions

To assess the capacity of the pipe network, the ILSAX modelling assumed that the inlet pits have an unlimited capacity. There are several areas where the drainage is shown to be adequate, but additional pits may be needed to capture the flow arriving at an inlet. Local effects such as pit blockage will also increase the occurrence of observed flooding at local inlet pits above what is predicted by the hydraulic model.

The ILSAX model was run in “Design mode”. This mode assumes that all flows in a catchment (both surface and pipe flows) are able to reach the downstream extents of the system. Hence, there is no assumed storage in the catchment, and flows are not subject to being ‘held back’ due to natural surface gradients in the system. This approach allows the existing pipe capacity to be assessed based on the assumption that all flows are able to arrive at the upstream inlet of each pipe segment in the catchment. Again this allows the capacity of the pipework to be assessed against an ideal “design” case whereas in reality it is likely that upstream constraints and storage limit the true flows arriving at the downstream parts of the network.

### 4.5.3 Model Results

The ILSAX model was run for the 1, 2, 5, 10, 20, 50 and 100 year ARI storm events to assess the standard of each drain. The model was run in design mode (no restrictions in the pipe network) to provide estimates of the design flow for each pipe for each ARI. The model was then run in existing mode to provide estimates of the capacity for each pipe. The capacity was then compared with the design flows for each ARI to determine the point at which the design flow exceeded the pipe capacity and hence to determine the standard of each pipe. The results were loaded into the GIS pipe database and colour mapped by standard to provide a visual assessment of the pipe network and highlight sections that may require a more detailed assessment and future upgrade works.

The capacity of each pipe is based on the pipe size information obtained from Council and the pipe grade taken from ground surface contours and discounted by 25% to account for losses. While every effort has been made to check all the information provided, the pipe standard information obtained from this study can only be as accurate as the information provided.

The drainage standards map resulting from this work is presented in Appendix E.

## 4.6 Issues Associated with the Quantity of Stormwater Runoff

Based on the hydrologic and hydraulic modelling investigations, issues associated with the quantity of stormwater runoff are as follows:

### 4.6.1 The Minor Drainage System

The minor drainage system is the pits and underground pipes throughout the study area whose primary function is to avoid nuisance flooding and ponding so as to maintain the serviceability and safety of the road network. The original South Western Suburbs Drainage Scheme which drains the majority of the study area was designed to achieve a 1 in 5 year ARI standard.

The modelling of the capacity of the pit and pipe network shown on maps included in Appendix E indicate that many of the pipes do not have the theoretical 5 year capacity when measured against contemporary standards. This is likely to be due to increased imperviousness of the catchment since the system was designed in the 1960s and also due to changing design storm intensities which have been refined since that time.

The floodplain modelling shown on the maps included in Appendix A does not indicate any major issues and demonstrates that by and large the 5 year storm is contained within roadways without causing serious flooding. To this extent the pipe system is reasonably adequate to cater for the 5 year ARI storm. This apparent contradiction with the pipe capacity can be explained by the fact that storage of runoff is occurring throughout the road network, which is acting to attenuate flows in the drains to less than the theoretical flows which would occur if there were no pit or pipe capacity constraints in the upstream pipe network.

There are no compelling reasons to indicate that the existing pipe network ought to be upgraded purely to cater for the 5 year flows given the enormous cost that would be entailed in such an upgrade.

There are however a number of deficiencies in the pipe network that are largely due to the presence of long overland flow paths and large subcatchments in which there is an absence of pipes and pits.

An analysis of the 5 year flows arriving at all the pits was undertaken to highlight those pits where subcatchment flows are far in excess of the pit capacity and where drainage issues are likely to be a problem. This analysis is shown graphically in Figure 4.5. Most of these problems are at the upstream ends of the pipe networks, suggesting a need to extend the pipe and pit network further.

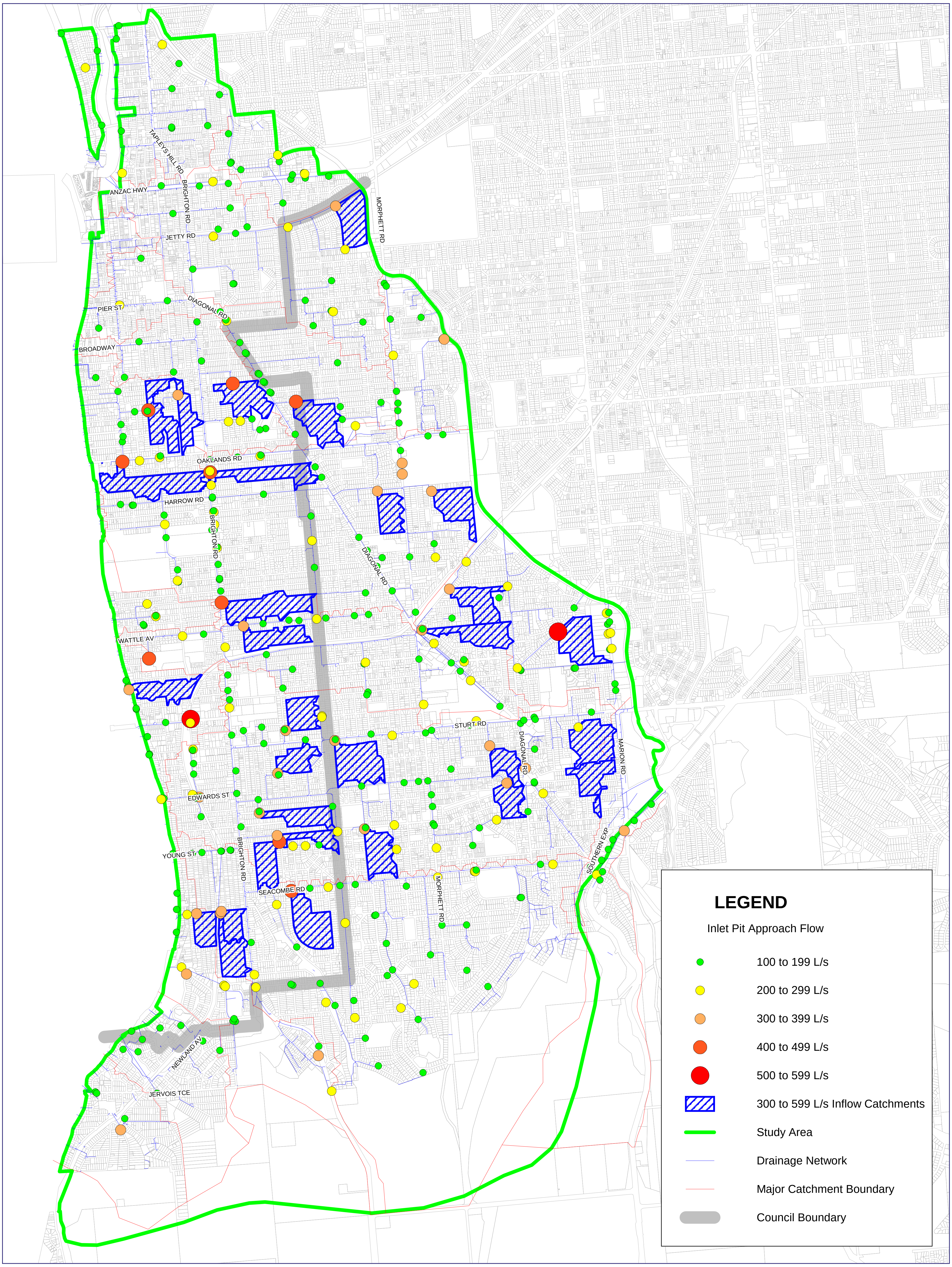
### 4.6.2 The Major Drainage System

The major drainage system includes the minor system but also includes the roads, open spaces, water courses and other overland flow routes which become engaged during a storm that exceeds the capacity of the minor system. The primary purpose of the major system is to prevent flooding that causes property damage or that threatens the safety of people in the floodplain.

The floodplain mapping developed for the study area shows a number of areas where the 100 year ARI flood breaks out of the road network and flows into properties. Mostly this floodwater is shallow, that is less than about 100 or 200 mm deep. Water of this depth will not necessarily cause flooding above the floors of buildings. Certainly if these allotments were to be redeveloped, this depth of water could be managed by setting floors at an appropriate level.

The hazard mapping shows that for the great majority of the floodplain, the hazard presented by flooding is low.

It is not considered cost effective to re-construct the road and drainage network to prevent all property flooding during the 1 in 100 year ARI event although during reconstruction of roads and replacement or augmentation of the drainage network the objective of a 1 in 100 year ARI protection to property ought to be considered.



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Scale: 1:30,000

Job Number: 2010.0878
Filename: Fig 4.5 5 year ARI Inlet Pit Approach Flows_A3.wor
Revision: A
Date: 1/11/2012
Drawn: TDC

**Cities of Marion and Holdfast Bay**

**5 YEAR ARI INLET PIT APPROACH FLOWS**

**Figure 4.5**

There are however some isolated areas in the floodplain where water depths are deeper and where the hazard rating is not low but becomes medium or high. It is considered appropriate that works should be considered in these areas.

### 4.6.3 Changes to Flood Risk Over Time

The floodplain mapping for the future scenario 1 in 5 year ARI flood, which assumes no controls over development, shows a noticeable increase in the amount of water in the road network during a 1 in 5 year ARI storm event. Calculated flood loss damages for the 5 year ARI storm increase significantly from $1.2 million to $4.9 million, a 300% increase.

The percentage increase in flooding during the 100 year ARI event is not as great as during the 5 year event, because pervious surfaces are unable to absorb all the water during the intensities experienced in a 100 year ARI event and tend to behave more like impervious surfaces. This behaviour reduces the impact of development at these higher intensities. Nonetheless, there is a significant increase in calculated damages for this event, from $44.1 million to $57.3 million, a 30% increase.

The future scenario has been modelled taking account of three changes, these being a 3% increase in rainfall intensity, increased development within the catchment and a sea level rise of 0.5 m. The sea level rise does not impact on flooding induced by rainfall but has other impacts discussed later.

In terms of the relative contribution towards increased flood risk from increased rainfall intensity and from development, these have not been independently analysed but the following can be concluded:

- Continuing development within the catchment is significantly increasing the areas of impervious surfaces and, in the absence of any controls to retain or detain runoff, will reduce the standard of flood protection across the catchment. Currently the redevelopment of older larger blocks with one dwelling into many smaller blocks can increase runoff volumes by 100% or more.
- The current predicted increase in rainfall intensity of around 3% due to climate change is slight and its impact will be dwarfed by the impact of the increasing imperviousness of the catchment arising from current development trends.
- Stormwater managers need to monitor developments in climate change science and continually reassess predictions about increases in rainfall intensity and should be prepared to adopt an adaptive management approach in regard to this.

## 4.7 Issues Associated with Sea Level

As part of the floodplain modelling a simple assessment was made of the effects of a 1 in 100 year tide on low lying land around the Patawalonga Lake. Such a scenario is conceivable in the event that flows in the Sturt River and Brown Hill catchments are sufficient to cause the Lake to fill to the same level as the tide. No detailed conjunctive probability analysis was carried out to understand the likelihood of such an event.

The results of this modelling are shown on the flood inundation maps included in Appendices A and B. The effects of high sea level are limited to areas east of the lake.

A similar analysis with a 0.5 m sea level rise is also included on the flood inundation maps included in Appendix B for the future scenario. For this scenario, not only does the depth and extent of flooding east of the Lake increase, but some areas behind the dunes south of Glenelg are also potentially affected by sea water that could travel back through the drainage network.

Whilst not specifically an issue for the Stormwater Master Plan, the issue of sea level rise is one that needs to be monitored.

## 4.8 Lorenzin Site

The “Lorenzin Site” is an area of land located to the west of Ocean Boulevard and south of Schofield Road at Seacliff Park. The site has had a number of previous uses, but is currently being considered for residential development.

The site lies within a valley and receives runoff from an upstream catchment approximately 58 ha in size. The floodplain mapping contained in Appendix A shows that even during a 1 in 5 year ARI event, significant overflows will occur through the site and travel toward the intersection of Schofield Road and Newland Avenue. During a 1 in 100 year ARI event, the peak flow through the site was calculated to be 3.7 $m^3/s$.

Apart from the overflows from the upstream catchment, the site itself contains a large amount of impervious area, and this generates significant flows even in minor rainfall events, which exceed the capacity of the drainage inlets at the intersection of Schofield Road and Newland Avenue.

As a result of these inflows, the system downstream of the site within Kauri Parade is shown on the drainage standard maps to have less than a 1 year ARI design standard.

Development of the site will need to be protected from inundation due to the upstream flows and will also provide the opportunity to address runoff from the site and its downstream impact.

This is further considered in Section 7.1.3.

# 5 Stormwater Runoff – Quality

## 5.1 Physical Changes to the Adelaide Coast

The development of a system that rapidly transports stormwater to the marine environment, combined with the impervious surfaces, human activities and industry that has been increasing since European settlement have all significantly altered the quantity, temporal distribution and quality of water discharged to the marine environment.

In addition to stormwater discharges, treated wastewater from Adelaide's wastewater treatment plants (WWTP) has also been discharged into the marine environment.

The cumulative impact of continuous and episodic land based discharges has resulted in a significantly degraded coastal environment. In particular about 5000 ha of seagrass has been lost from the near shore parts of the Adelaide coast. This is of particular concern given the scale of both the direct and indirect effects including loss of biological diversity and increased instability of the sea floor.

The direct discharge of stormwater carries the following pollutants to the marine environment:

- Turbidity which reduces the clarity of water and is implicated in the decline of seagrasses. It also results in a degradation of the aesthetics of the marine environment as well as reducing the safety of bathing by reducing visibility into the water. Fine particulate material can be resuspended during storm events so that its impact lasts for longer than just the initial event.
- Pollutants such as tyre and brake wear products and hydrocarbons from road surfaces
- Litter and debris including leaves and other vegetation
- Nutrients
- Pesticides and chemicals
- Bacteriological contamination which can reduce the quality of bathing waters

## 5.2 Available Data

The work by Wilkinson et al, undertaken for the Adelaide Coastal Waters Study (ACWS) (Wilkinson J, July 2006) was a very comprehensive compilation and analysis of water data for discharges to the Adelaide Coast. The following quantity and quality data are taken from this work. It should be noted that in this work analysis was undertaken on a catchment by catchment basis and the "Coastal Catchments", that is the catchments discharging to the coast north of the Field River and South of the Patawalonga correlate closely to the study area and are referred to in the following discussion.

### 5.2.1 Annual Flows

The estimated mean annual flow volumes for stormwater flows from the Coastal Catchments are listed in Table 5.1 for 10 year intervals from 1945 to 2005.

The estimates in mean annual flow reflect an increasingly developed catchment and the fall in the last decade reflects the impact of reduced rainfall due to drought.

In addition to the increasing discharge of stormwater, Wilkinson also concluded that there has been a change to the seasonality of flows, where in urbanised areas the stormwater season has broadened, that is the summer flows have shown a greater increase than the winter flows.

*Table 5.1 Annual Flows*

| Decade | Mean Annual Flow Volume (GL) |
|---|---|
| 1945–1954 | 1.02 |
| 1955–1964 | 1.13 |
| 1965–1974 | 1.22 |
| 1975–1984 | 1.41 |
| 1985–1994 | 1.87 |
| 1995–2005 | 1.68 |

*(after ACWS Technical Report 18, 2006)*

### 5.2.2 Stormwater Runoff Quality

Median concentrations of water quality determinants in stormwater from the Coastal Catchments measured during 2004 are given in Table 5.2.

*Table 5.2 Median Concentrations of Water Quality Determinants*

| Flow (ML/day) | EC (units) | SS (mg/L) | TN (mg/L) | TKN (mg/L) | NOx (mg/L) | TP (mg/L) |
|---|---|---|---|---|---|---|
| 2.1 | 117 | 30.7 | 1.02 | 0.97 | 0.13 | 0.19 |

*(after ACWS Technical Report 18, 2006)*

Wilkinson was also able to compare data for the coastal catchments from the Gulf St Vincent Water Pollution Studies carried out between 1973 and 1978 with data collected for the ACWS in 2004. This comparison showed that there are several differences between the data collected during the 1970s and that collected in 2004. Specifically:

- a fourfold reduction in the suspended solids concentrations
- reductions in nitrogen concentrations
- a reduction in pH
- no change in phosphorus concentrations.

In relation to copper, lead and zinc, Wilkinson assessed data for the River Torrens, Sturt River and Brown Hill Creek which are likely to be representative of water from the coastal catchments. Median copper and zinc concentrations were up to ten times and lead concentrations up to four times the ANZECC/ARMCANZ trigger concentrations for the protection of 95% of marine species.

Wilkinson also reported that there has been a significant reduction in the levels of lead and zinc measured in the metropolitan watercourses. The analysis did not reveal any long term trend in copper levels. The reductions in the levels of lead in stormwater can be attributed to the cessation of the sale of leaded petrol.

Wilkinson estimated the historic (1975–1985) and current (2004) loads discharged to the Adelaide Coastal Waters from the coastal catchments. This data is provided in Table 5.3.

It must be remembered that the nature of stormwater flows is that they are episodic and whilst the use of means and medians allows for detailed analysis to be undertaken that reveals important information, it should be remembered that stormwater flow and quality are highly variable. Typically, as flow rates increase so the concentration of contaminates also increases.

This variability in flow rate and quality presents challenges for the design of quality improvement measures.

*Table 5.3 Historic (1975–1985) and Current (2004) Loads Discharged (tonnes per annum)*

| | 2004 | 1975–1985 |
|---|---|---|
| Flow (GL/annum) | 1.7 | 1.4 |
| Area of catchment (km$^2$) | 21.0 | 21.0 |
| Yield (mm/year) | 79.8 | 67.4 |
| Suspended Solids (T/annum) | 97 | 235 |
| Total N (T/annum) | 1.8 | 3.2 |
| TKN (T/annum) | 1.6 | 1.9 |
| NOx N (T/annum) | 0.24 | 1.29 |
| Total P (T/annum) | 0.29 | 0.50 |
| $NH_3$ N (T/annum) | 0.12 | |
| Total Cu (T/annum) | 0.03 | 0.17 |
| Total Pb (T/annum) | 0.10 | 0.38 |
| Total Zn (T/annum) | 0.13 | 0.88 |

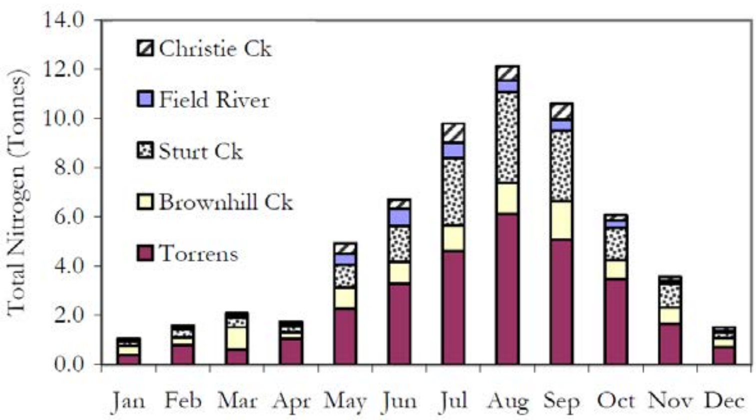
*(after ACWS Technical Report 18, 2006)*

### 5.2.3 Seasonality of Pollutant Load Inputs

Catchment specific data on the seasonal variability of pollutant load inputs due to stormwater runoff from the Study Area is not provided in the Adelaide Coastal Waters Study. However, information on the seasonal variation in loads of key pollutants (total nitrogen, total phosphorous and suspended solids) for the major outfalls to the Gulf near the Study area is provided.

This data is provided in Figure 5.1 below.

The data shows that for each of the key pollutants, the largest inputs tend to occur during the winter months, with smaller inputs, associated with lower average rainfall, during the summer months.



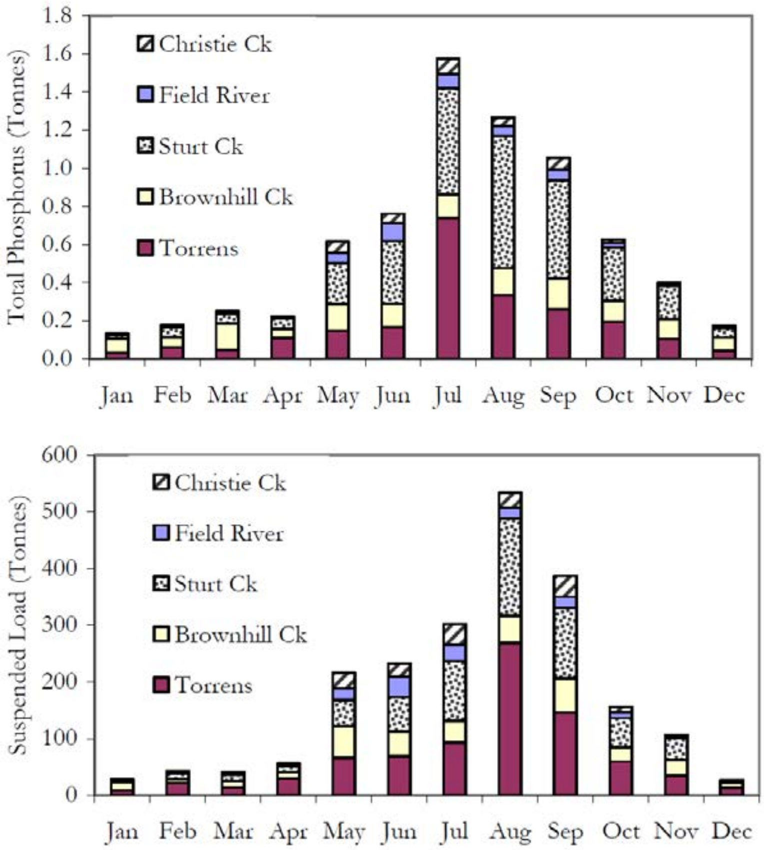



***Figure 5.1 Seasonal Variation In Pollutant Load Inputs (after ACWS Technical Report 18, 2006)***

### 5.2.4 Stormwater in the Context of all Discharges to the Gulf

It is useful to compare the relative contribution that stormwater makes to loads into the Adelaide Coastal Waters as this aids in understanding the key issues for stormwater. Again the work undertaken by Wilkinson for the Adelaide Coastal Waters sets this out and the contribution of loads from stormwater and treated wastewater are set out in Table 5.4.

*Table 5.4 Contribution of Loads from Stormwater and Treated Wastewater – All Adelaide Coast*

| Annual loads (GL or tonnes) | Stormwater | Treated wastewater | % Contribution from stormwater |
|---|---|---|---|
| Flow | 114.9 | 62 | 65 |
| Suspended solids | 6849 | 1579 | 81 |
| Total N* | 153.5 | 1204.2 | |
| TKN* | 117 | 766.8 | 13 |
| NOx N | 36.1 | 437.5 | 7.6 |
| Total P | 20.3 | 335 | 5.7 |
| NH3 N* | 5.3 | 504 | 1.0 |
| Total Cu | 1.31 | 2.96 | 31 |
| Total Pb | 1.48 | 0.21 | 88 |
| Total Zn | 10.19 | 3.69 | 73 |

*(after ACWS Technical Report 18, 2006)*

* *Note these figures do not include the contribution of Penrice to the Total Nitrogen load.*

## 5.3 Environment Protection Authority's Ambient Monitoring of Gulf St Vincent

### 5.3.1 Background

The Ambient Water Quality Monitoring of the Gulf St Vincent Metropolitan Bathing Waters Report, conducted in 1997 (reporting on data collected from 1995 to 1996) and 2005 (reporting on data collected from 1995 to 2002), (SA Environment Protection Authority, January 2004) analysed monthly (and in summer fortnightly) samples collected off each of the metropolitan jetties for various pollutants and compared these characteristics to Port Hughes which has been taken as a reference site, largely unaffected by urban development and other discharges.

The characteristics monitored in the program were:

- nutrients
- chlorophyll *a* which indicates the presence of algae
- heavy metals
- indicators of faecal contamination
- water clarity (turbidity).

The characteristics measured are based on the water quality requirements to support the designated environmental values in the Australian and New Zealand Guidelines for Fresh and Marine Waters (ANZECC 1992 and 2000) and the National Health and Medical Research Council (NHMRC) Australian Guidelines for the Recreational Use of Water (NHMRC 1990 and 1996) and the Environment Protection (Water Quality) Policy (EPA 2003).

The reports set criteria for each characteristic such that water quality can be described broadly as good, moderate or poor. This was determined by developing criteria based on the percentage of time that the water quality conditions exceed the ANZECC Australian Water Quality Guidelines for Fresh and Marine Waters and the NHMRC Guidelines for Recreational use of Water. The ANZECC Guidelines state that a toxicant water quality trigger value in a slightly or moderately disturbed system (Gulf of St Vincent Classification) is set at a level that will protect 95% of species.

The objective of the ANZECC Guidelines is to use the trigger values as concentrations that, if exceeded, would indicate a potential environmental problem and therefore prompt further investigation. The water quality parameter classifications of good, moderate or poor are generated by the positions of the 90th and 50th percentiles in relation to the trigger values. These percentiles are used to protect organisms from chronic effects of the toxicants. If the 90th percentile is less than the trigger value, the parameter is classified as good. If the 90th percentile is greater than the trigger value but the 50th percentile (median) is less than the trigger value, the parameter is classified as moderate. If the 50th percentile is greater than the trigger value, the water quality is classified as poor - this is a level at which chronic toxicities may be exhibited in some sensitive organisms.

Each of the monitored characteristics is discussed below.

### 5.3.2 Turbidity

In stormwater, high turbidity can originate from a number of sources, such as soil erosion, decaying organic matter and other pollutants. Turbidity can also result from elevated nutrient levels resulting in an increased amount of algae in the water column.

Turbidity can impact the marine environment as a consequence of reduced light penetration and an increase in the amount of suspended particles.

Turbidity has been assessed by the EPA against trigger values for the ecosystem as well as recreation and is classified by the EPA as being moderate along the coast within the study area.

### 5.3.3 Metals

Metals entering the metropolitan coastal waters can be linked to two major sources, urban stormwater and industrial sites. Urban stormwater can contain metals deposited on impervious surfaces such as roads and metals can come from vehicle wear products (brake linings and tyres etc.). Metals can also be naturally occurring and enter the environment through the weathering of mineral deposits.

Metals can affect organisms in a number of ways. They can be acutely toxic, causing mortality, or chronically toxic, resulting in mortalities or other sub-lethal effects.

The metals that were sampled were; aluminium, chromium, copper, lead, nickel and zinc.

Metals have been assessed against trigger values for the protection of ecosystem and aquaculture. Overall the EPA has classified chromium, copper and lead as good when assessed against both guidelines. Nickel at Glenelg and Brighton has been classified as moderate for the protection of ecosystems but good for protection of aquaculture and zinc has been classified as poor at all sites when assessed against the guidelines for ecosystems and for aquaculture.

In relation to nickel, the EPA notes that the results for Brighton and Glenelg are not significantly different to results from Port Hughes, albeit sufficient to change the classification for Brighton and Glenelg to moderate (90$^{th}$ percentile value at Port Hughes is 0.007 mg/L and the values for Brighton and Glenelg are 0.008 mg/L compared to a trigger value of 0.007 mg/L). It was suggested that the small difference in measured concentration may be due to the impacts of stormwater or wastewater discharges, but could also be due to sampling error.

Similarly, the results from the Adelaide coast for zinc are not significantly different from the results for Port Hughes. In fact Port Hughes, the site with the least amount of urban development, had the highest concentrations of both soluble and total zinc. However, the EPA note that the high concentrations of zinc in the Gulf may not be an indicator of human pollution but are more likely to be due to natural weathering of rocks and soils.

Three studies investigating the potential metal contamination of fish, molluscs and other invertebrates have shown that marine organisms are not generally at risk of bio-accumulating the metals covered in the report from pollution in the Gulf St Vincent (EPA 2000, Maher 1986, Olsen 1983).

### 5.3.4 Nutrients

Nutrients are required by aquatic organisms for growth and reproduction however land based discharges such as sewage effluent and urban stormwater runoff have enriched gulf waters beyond what was naturally the case. With the exception of ammonia, nutrients are not of themselves toxic to marine organisms, but this enrichment causes an increase in the growth of algae which can be toxic to some marine organisms. Growth of algae also reduces the clarity of water and can cause smothering of seagrass.

The nutrients measured were; total nitrogen, ammonia, oxidised nitrogen and phosphorus. The EPA classified nutrients against the two environmental values: the protection of the aquatic ecosystem and the protection of aquaculture. These were all classified as good except for ammonia which was generally only moderate and oxidised nitrogen which was moderate for three of the seven metropolitan beaches measured.

### 5.3.5 Biological Parameters

The biological parameters measured in the study were chlorophyll *a* (indicative of algae) and faecal coliforms. Biological indicators of water pollution are often considered to be the best indicators of impacts on aquatic environments.

Chlorophyll *a* is the major photosynthetic pigment in algae and therefore provides a good indicator of the amount of algae in the water. The EPA classified chlorophyll *a* concentrations as poor for the areas covered in the study when compared with the ecosystem guidelines. Poor water quality classifications along the metropolitan coast are consistent with the responses of other biological systems, such as loss of seagrass and degradation of subtidal reefs.

The poor rating for chlorophyll *a* combined with the other biological responses indicate a system under stress from nutrient enrichment.

Faecal coliforms*, E.coli* and enterococci were measured and an assessment made against the NHMRC's human health guidelines for direct contact (i.e. bathing and swimming). For microorganisms, the water quality classifications were good for all sites however the EPA noted that occasional high levels indicate that pollution is probably a short term response to rainfall.

## 5.4 The Adelaide Coastal Waters Study

### 5.4.1 Background

The ACWS was commissioned by the South Australian Government in 2001 in response to concerns about the decline in water quality and the loss of over 5000 ha of seagrass from the near shore area of Adelaide's beaches. This was a large-scale study that included the entirety of the Adelaide metropolitan area.

### 5.4.2 Principal Conclusions of the Adelaide Coastal Waters Study

The ACWS had three key focus points being; water quality, seagrasses and sediments. Like many other coastal environments near cities, the Adelaide coast has undergone significant modification and degradation as a result of many years of near continuous inputs of nutrient rich, turbid and coloured water and wastewater.

Some of the key conclusions in the ACWS final report (CSIRO, November 2007) in so far as they impact on stormwater management were as follows:

- It is unlikely that freshwater is implicated in the initial loss of seagrass off Adelaide's coastline, although it may play a role in determining the capacity for natural regeneration/recovery at sites close to land-based inputs (rivers, creeks, drains, and WWTPs). Both seagrass species present off the Adelaide coast (*Amphibolis antarctica* and *Posidonia sinuosa*) are highly tolerant to short-term (72 hours) reductions in salinity, with major salinity reductions required for prolonged periods (weeks) to kill adult plants. The reductions in salinity experienced along Adelaide's coast (particularly in the near-shore region where stormwater enters Adelaide's coastal waters and at locations adjacent to wastewater outfalls) are minor. However, short-term reductions in salinity can affect *A. antarctica* seedlings and *P. sinuosa* fruits. Thus, it is possible that reductions in salinity caused by stormwater and wastewater could influence recruitment processes on a very localised scale.
- It is unlikely that toxicants (pesticides, organochlorines, hydrocarbons, herbicides,etc.) have been responsible for broad-scale historical seagrass losses. This is supported by the fact that toxicants have only been sporadically detected in very low concentrations in freshwater entering Adelaide's coastal water (which are then rapidly diluted) and that the concentrations required to affect seagrasses are relatively high. Sampling of the coastal waters following peak stormwater flows (when detection would be most likely) failed to detect any toxicants. Similarly, toxicant levels in sediments adjacent to stormwater outlets, as well as sites further off shore, found very low or undetectable toxicant levels. Finally, the historical levels detected in stormwater could never have reached levels capable of having an impact.
- It is possible that increased turbidity from stormwater contributed to the broad-scale loss of near-shore seagrass. Modelling results indicate that coastal inputs can become 'trapped' in the near-shore zone, resulting in a greatly diminished benthic light climate and that at times, light levels at 3 m depth can be so low as to cause the death of *Amphibolis* (but not at deeper depths). It is highly likely that near-shore light conditions were worse during the 1940s to 1960s (when much of the near-shore seagrass loss occurred) because discharges

from the Torrens River then were significantly greater than at present. Experimental results from the ACWS have not been able to conclusively establish that a compromised light climate alone could have caused the loss of seagrass, although this remains a possibility.

- It is most likely that nutrients from stormwater and wastewater were responsible for broad-scale historical seagrass losses. There are multiple lines of evidence in support of this hypothesis, including laboratory and field-based experimental results. The coastal waters have received almost continuous inputs of stormwater and wastewater for the last 70 years as well as industrial contaminants and sewage sludge between the early 1960s and 1993. This served to increase the levels of water-column nutrients in Holdfast Bay and in the vicinity of wastewater outfalls. Perhaps the most significant initial activities (in terms of environmental impact) included the diversion of the Torrens River, waste disposal from the Glenelg WWTP, and the commencement of Penrice Soda discharges in 1940. The ACWS has provided clear evidence that the existing offshore seagrasses from Port Gawler to Port Noarlunga continue to receive nitrogen sourced from WWTP and industrial outfalls (most notably Penrice Soda). Clearly, near-shore seagrasses (prior to their loss) would also have been exposed to those same nutrient sources.
- Results of the ACWS unambiguously prove that chronic, yet minor, increases in water column nutrients (as might be associated with WWTP and industrial inputs) could have caused the slow decline of *Amphibolis* and *Posidonia* in shallow, previously nutrient poor, coastal waters. Mesocosm experiments have demonstrated that *Amphibolis* is more susceptible to nutrient enrichment than *Posidonia* — particularly in the presence of epiphytes and irrespective of depth. The existence of this effect on *Amphibolis* in the absence of epiphytes has not been unequivocally established.
- The findings of the ACWS are consistent with observations in similar, nitrogen-limited marine environments which suggest that nitrogen, rather than phosphorus, plays a key role in the degradation of marine (and seagrass) systems. Inorganic forms of nitrogen (such as ammoniacal nitrogen and oxidised nitrogen) are generally of greater concern since they are biologically available. Further work would need to be undertaken to understand the effects of reducing nitrogen in Adelaide's coastal waters while leaving phosphorus loads unchanged.

### 5.4.3 Principal Recommendations of the Adelaide Coastal Waters Study

The ACWS made a number of recommendations for key management actions for the protection and eventual enhancement of the Adelaide Coastal Waters. The ACWS report notes that recommendations were made based on the knowledge gained during the study but did not integrate social, political, economic and environmental values which were beyond its scope.

The recommendations relevant to stormwater management were:

- As a matter of priority, steps must be taken to reduce the volumes of wastewater, stormwater, and industrial inputs into Adelaide's coastal environment. This should be done within the context of an overarching strategy designed to remediate and protect the metropolitan coastal ecosystem.
- The total load of nitrogen discharged to the marine environment should be reduced to around 600 tonnes (representing a 75% reduction from the 2003 value of 2400 tonnes).
- Commensurate with efforts to reduce the nitrogen load, steps should be taken to progressively reduce the load of particulate matter discharged to the marine environment. A 50% load reduction (from 2003 levels) would be sufficient to maintain adequate light levels above seagrass beds for most of the time. The reduced sediment load will also contribute to improved water quality and aesthetics.
- To assist in the improvement of the optical qualities of Adelaide's coastal waters, steps should be taken to reduce the amount of CDOM (coloured dissolved organic matter) in waters discharged by rivers, creeks, and stormwater drains.

- While the available data suggests that toxicant levels in Adelaide's coastal waters pose no significant environmental risk, loads from point sources such as the Port River, WWTPs, and drains should continue to be reduced. Routine monitoring of toxicant loads and concentrations should be undertaken every 3–5 years.

## 5.5 Key Stormwater Quality Issues for the Catchment

Based on the water quality data, the ambient water quality monitoring program and the results of the ACWS, the key issues for the stormwater management plan are considered to be:

### Volumes of Water

- The hydrological regime of the Adelaide Plains has been significantly changed.
- Increasing urbanisation has increased the annual discharge of stormwater to the marine environment and has also changed the seasonality of flows with a greater increase occurring for summer flows.
- The ACWS recommended that steps should be taken to reduce the volume of wastewater and stormwater discharged into the marine environment.

### Suspended Solids

- Stormwater contributes approximately 80% of all the suspended solids delivered to the Adelaide Coast.
- Stormwater discharges are episodic in nature. Much of the loads transported by stormwater occur during a relatively very short period of time during major flow events.
- It is possible that increased turbidity in stormwater contributed to a broad scale loss of near shore seagrass.
- The ACWS recommends that the load of particulate matter discharged to the marine environment should be reduced by 50% from 2003 levels.
- Turbidity in the Adelaide Coastal Waters is an issue and is classified as only "moderate" by the EPA.
- Algae make a contribution to the turbidity of the coastal waters.

### Heavy Metals

- Stormwater is the principal contributor of lead and zinc to the Adelaide Coast.
- In the major urban watercourses significant reductions in lead and zinc in stormwater have been measured from the late 1990s the early 2000s. Reductions in lead can be attributed to the cessation of the use of leaded fuel for vehicles.
- Concentrations of copper, lead and zinc in stormwater exceed by many times the trigger levels for the protection of marine species. However, due to dilution, the concentrations of these contaminants in the marine waters is significantly lower and do not appear to currently pose a significant environmental risk.

### Nutrients

- Nutrient enrichment of the Adelaide Coast, particularly ammonia and oxidised nitrogen, is a serious issue that has contributed to the loss of seagrasses and to the high levels of algae in the coastal waters.
- The EPA classifies the level of chlorophyll *a* as poor and notwithstanding the classifications for nutrients the levels of chlorophyll *a* are indicative of a system under stress from nutrient enrichment.
- The chronic, yet minor increases in water column nutrients were most likely the primary cause of the historical loss of seagrasses. The marine environment is limited by nitrogen

rather than phosphorus and particularly the inorganic forms of nitrogen (ammonia and oxidised nitrogen) are the nutrient sources of greatest concern.

- The ACWS recommended that the total load of nitrogen discharged to the metropolitan coastal waters should be reduced from 2400 tonnes (2003) to 600 tonnes. While discharges from the WWTPs provide the most significant portion of this load, a proportionate reduction from stormwater discharges will be required to meet the target.

### Recreation

- Bacteriological indicators in the gulf waters are good and meet the requirements for direct human contact. However, there are occasions when levels are high due to the impact of animal faecal contamination during periods of heavy rainfall. Signs along the metropolitan beaches suggest that swimmers and bathers should avoid contact with discoloured water.

### Toxicants, Chemicals and Pesticides

- Toxicants (pesticides, organochlorins, hydrocarbons, herbicides etc.) are unlikely to have had any detrimental impacts on the coast and have only ever been sporadically detected at very low concentrations.
- Notwithstanding that toxicants pose no significant environmental risk, the ACWS recommends loads for point sources should continue to be reduced.
- The residual herbicide simazine has been detected in urban stormwater on a number of occasions. T he source of this in the urban areas is likely to be from easily available path weeders. The presence of this chemical renders stormwater unsuitable for use in aquifer storage schemes and other reuse schemes.

### Litter and Debris

- To assist with improving the optical qualities of the coastal waters, the amount of coloured dissolved organic matter in stormwater should be reduced.
- Leaf matter contributes to stormwater colouration and gum tree throughfall is also coloured.
- Litter and other anthropological debris detracts from aesthetic values and poses a potential risk to humans and marine animals.

# 6 Stormwater Management Objectives and Strategies

## 6.1 Introduction

The Stormwater Management Planning Guidelines published by the Stormwater Management Authority includes the following in relation to stormwater management objectives:

*Catchment specific objectives for the management of stormwater within the area are to be set and are to be based on the problems and opportunities identified. The objectives should provide measurable goals for the management of stormwater in the catchment.*

As a minimum, the objectives are to set goals for:

- an acceptable level of protection of the community and both private and public assets from flooding;
- management of the quality of runoff and effect on the receiving waters, both terrestrial and marine where relevant;
- extent of beneficial use of stormwater runoff;
- desirable end-state values for watercourses and riparian ecosystems;
- desirable planning outcomes associated with new development, open space, recreation and amenity;
- sustainable management of stormwater infrastructure, including maintenance.

## 6.2 Stormwater Management Guidelines

### 6.2.1 Introduction

To ensure that this stormwater management plan aligns with other strategies and guidelines, stormwater targets from other documents have been reviewed. These include the recommendations made by the ACWS, Australian Runoff Quality: A Guide to Water Sensitive Urban Design (Engineers Australia, 2006) and targets from the Water Sensitive Urban Design Consultation Statement (Department for Water, 2012). These are summarised below.

### 6.2.2 Adelaide Coastal Waters Study

Recommendations given in the ACWS include the following in relation to stormwater management:

- As a matter of priority, steps must be taken to reduce the volumes of wastewater, stormwater, and industrial inputs into Adelaide's coastal environment.
- The total load of nitrogen discharged to the marine environment should be reduced to around 600 tonnes (representing a 75% reduction from the 2003 value of 2400 tonnes).
- Commensurate with efforts to reduce the nitrogen load, steps should be taken to progressively reduce the load of particulate matter discharged to the marine environment. A 50% load reduction (from 2003 levels) would be sufficient to maintain adequate light levels above seagrass beds for most of the time. The reduced sediment load will also contribute to improved water quality and aesthetics.

### 6.2.3 Australian Runoff Quality

Guidelines on the reduction of pollutant loads are set out for Victoria and New South Wales in the Australian Runoff Quality Guidelines. Stormwater treatment objectives are as follows:

- Total suspended solids – 80% reduction of the average annual load
- Total phosphorus – 45% reduction of the average annual load
- Total nitrogen – 45% reduction of the average annual load
- Litter – Retention of litter greater than 50mm for flows up to the 3 month ARI peak flow
- Coarse sediment – Retention of sediment coarser than 0.125 mm for flows up to the 3 month ARI peak flow
- Oil and grease – No visible oils for flows up to the 3 month ARI peak flow.

### 6.2.4 Water Sensitive Urban Design (Consultation Statement)

The Department for Water has developed a Water Sensitive Urban Design (WSUD) Consultation Statement on policy directions the State Government could consider to enhance the uptake of WSUD initiatives. This document does not represent government policy but is a useful guide as to the possible direction of WSUD. The statement contains the principles and targets under four headings; water conservation, runoff management-quantity, runoff management-quality and integrated design. These principles and targets are reproduced below.

*DESIGN PRINCIPLE: WATER CONSERVATION*

*Water systems should be as efficient as possible and, where feasible, mains supplied water should not be the primary or sole source for purposes that do not require potable standards (i.e. many uses outside of the home or buildings, such as irrigation of reserves and gardens, and car washing).*

*Principle Intent*

*Better management of water resources and water infrastructure by:*

- *Promoting water systems that support efficient water use.*
- *Promoting alternatives to mains water for non-potable uses, where feasible (for example roof runoff, treated stormwater and treated wastewater).*
- *Enhancing the capacity of water infrastructure (for example stormwater and sewerage systems by reducing stormwater flow and sewage).*

*Performance Target*

- *Indoor mains water consumption: does not exceed 100 litres per person per day (see Target Focus, below).*
- *Outdoor irrigated open spaces: evidence demonstrating how best practice irrigation management is to be promoted in irrigated open spaces.*

*In recognition of the significant differences in water requirements of industry and commercial enterprises, no specific indoor water use target is proposed for such enterprises at this time.*

*Target Focus*

- *Indoor water use target: New Class 1 residential buildings.*
- *Outdoor irrigated open spaces: New developments where outdoor irrigated open space is to be provided.*

*DESIGN PRINCIPLE: RUNOFF MANAGEMENT - QUANTITY*

*Post-development hydrology should, as far as practical and appropriate, seek to reduce the hydrological impacts of built environments on watercourses and their ecosystems (including in-stream erosion impacts), and promote, where appropriate, opportunities to restore watercourses.*

*This principle complements existing local flood management requirements for developments.*

*Principle Intent*

- *Help protect waterways and, where relevant, promote their restoration by seeking to limit flow from development to near natural levels.*

*Performance Target*

- *For catchments with a total impervious area of up to 20 per cent: capture of 5 millimetres of rainfall from connected impervious area.*
- *For catchments with a total impervious area of greater than 20 per cent: capture of 10 millimetres of rainfall from connected impervious area.*

*The captured runoff must be capable of being drawn down within a period of 24 hours (e.g. discharged and/or infiltrated or reused as appropriate).*

*Target Focus*

*Residential: developments that will provide for public or shared management of the infrastructure that would be needed to achieve the target.*

*Other: all other circumstances involving significant paved surface areas, for example:*

- *streetscapes (including roads and footpaths); and*
- *commercial, industrial, institutional and community service buildings and car parks, and streetscapes.*

***DESIGN PRINCIPLE: RUNOFF MANAGEMENT - QUALITY***

*Positively manage the quality of urban runoff through implementing water-sensitive urban design.*

***Principle Intent***

*To help protect and, where required, enhance, the quality of runoff entering receiving water environments, in order to support environmental and other water management objectives.*

***Performance Target***

*Reduce the average annual loads of:*

- *total suspended solids by 80 per cent;*
- *total phosphorus by 60 per cent;*
- *total nitrogen by 45 per cent;*
- *litter/gross pollutants by 90 per cent;*

*This would be demonstrated based on modelling procedures which compare proposed catchment design with an equivalent untreated catchment.*

***Target Focus***

*Residential: development, including residential subdivisions that require the carrying out of road, stormwater or other communal infrastructure works.*

*Other: commercial, industrial, institutional, community service, recreational development.*

***DESIGN PRINCIPLE: INTEGRATED DESIGN***

***Principle***

*That the planning, design, and management of WSUD measures seeks to support other relevant state, regional and local objectives.*

***Principle Intent***

*To provide that WSUD is implemented in a way that maximises the potential to achieve multiple outcomes, including for example promoting public amenity, habitat protection and improvement, reduced energy use and greenhouse emissions, and other outcomes that contribute to the wellbeing of South Australians.*

***Performance Target***

*Relevant stakeholders to be engaged at relevant stages of planning, designing, constructing, and managing WSUD measures so as to maximise the potential for WSUD to support and sustain multiple outcomes.*

***Target Focus***

*Subdivision scale.*

## 6.3 Strategies and Objectives

The Cities of Holdfast Bay and Marion have adopted the following objectives and strategies for stormwater management in the catchment.

### 6.3.1 Overarching Objective

The Cities of Holdfast Bay and Marion have an overarching objective of progressing towards becoming “Water Sensitive Cities” and to minimise flooding and harness the potential of stormwater to overcome water shortages, reduce urban temperatures and improve waterway health and the landscape of their cities. Water Sensitive Urban Design is the process that will lead to Water Sensitive Cities.

### 6.3.2 Acceptable Level of Protection of the Community for Both Private and Public Assets from Flooding

Objectives have been set with reference to the minor drainage system and major drainage systems.

The minor system is the pits and underground pipes throughout the study area whose primary function is to avoid nuisance flooding and ponding so as to maintain the serviceability and safety of the road network.

The major drainage system includes the minor system but also includes the roads, open spaces, water courses and other overland flow routes which become engaged during a storm that exceeds the capacity of the minor system. The primary purpose of the major system is to prevent flooding that causes property damage or that threatens the safety of people in the floodplain.

In terms of the standard of protection that should be offered by the major system, it is relatively well established that this should be at least a 1 in 100 year ARI standard. There are numerous references in the existing Development Plans of both Councils to reinforce the need for new development to be protected from the 1 in 100 year ARI flood. Ideally the road network, in combination with the minor drainage system, should be capable of conveying the 100 year ARI flows without causing inundation of property.

The floodplain mapping developed for the study area shows a number of areas where the 100 year ARI flood breaks out of the road network and flows into properties. Mostly this floodwater is shallow, that is less than about 100 or 200 mm deep. Water of this depth will not necessarily cause flooding above the floors of buildings. Certainly if these allotments were to be redeveloped, this depth of water could be managed by setting floors at an appropriate level.

It is not considered cost effective to reconstruct the road and drainage network to prevent property flooding during the 1 in 100 year ARI event, although during reconstruction of roads and replacement or augmentation of the drainage network, the objective of 1 in 100 year ARI protection ought to be considered. There are however some isolated areas in the floodplain where water depths are deeper. It is considered appropriate that works should be considered due where floodwaters are predicted to exceed 300 mm in depth.

Continuing development within the catchment, which is significantly increasing the areas of impervious surfaces, has the potential to significantly reduce the standard of flood protection across the catchment in the absence of any controls to retain or detain rainfall runoff. To a lesser extent, an increase in rainfall intensity and sea level rise due to a changing climate will also reduce the standard of flood protection.

In terms of the capacity of the minor system, design standards can vary. The City of Holdfast Bay's Development Plan specifies 1 in 2 year or 1 in 10 year ARI standard for the minor system in land divisions depending on the density of development. The Council's asset management plan notes an objective to meet a 1 in 5 year ARI standard. The original South Western Suburbs Drainage Scheme, which drains the majority of the study area, was designed to achieve a 1 in 5 year ARI standard. It is considered that a 1 in 5 year ARI standard is an appropriate target standard for the minor system noting that a lesser standard may well be acceptable when balanced against the cost to replace assets that still have a significant life ahead of them.

Assessment of the drainage network shows that much of the underground pipe network has a capacity less than the 1 in 5 year ARI standard. However, the floodplain mapping developed for the 1 in 5 year ARI flood event demonstrates that, by and large the 1 in 5 year ARI event stays in the road reserves and so storage in the road network is mitigating flow rates in the pipe network. The floodplain mapping for the future scenario 1 in 5 year ARI flood, which assumes no controls over development, shows a noticeable increase in the amount of water in the road network during a 1 in 5 year ARI storm event.

Based on the above, the following objectives are proposed:

#### Objective 1.1

All new development must be built at a level, with an appropriate freeboard allowance, that ensures that buildings are not subject to inundation during a 100 year ARI flood.

#### Objective 1.2

All new development must provide for overland flow paths as required to ensure the free passage of floodwaters during a 100 year ARI event.

#### Objective 1.3

Wherever it is technically possible and financially viable, the 100 year ARI flood should be contained within road reserves and public open spaces.

#### Objective 1.4

The major drainage system must have sufficient capacity to ensure that during a 100 year ARI flood no property is subject to flood hazard categorised as medium, high or extreme.

#### Objective 1.5

The 5 year ARI flood must be contained within road reserves and public open spaces.

### Objective 1.6

Wherever it is technically possible and financially viable, the minor drainage system should, in the case of local roads, be capable of restricting gutter flow widths to a maximum of 2.5 m during a 5 year ARI flood and, in the case of arterial, sub-arterial and collector roads, some lesser gutter flow width in accordance with the road authority's design standards.

### Objective 1.7

New development must be constructed so as to not cause an increase in 5 year ARI flow rates.

### Objective 1.8

New stormwater infrastructure must not increase the risk of flooding in downstream areas.

To meet these objectives the following strategies are proposed:

#### *Strategy 1.1*

Progressively implement stormwater drainage upgrades to provide sufficient drainage capacity to meet Objectives 1.4, 1.5 and 1.6.

#### *Strategy 1.2*

Where necessary, develop planning policy and design techniques to ensure that Objectives 1.1, 1.2, and 1.7 are met.

#### *Strategy 1.3*

Implement joint audit programs and policies to ensure that planning policy and design techniques are built and maintained to ensure they continue to function as required.

Further detail regarding the above strategies is provided in Section 7.

### 6.3.3 Management of Runoff Quality and its Effect on Receiving Waters both Terrestrial and Marine Where Relevant

The receiving waters for stormwater runoff from the study area are the Adelaide Coast. The principal issues of concern for the Adelaide Coastal Waters have been identified by the EPA in its Adelaide Coastal Water Quality Improvement Plan (ACWQIP) as the nutrient nitrogen, suspended solids and CDOM. Targets for stormwater include a reduction in nitrogen of 67%, a reduction in suspended solids of 50%, and a decrease in CDOM. The ACWQIP proposes that this issue be dealt with through the reuse of stormwater noting the Natural Resource Management (NRM) Board's target of 75% reuse, and the widespread adoption of WSUD.

During discussions, the EPA has also suggested that coastal waters would benefit from a reduction in the number of runoff events. This could be achieved by providing retention devices throughout the catchments to capture the smaller rainfall events.

Secondary, but still important, issues associated with stormwater pollution include:

- pathogens that impact on recreational water quality
- litter and debris that detracts from aesthetic qualities and contributes to CDOM
- toxicants including pesticides, that whilst not implicated in causing any environmental harm to the marine environment, ought not to find their way into stormwater.

Existing plans and policies provide plenty of support for programs that lead to water quality improvements, particularly WSUD, street sweeping, enforcement of codes of practice and GPT construction and maintenance. The Development Plans of both Councils also include objectives and principles to require that development does not contribute to pollution, and these plans will be strengthened by conversion to the Better Development Plan policies.

Based on the above, the following objectives are proposed:

### Objective 2.1

Stormwater discharged to the marine environment should meet targets that are set by the Adelaide Coastal Waters Study and other relevant state and regional plans within Council's control and responsibility.

To meet this objective, the following strategies are proposed:

#### *Strategy 2.1*

To the extent that it is technically possible and financially viable, the road and drainage network should be progressively retrofitted with WSUD devices that strive to capture and treat road runoff to meet the water quality improvement requirements for stormwater required by the Adelaide Coastal Waters Study and other relevant state government policy.

#### *Strategy 2.2*

Redevelopment of open spaces and other community facilities should incorporate WSUD devices that strive to meet the water quality improvement requirements for stormwater required by the Adelaide Coastal Waters Study and other relevant state government policy.

#### *Strategy 2.3*

Develop a statutory planning policy that requires all new development to incorporate WSUD techniques that assist in meeting pollutant reduction targets.

#### *Strategy 2.4*

Wherever technically feasible and without compromising flood protection objectives, all stormwater outlets discharging to the Adelaide beaches should be fitted with gross pollutant traps.

#### *Strategy 2.5*

The quantity of pollutants entering the drainage network should be minimised by maintaining an effective program for:

- cleaning and maintenance of gross pollutant traps
- street sweeping
- installation and regular emptying of rubbish bins, particularly in commercial precincts
- management of unpaved road verges to minimise sediment transport.

#### *Strategy 2.6*

The quantity of pollutants entering the drainage network should be minimised by maintaining enforcement programs to ensure compliance with:

- anti-littering laws
- codes of practice for building sites and the construction industry
- responsible chemical use.

#### *Strategy 2.7*

Soil erosion from hillsides and gullies should be managed by adopting appropriate land management practices and revegetation and stabilisation of gullies.

*Strategy 2.8*

Cooperate with other agencies to develop and conduct stormwater quality monitoring and reporting programs.

Further detail regarding the above strategies is provided in Section 8.

### 6.3.4 Extent of Beneficial Use of Stormwater Runoff

The NRM Board's target for reuse of stormwater is 75%. This is an ambitious target that will be difficult to achieve in the study area because of the lack of open space to capture, treat and store stormwater. Notwithstanding this, opportunities exist for capture and beneficial reuse of runoff. These options include:

- rainwater tanks
- rain gardens and other WSUD techniques where they supplement landscape plantings, for example street trees
- capture and reuse for irrigation of large open spaces.

It should be noted that there are synergies between objectives for reuse and water quality, for example Strategy 2.1 which is to implement WSUD devices for water quality improvements for road runoff will also provide water for street tree and streetscape improvement.

Similarly, there are synergies with Objective 1.7 to enable water reuse through retention and detention.

**Objective 3.1**

Maximise the reuse of stormwater for beneficial purposes.

**Objective 3.1**

To the extent that it is technically possible and financially viable, the road and drainage network should be progressively retrofitted with WSUD devices that strive to capture road runoff to replenish soil moisture for maintenance of street trees and plantings. (Note also Strategy 2.1).

**Objective 3.2**

Encourage on-site use of stormwater by rainwater tanks, detention and retention systems. (Note also Objective 1.7).

**Objective 3.3**

Where feasible, implement stormwater reuse for watering of community or private open spaces.

**Objective 3.4**

Sufficient water shall be allocated for environmental flows to maintain water dependent ecosystems.

Further information regarding specific actions to be taken in relation to stormwater reuse is provided in Section 9.

### 6.3.5 Desirable End-State Values for Watercourses and Riparian Ecosystems

**Objective 4.1**

Watercourses, gullies, drains, pipes and other elements of the stormwater drainage network in urban areas should be in public ownership.

### Objective 4.2

Open watercourses and gullies should be preserved in as natural condition as possible, be revegetated and managed to maximise their ecological values and functions, and should minimise any potential for stream erosion.

Further information regarding specific actions to be taken in relation to stormwater reuse is provided in Section 10.

## 6.3.6 Desirable Planning Outcomes Associated with New Development, Open Space, Recreation and Amenity

Good planning and changes to the manner in which new development is undertaken have the potential to have a significant beneficial effect impact on the way in which stormwater is managed in the catchment.

The following objectives are proposed.

### Objective 5.1

New development should be planned to minimise its impact on downstream stormwater systems and to capitalise on opportunities for provision of open space incorporating WSUD and other stormwater management systems.

To achieve this objective, the following strategies are proposed:

#### *Strategy 5.1*

The Cities of Marion and Holdfast Bay will work co-operatively to promote a development form in which higher density precincts are established with stormwater management infrastructure in planned open space.

#### *Strategy 5.2*

The Cities of Marion and Holdfast Bay will work co-operatively on changes to their respective Development Plans focussing on aligning stormwater provisions and reinforcing the objectives and strategies of this Stormwater Management Plan.

This objective and its associated strategies has linkages to Strategy 1.2 for minimisation of flooding impacts associated with new development and for ensuring that new development is protected from flooding, Strategy 2.3 requiring new development to incorporate WSUD techniques to improve water quality and Objective 3.2 encouraging on-site retention and use of stormwater.

### Objective 5.2

Open space should be used to achieve multiple outcomes including maximisation of permeable surfaces, on-site retention, infiltration and stormwater reuse wherever possible.

This has linkages to Objective 2.2 for stormwater quality improvement, Objective 3.3 for stormwater reuse and Objective 4.2 for environmental enhancement above.

Further detail regarding the above strategies is provided in Section 11.

## 6.3.7 Sustainable Management of Stormwater Infrastructure, Including Maintenance

### Objective 6.1

Stormwater infrastructure is to be managed sustainably.

### Objective 6.2

Stormwater management must be cognisant of the impacts of climate change and sea level rise.

### Objective 6.3

All stormwater infrastructure including WSUD schemes are to be maintained in accordance with cost-effective maintenance management plans.

#### *Strategy 6.1*

Development of a strategy that sets out the timeframe for achieving the objectives set out in the Stormwater Management Plan, and which integrates all elements of Council business to ensure that WSUD initiatives are embedded in all Council activities.

#### *Strategy 6.2*

Development of an Asset Management Plan for stormwater that ensures that the objectives of the Stormwater Management Plan are achieved.

#### *Strategy 6.3*

The Cities of Holdfast Bay and Marion will work cooperatively on the development and implementation of stormwater management objectives to ensure a “whole of catchment” approach is achieved.

Further detail regarding the above strategies is provided in Section 12.

# 7 Description of Strategies – Runoff Quantity

The strategies proposed to address runoff quantity issues in the catchment are described in the following section.

## 7.1 Strategy 1.1

**Progressively implement stormwater drainage upgrades to provide sufficient drainage capacity to meet Objectives 1.4, 1.5 and 1.6 (refer Section 6.3.2)**

### 7.1.1 Upgrades to the Major Drainage System

The 1 in 100 year floodplain maps contained in Appendices A and B show that areas of deep ponding will occur behind the dunes in the following general locations:

- Tarlton Street, Somerton Park
- Minda Homes property
- Jetty Road, Brighton (east of the railway)
- Edwards Street, Brighton

Each of these locations is in relatively close proximity to the coast, and as a result, the feasibility of constructing outfalls to cater for the 1 in 100 year flows has been investigated and is discussed below.

There are a number of other locations throughout the catchment where floodwaters accumulate during a 1 in 100 year event. Most of these areas are of relatively limited extent, and as a result, the cost of works necessary to address the particular issue is likely to exceed the benefit in terms of reduced flood damage.

The modelling has shown that there is the potential for a particularly deep area of flooding to occur between Melton and Moore Street, Somerton Park in a 1 in 100 year event. This area is located some distance from a potential outfall point for these major flows, and as a result, the cost of undertaking works to reduce potential flooding in this area is unlikely to be balanced by the benefit.

It is particularly important that new development occurring in the areas identified above have strict controls placed on floor levels so that over time an improved standard of protection is achieved.

#### Tarlton Street Area

Flood flows reaching the Tarlton Street area in a 1 in 100 year event pond primarily in properties on the western side of the road between Whyte Street and Marine Street. Floodwaters overflow from this area further to the north into low-lying areas around Yarrum Grove.

Modelling has shown that construction of a new outfall from this area that collects flows in Tarlton Street, has the potential to substantially reduce the extent of flooding in the area and prevent the overflow of floodwaters further to the north. The capacity of this outfall would need to be approximately 4.3 $m^3/s$ to cater for the 100 year event.

A possible alignment for this outfall is shown in Figure 7.1. The proposed outfall extends along Wilkinson Avenue into Tarlton Street and then runs north to Phillips Street and into Turner Street. The area around Tarlton Street is very low lying and there is limited grade available for the proposed outfall. Preliminary sizing has indicated that a 2.7 m x 1.2 m box culvert will be required to carry the flow, with a significant number of inlets constructed along Tarlton Street to collect the floodwaters.

Construction of this outfall will reduce or prevent flooding of approximately eighty properties in the area, some of which will be inundated to depths of up to 800 mm in a 100 year event.

The feasibility of constructing this outfall requires further investigation in relation to geotechnical conditions, services and the interaction with the existing drain in Tarlton Street. Such a feasibility investigation would also enable a firmer cost estimate for the drain to be derived.

### Minda Homes Site

The pattern of flooding around the Minda Homes site is shown in Figure 7.2. Floodwaters enter the site along the eastern and southern boundaries, pond within the site and flow further to the north.

Development of the site is currently being planned. This development will need to cater for the identified flows to ensure that:

- flow paths are provided through the development so as not to back flood waters up into surrounding properties; and
- new allotments within the development are sited above the predicted 1 in 100 year flood level.

As a part of the preparation of development proposals for the site, the City of Holdfast Bay and Minda have been working collaboratively to investigate potential flood mitigation options. This investigation has considered the potential for establishing flood detention storages in the upstream catchment on Bowker Reserve and Brighton Secondary School, the impact of increasing the capacity of drainage systems serving the area around Walsh Street to the south of the Minda site as well as construction of a wetland / flood detention storage within Minda in conjunction with an upgraded outfall.

The flood mitigation benefit associated with the various works investigated are set out in Table 7.1 below.

*Table 7.1 Flood Mitigation Benefits of Various Potential Works In the Vicinity of Minda Homes*

| Component | Benefits |
|---|---|
| Walsh Street Drain | • Reduces overflow into the Minda site from the south in a 1 in 100 year event by 0.27 $m^3/s$.<br>• Improves flood protection to residences in the vicinity of Walsh Street. |
| Outfall from Minda Homes Wetland / Detention Basin | • Reduces flood levels at the lowest point of the Minda site from 5.6 mAHD to 5.43 mAHD.<br>• Reduces overflows into the Minda site from King George Avenue by approximately 1 $m^3/s$ by redirecting existing underground systems away from the King George Avenue system.<br>• Reduces flooding of properties north of Minda. |
| Bowker Reserve Basin | • Reduces flooding in the area immediately downstream of the reserve. Impact on flooding at the Minda site is small. |
| Brighton Secondary School Basin | • Reduces overflows into the Minda site by approximately 0.7 $m^3/s$.<br>• Reduces the peak flow in the proposed outfall by 0.5 $m^3/s$<br>• Reduces flood levels at the lowest point of the Minda site from 5.43 mAHD to 5.18 mAHD (if constructed in conjunction with the outfall). |

Based on the above assessment, Minda and Council have been progressing implementation of a scheme based on construction of a wetland / detention basin in the Minda site in conjunction with a new outfall. The potential benefit to be gained in optimising the outfall design by construction of a basin in Brighton Secondary School warrants further consideration as it has the potential to reduce flood levels in Minda and reduce the size (and cost) of the outfall, and would be subject to the availability of land within the school for this purpose.

A possible alignment for the proposed outfall is shown in Figure 7.1. The outfall extends from the Minda site, along Repton Road, Prior Road and Harrow Road to the coast. Preliminary sizing has indicated that the outfall from Minda to Harrow Road would be a 2.7 x 0.9 BC (design flow 4.3 $m^3/s$) and the section along Harrow Road would be a 3.3 x 1.5 BC.

It is anticipated that the proposed works will be constructed on a shared cost basis between Council and Minda, although agreement on the cost sharing proportions has yet to reached.

### Jetty Road Area

The floodplain mapping has indicated that significant ponding of stormwater may occur immediately to the east of the railway line at Jetty Road, Brighton in a 1 in 100 year event.

Modelling has shown that construction of a new outfall from this area that collects flows along the railway between Preston Avenue and Commercial Road has the potential to substantially reduce the extent of flooding in the area. The capacity of this outfall would need to be approximately 6.4 $m^3/s$ to cater for the 100 year event.

A possible alignment for this outfall is shown in Figure 7.1. The proposed outfall extends along Jetty Road and then along the railway in Torr Avenue and into Preston Avenue. Preliminary sizing has indicated that a 3 m x 1.5 m box culvert will be required to carry the flow, with inlets to the system provided by swales and inlets adjacent to the railway reserve.

Construction of this outfall will reduce or prevent flooding of approximately thirty properties in the area, some of which will be inundated to depths of up to 500 mm in a 100 year event.

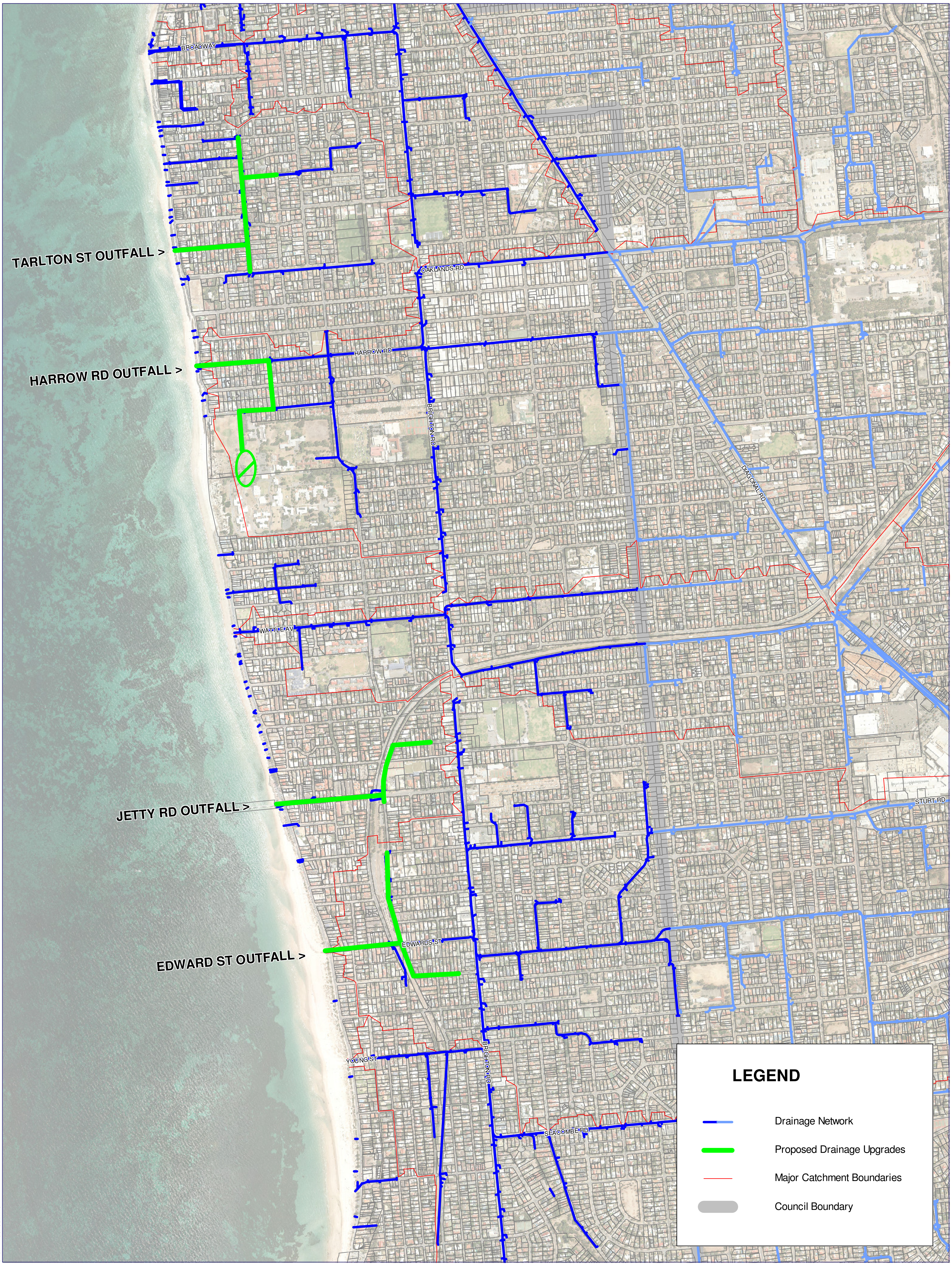
The feasibility of this outfall requires further investigation in relation to geotechnical conditions, services and the interaction with existing infrastructure along the railway. Such a feasibility investigation would also enable a firmer cost estimate for the drain to be derived.

### Edwards Street Area

Modelling of the area in the vicinity of Edwards Street, Brighton has shown the potential for significant ponding of stormwater to occur in a 1 in 100 year event.

In order to reduce this flooding, the possibility of constructing a new outfall that collects flows along the eastern side of the railway in Commercial Road and Scott Street and Oleander Street was modelled. The modelling showed that construction of such an outfall has the potential to substantially reduce flooding in the area. The capacity of the outfall would need to be approximately 16.3 $m^3/s$ to cater for the expected 100 year flows.

The most convenient location for the outfall is along Edwards Street, as alignments along roads to the north and south of Edwards Street (Marlborough Street and Oleander Street West respectively) would require the outfall to cut through coastal dune ridges. However, the presence of the existing trunk drain system in Edwards Street is likely to prevent the construction of a second trunk outfall. Construction of the outfall is therefore most likely to require replacement of the existing system with a new drain. Preliminary sizing has shown that the new outfall will need to be approximately 3.6 x 2.4 m in size.



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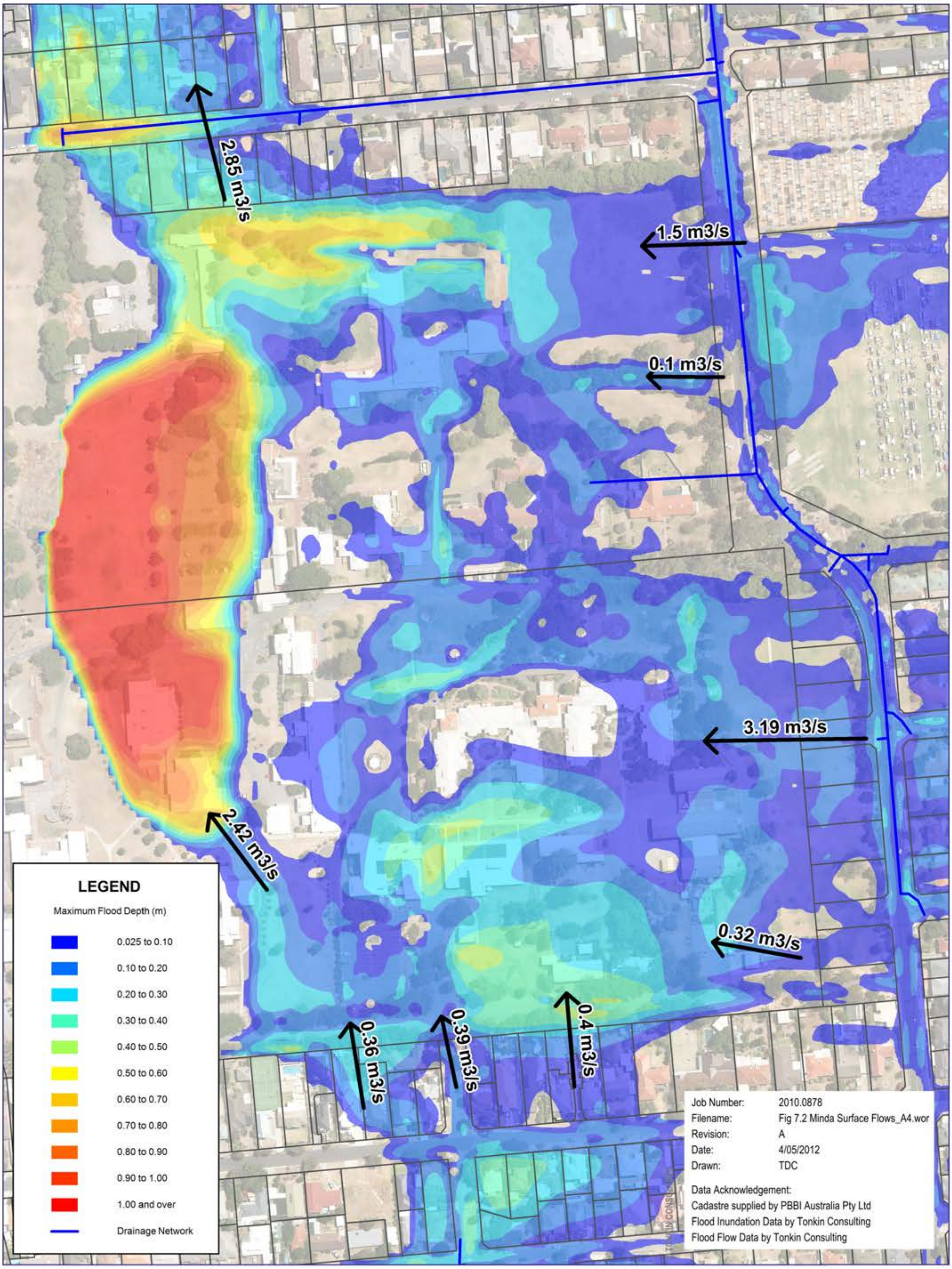
**Cities of Marion and Holdfast Bay**

**PROPOSED MAJOR DRAINAGE SYSTEM UPGRADES**

**Figure 7.1**

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City of Holdfast Bay

**MINDA SURFACE FLOOD FLOWS**
**100 YEAR ARI LONG-TERM SCENARIO**

Figure 7.2

Construction of this outfall will reduce or prevent flooding of approximately forty-five properties in the area, some of which will be inundated to depths of up to 800 mm in a 100 year event.

The feasibility of this outfall requires further investigation in relation to geotechnical conditions, services and the interaction with existing infrastructure. Such a feasibility investigation would also enable a firmer cost estimate for the drain to be derived.

### 7.1.2 Upgrades to the Minor Drainage System

Extension of the underground drainage system is proposed in those areas where the modelling has shown there are likely to be significant gutter flows. Locations having inlet flows of greater than 300 L/s in a 1 in 5 year event (approximately three times the capacity of a typical double side entry pit) were selected as the highest priority locations for these extensions, giving rise to a total of twenty-five drain extension projects across the catchment. These are listed below, together with a brief description of each project. The locations are shown in Figure 7.3 together with a proposed alignment for the drains and the catchment area served. It should be noted that in each case, a more detailed investigation and preliminary design will be required to confirm the final layout and extent of works required. It is proposed that these drains be constructed having a 1 in 5 year design standard unless noted otherwise below.

#### Location 1: Francis Avenue, Glengowrie

This drain extends from the intersection of Elder Terrace and Gowrie Avenue to Francis Avenue so as to intercept a flow in excess of 300 L/s which reaches the corner of Francis Avenue and Maxwell Terrace. There is an existing 450 mm diameter stub at Elder Terrace for the connection of the new drain.

#### Location 2: Whiteleaf Crescent, Glengowrie

This drain extends from Diagonal Road along Whiteleaf Crescent and St Peters Way to intercept flows from a catchment that currently reaches the intersection on Panton Crescent and Baker Street. This drain was proposed as part of the South Western Suburbs Scheme.

The drain lies within the Cities of Holdfast Bay and Marion, and a cost-sharing arrangement will be required.

#### Location 3: Bombay Street, Oaklands Park

This drain extends from Bombay Street, along Campbell Street and into Rangoon Street, Oaklands Park to intercept surface flows that currently reach the corner of Bombay Street and Barry Road. The drain was proposed as part of the South Western Suburbs Scheme.

#### Location 4: Soho Street, Warradale

This drain runs along Soho Street and discharges to the north into the existing drain in Cedar Avenue. The drain intercepts flows from a catchment which lies between Soho Street and Morphett Road and extends southward to Keynes Avenue.

#### Location 5: Crozier Terrace, Oaklands Park

This drain runs from the intersection of Crozier Terrace and Diagonal Road, along Selway Street and into Johnstone Road. The drain intercepts surface flows which currently reach the intersection of Crozier Terrace and Selway Street. The existing drainage system serving this intersection discharges to the Sturt River at a flat grade and has limited capacity. For this reason, it is proposed to drain this area back toward Diagonal Road where the drains downstream of the discharge point appear to have greater capacity.

### Location 6: Dwyer Road, Oaklands Park

This drain runs from Diagonal Road along Dwyer Road to Johnstone Road. The system intercepts flows from a sizable catchment which current runs along Dwyer Road. The drain was proposed as part of the South Western Suburbs Scheme and will connect to an existing 525 mm diameter stub at Diagonal Road.

### Location 7: Township Road, Marion

This system is proposed to reduce the surface flows arriving at pits in Finniss Street. The pipe system in Finniss Street has less than a 1 year standard, and it is therefore proposed to construct a new outfall from Township Road along Finniss Street to the Sturt River.

In addition, extension of the existing system in Norfolk Road to intercept flows just east of Township Road will further reduce these surface flows.

### Location 8: Grandview Grove, Sturt

The Sturt area currently has little underground drainage. A significant catchment drains to inlets at the intersection of Grandview Grove and Meadowvale Road. There is an existing 525 mm diameter drain that runs from this intersection and connects with the drain in Sturt Road. It is proposed that the existing underground drainage be extended into the upstream catchment to intercept surface flows.

The downstream system in Sturt Road has been assessed to have approximately a 1 in 2 year ARI design standard. As a result, the proposed drain extension for this catchment is proposed to also have a 2 year ARI design standard to match this capacity.

### Location 9: Travers Street, Sturt

There is a large sub-catchment that drains along Travers Street toward Diagonal Road, where the surface flow arriving at pits on the corner of Carlow Street and Diagonal Road is in excess of 400 L/s in a 1 in 5 year event.

It is proposed that a new drainage system be constructed in Travers Street to intercept flows from the catchment. Due to the limited capacity of both the Sturt Road and Diagonal Road drains, it is proposed that a small detention basin be constructed in the reserve located between Travers Street and Meyer Road to reduce flows from the drain (to say less than 50 L/s). The outfall from the basin can then be directed to the underground drainage system in Meyer Road.

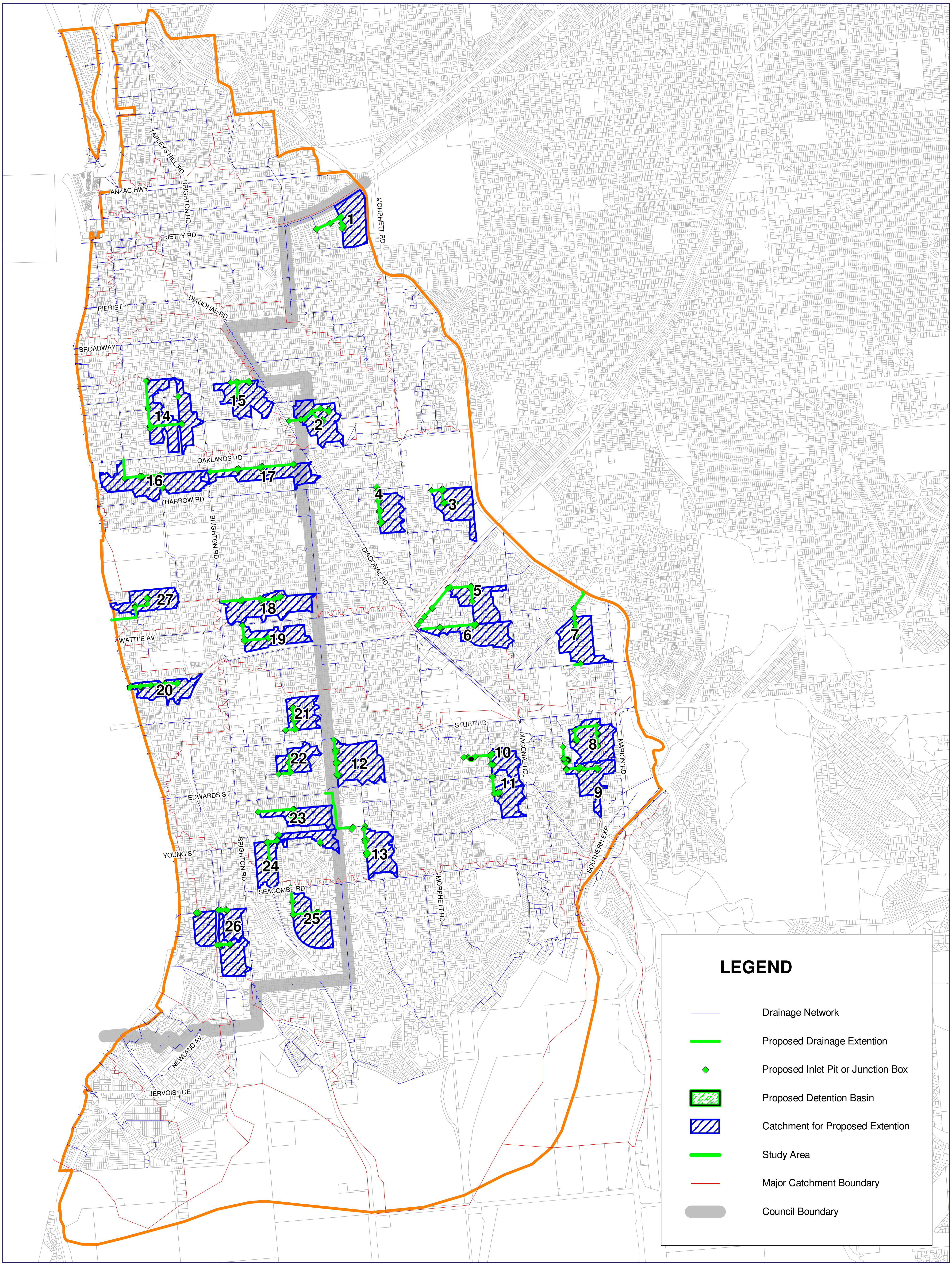
### Location 10: Glamis Avenue, Seacombe Gardens

This proposed drain intercepts flows from the Glamis Avenue catchment and re-directs them to an existing drainage system in Sweetwater Street further to the west. A detention basin is proposed in the reserve lying on the corner of Sandery Avenue and Sweetwater Street to reduce peak flows to be within the capacity of the downstream system.

### Location 11: Wilga Street, Seacombe Gardens

Inlets at the intersection of Shearer Avenue and Limbert Avenue receive excessive surface flows in a 1 in 5 year event. The catchment draining to these inlets lies to the south.

It is proposed to extend a drain along Glamis Avenue to the intersection of Wilga Street and Vardon Street to intercept flows from this catchment. The proposed system will also address property flooding that is currently being experienced at the junction of Harbrow Grove and Wilga Street.



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**Cities of Marion and Holdfast Bay**

**PROPOSED MINOR DRAINAGE SYSTEM UPGRADES**

**Figure 7.3**

### Location 12: Laurence Street, Dover Gardens

This drain runs along Laurence Street and intercepts flows from a number of side streets to the east of its alignment. The drain will reduce surface flows along Laurence Street which currently are collected by a single set of inlets at Sturt Road.

### Location 13: Quintus Terrace, Dover Gardens

Inlets draining the intersection of Quintus Terrace and Folkestone Road receive significant surface flows during a 5 year event. The downstream pipe system has less than a 1 year standard.

It is proposed that the existing drain in Neath Avenue be upgraded and extended to Seaforth Avenue to reduce flows in the Quintus Terrace drain. Interception of these flows will enable the Quintus Terrace system to be extended to the south to reduce surface flows reaching Folkestone Road.

Alternatively, it is possible that flows from this upstream catchment could be managing within a potential development of the Dover Gardens School.

### Location 14: Walkers Road/Scarborough Street, Somerton Park

In this area, inlets at the intersection of Walkers Road and Turner Street, as well as at Scarborough Street and Bond Street receive significant flows during a 1 in 5 year event. It is proposed that drainage works be undertaken in Walkers Road and along Cudmore Street to intercept flows within the catchment and reduce surface flows.

### Location 15: Moore Street, Somerton Park

This drain runs along Moore Street from Healesville Avenue and into Gilbert Road to intercept flows from a catchment lying to the south. Due to the limited capacity of the downstream drainage system, a 1 in 2 year standard is proposed.

### Location 16: College Road, Somerton Park

This drain extends from the intersection of Whyte Street and Tarlton Street into College Road and King George Avenue. The drain will reduce significant surface flows along College Road and in the southern section of Tarlton Street.

### Location 17: Byre Avenue, Somerton Park

Significant surface flows will reach Brighton Road from light industrial areas along Byrne Avenue. The proposed Byrne Avenue drain will intercept these flows.

### Location 18: Cecelia Street, North Brighton

This drain runs along Cecilia Street to intercept flows from a catchment lying generally between Cecelia Street and Dunrobin Road. The drain collects flows that currently reach inlets at Brighton Road.

### Location 19: Caroona Street, Hove

This drain extends from the intersection of Dunrobin Road and Caroona Avenue into Illawarra. Avenue. The drain intercepts excessive surface flows that currently will travel along Caroona Avenue and are collected at inlets on Dunrobin Road.

### Location 20: Alfreda Street, Brighton

Significant surface flows will currently travel along Alfreda Street and are collected at inlets on The Esplanade. The proposed Alfreda Street drain intercepts these flows.

### Location 21: McCoy Street, Brighton

Inlets near the intersection of Roberts Street and Essex Street have a surface flow of in excess of 400 L/s arriving in a 5 year event. It is proposed to extend the drain from Roberts Street into McCoy Street to intercept these flows.

### Location 22: Rudford Street, Brighton

Inlets near the intersection of Fulton Street and Lewis Street have a surface flow in excess of 400 L/s arriving in a 5 year event. It is proposed to extend the drain from Fulton Street into Rudford Street to intercept these flows.

### Location 23: Yarmouth Street, South Brighton

This drain extends along Yarmouth Street from Margate Street, and then turns south to collect flows around Dover Square. The drain will reduce surface flows along Yarmouth Street.

### Location 24: High Street, South Brighton

This drain collects flows from High Street, to significantly reduce the surface flows arriving at inlets near the intersections of Stephenson Avenue, Deer Street and Folkestone Road. In addition to these works, it is proposed that additional inlets be constructed to intercept flows at the intersection of Clifford Street and Morgan Street. This will reduce the surface flow along Folkestone Road and limiting flows reaching the intersection with Deer Street.

### Location 25: Ophir Crescent, Seacliff Park

There is a large catchment to the south of Seacombe Road which drains to inlets at the intersection of Ophir Crescent and Seacombe Road. It is proposed that a new drain be constructed to intercept flows within this catchment. The drain will extend along Ophir Crescent and into Mott Terrace.

### Location 26: Wheatland Street, Seacliff

There are two locations along Wheatland Street where significant surface flows are likely to arrive at inlets during a 5 year event. These locations are at the railway line and at Myrtle Road. Additional inlets and a minor pipe extension from the railway to Yacca Road are proposed.

In addition, construction of a second drain extension further south within the catchment is proposed along Maitland Terrace.

### Other Projects – City of Marion

In addition to the above, the City of Marion has a number of other upgrade projects programmed within the catchment to rectify localised deficiencies in the existing drainage system. These projects have been identified from resident complaints of property flooding and staff observations. Council has developed a prioritised list of these projects. The projects which have the highest priority from this list include:

- Davenport Terrace, Seaview Downs

  Council has completed the first stages of drainage works in Davenport Terrace, Seaview Downs. These works will address property flooding which has occurred due to overflow of floodwaters from roadways within the area to the east of Davenport Terrace. A number of lateral branches remain to be constructed. These branches are required to intercept flows upstream of the properties being flooded.

- Radstock Street, Morphettville

  The cul-de-sac off Radstock Street, Morphettville is currently drained via a surface flow path along a laneway which runs toward the north. The laneway has limited capacity and adjacent residences have floor levels which are only just above top of kerb. Development within the catchment is ongoing and in a large event it is possible that flooding of these adjacent residences could occur.

  A new pipe system to the Sturt River is proposed to address this issue.

- Jervois Terrace, Marino

  The City of Marion has completed design and commenced construction of the first stages of works within Bandon Terrace and Jervois Terrace, Marino which address property flooding at a low point in Jervois Terrace and replaces existing structurally deficient sections of drain.

- Brigalow Avenue, Seacombe Gardens

  Council has had a reported issue of property flooding at the intersection of Brigalow Avenue and Morphett Road. The residence on the northern eastern corner of the intersection is of slab on ground construction and is set below road level. The peak 5 year ARI flow reaching inlets at the intersection is shown in Figure 4.5 as being in the range between 200 and 300 L/s. Some extension of the existing underground drainage system (possibly in conjunction with modifications to the footpath and driveway crossover levels in front of the property) may be required to address this issue.

The priority ranking of these additional projects has been incorporated into Section 14.

#### Other Projects – City of Holdfast Bay

In addition to the projects listed above, the City of Holdfast Bay has received resident complaints about significant flooding of roadways in the vicinity of Walsh Street at Hove. This area is located in a localised depression behind the coastal dunes to the west. It is understood that floodwaters have been reported to enter properties in the area after heavy rainfall and this behaviour is confirmed by the floodplain mapping, which indicates flooding of some allotments in a 5 year ARI event.

Options for upgrading the existing pipe system have been investigated separately to the preparation of this Stormwater Management Plan. These investigations have shown that in order to achieve a significant increase in the design standard of drainage in the area, the entire length of the outfall running from Walsh Street to the coast would need to be upgraded. Given that the proposed works drain a low point, a 20 year standard has been proposed for the drain.

Construction of this drain is also likely to reduce (slightly) the overflow of floodwaters into the Minda Homes site which lies immediately to the north.

The priority ranking of this project has been incorporated into the recommendations contained in Section 14.

### 7.1.3 Lorenzin Site

Development of the Lorenzin site will require two primary issues to be addressed in relation to stormwater management; these being:

- protection of the development from flooding due to flows from the upstream catchment;
- management of runoff from the development (and the upstream catchment) so that frequent flooding of the intersection of Schofield Road and Newland Avenue is addressed.

Each of these issues is discussed below.

#### Protection of Development from Flooding

The floodplain maps show that in a 1 in 100 year event, the path followed by floodwaters through the site is generally along the southern boundary and then diagonally across the site to the Newland Avenue intersection.

It is understood that land along the western boundary of the site is currently not being considered for development due to contamination issues. A potential strategy for dealing with flooding would be to provide an overflow path along the southern and western site boundaries to cater for the 1 in 100 year event. The existing site topography would appear to lend itself to such a strategy, with the formation of the flow path being undertaken by construction of bunding to prevent overflow of floodwaters into the proposed residential areas of the site.

The cost of providing this overflow path should be borne by the developer of the land.

#### Management of Flooding and Downstream Impacts

The floodplain mapping shows that despite the system downstream of the site having less than a 1 in 1 year standard, the road network will carry a substantial portion of the 1 in 100 year flows. However, there is an issue of frequent (nuisance) flooding of roadways downstream of the site as a result of runoff from the site itself and from the upstream catchment.

As a minimum, works should be undertaken in association with development of the site to mitigate these downstream impacts. The design standard of these works should be such that the downstream issues associated with nuisance flooding are addressed for events having up to a 1 in 5 year ARI.

Mitigation of larger events would be desirable, given the magnitude of flows involved, but may be difficult to achieve due to a number of constraints associated with the site, including topography and contamination from past land uses.

Preliminary sizing of a detention storage on the site was undertaken for a 1 in 5 year event. This basin was designed to limit outflows to 0.1 $m^3$/s in such an event. The basin storage required was calculated to be of the order of 3500 $m^3$.

A basin of this size could be constructed on the existing Les Scott Reserve adjacent to the Newland Avenue intersection by construction of an embankment along the northern and western boundaries. Such a basin would permit drainage from development of the Lorenzin site to be directed into it.

Increasing the size of the basin to provide greater storage is likely to require excavation of the reserve. It is understood that this is likely to be extremely costly due to site contamination issues, and has therefore not be investigated further at this stage.

## 7.2 Strategy 1.2

**Where necessary develop planning policy and design techniques to ensure that Objectives 1.1, 1.2, and 1.7 are met.**

### 7.2.1 Infiltration Systems

#### Description

Infiltration systems generally consist of a shallow excavated trench or “tank”, designed to retain a certain volume of runoff and slowly infiltrate the stored water to the surrounding soils. As a consequence they reduce runoff volumes by providing a pathway for treated runoff to recharge local groundwater aquifers. They can also reduce the peak flow rate off a site, since they are able to absorb the first part of the hydrograph. The impact that they have on peak flows is greatest for short duration storms.

Infiltration systems typically consist of a storage that is made up from void spaces in media such as single size gravel, or are underground structures such as Ausdrain or Atlantis Cells. The storage is usually wrapped in geotextile type fabric and “clean” water is allowed to infiltrate into the surrounding soil. It is important that infiltration systems only receive “clean” water; even runoff from roofs can require some form of pre-filtering.

A typical infiltration system is illustrated in Figure 7.4.

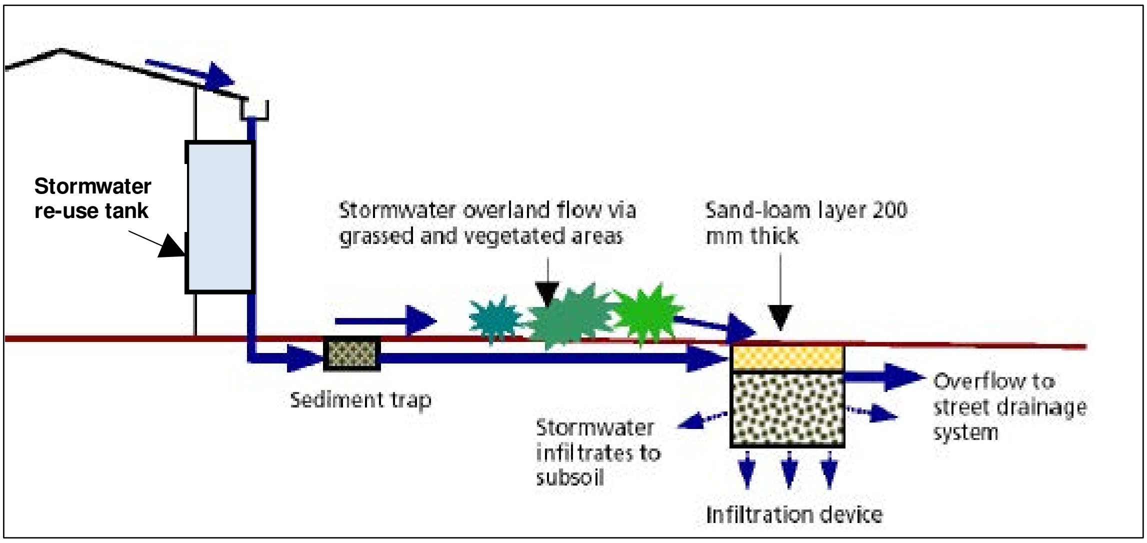



*Figure 7.4 Typical Infiltration Strategy*

Infiltration systems are an effective tool for achieving reductions in peak flow rates and volumes of runoff from developed sites.

### Effectiveness of Infiltration Systems as a Flow Rate Reduction Technique

To assess the effectiveness of on-site infiltration as a tool to mitigate the impacts of increasing flows arising from redevelopment of the catchment, an analysis was undertaken of scenarios with various extents of redevelopment and various retention volumes. This is reported on in more detail in Tonkin Report “Discussion Paper – Retention Storage Systems” but the conclusions from this work are discussed below.

The scenarios are based on a 1 ha catchment that prior to redevelopment has 27% directly connected impervious area, 26% indirectly connected impervious area, and 47% pervious area. Post redevelopment, it is assumed that a portion of the properties in the catchment are developed such that they have 85% directly connected impervious area and 15% pervious area. In all cases, the impervious area surface storage has been assumed to be 1 mm and pervious area surface storage 45 mm. The analysis assumed that runoff from all of the redeveloped allotments is discharged through an infiltration system and that the storage is empty at the start of the rainfall event.

The peak runoff from the 1 ha catchment for the various development scenarios and the rainfall depth used to determine the volume of the infiltration system are provided in Table 7.2 below.

*Table 7.2 Relationship Between Peak Runoff and Volume of Infiltration System*

| | Development scenario | | | | |
|---|---|---|---|---|---|
| Percentage of 1 ha catchment redeveloped to 85% imperviousness | None | 67% | 67% | 67% | 100% |
| Equivalent depth of on-site retention provided for redeveloped area | 0 mm | 0 mm | 10 mm | 15 mm | 15 mm |
| **Storm duration** | **Peak flow rates (L/s) for a 1 in 5 year ARI storm** | | | | |
| 5 minutes | 17 | 42 | 6 | 6 | 0 |
| 15 minutes | 34 | 82 | 20 | 11 | 0 |
| 30 minutes | 36 | 86 | 39 | 12 | 7 |
| 45 minutes | 34 | 81 | 45 | 25 | 33 |
| 1 hour | 34 | 82 | 53 | 30 | 40 |
| 1.5 hours | 30 | 72 | 60 | 35 | 45 |
| 3 hours | 26 | 62 | 41 | 30 | 39 |
| 4.5 hours | 24 | 57 | 30 | 28 | 37 |
| 6 hours | 17 | 39 | 37 | 25 | 33 |
| 12 hours | 17 | 43 | 35 | 35 | 26 |

Taking the case of a catchment that undergoes redevelopment of two-thirds of its area (likely to be typical of much of the Coastal Catchment), the following is concluded:

- Prior to redevelopment the peak flow is generated by a 30 minute storm and is 36 L/s.
- Post redevelopment the peak flow increases to 86 L/s if no retention system is provided.
- If an infiltration system having a volume equivalent to 10 mm of runoff from the impervious area on the redeveloped allotments is provided, the peak flow occurs during the 1.5 hour storm and is 60 L/s.
- If an infiltration system having a volume equivalent to 15 mm of runoff from the impervious area on the redeveloped allotments is provided, the peak flow occurs during the 1.5 hour storm and is 35 L/s.
- If an infiltration system having a volume equivalent to 15 mm of runoff from the impervious area on the redeveloped allotments is provided, peak flows are reduced when compared to the undeveloped case for all storms up to and including the 1 hour storm, whereas for a 10 mm storage, only storms with a duration of 15 minutes and less are reduced.

The last point is important in the context of a catchment, as most of the Coastal Catchments have a critical time of concentration of around 1 hour.

In the case of a catchment that is entirely redeveloped to be 85% impervious, 15 mm of on-site retention is insufficient to restrict flows to the original flows.

### Emptying Times

Emptying time for an infiltration system depends on the permeability of the surrounding soils. Australian Runoff Quality provides a guideline figure for the emptying of a trench at 1.5 days for a 5 year storm.

For a system sized to meet the requirement for storage of 15 mm of runoff from a 250 $m^2$ development with an impervious fraction of 85%, a trench 2.5 m x 4.9 m x 0.75 m deep would be required if it were filled with gravel with a voids ratio of 0.35. This is approximately the same area as one car park space. The emptying times for such a trench are estimated to be as listed in Table 7.3.

*Table 7.3 Emptying Time for 750 mm deep Infiltration Trench*

| Soil | Infiltration Rate m/s (mm/hour) | Time to Empty |
|---|---|---|
| Sandy Clay | 1 x$10^{-5}$ (36) | Approximately 0.5 days |
| Medium Clay | 1 x$10^{-6}$ (3.6) | 3–6 days |
| Heavy Clay | 1 x$10^{-7}$ (0.36) | 30–60 days |

It is noted that the investigation of soil permeability described in Section 2.9 indicates that across much of the catchment, infiltration rates are not likely to achieve the required emptying times. In this situation, a slow flow outlet is suggested in Australian Runoff Quality.

### Effectiveness of Infiltration Systems in Reducing Volumes of Discharge

On-site retention will reduce the total volume of water discharged from a site by the amount that can be infiltrated into the soil. The greater the infiltration capacity, the greater the potential to reduce off-site discharges. Hydrological effectiveness of a retention device is a measure of the volume of runoff retained by the retention system compared to the total runoff directed to the system. The hydrological effectiveness is dependent on the size of the system and the infiltration capacity of the soils.

For a device sized to collect 15 mm of rainfall as described for the analysis above, the hydrological effectiveness for various soils types is as set out in Table 7.4. The hydrological effectiveness values provided assume no ‘slow flow’ outlet. If such an outlet is provided, additional storage capacity below the slow flow outlet would be required to achieve the effectiveness values provided below.

*Table 7.4 Hydrological Effectiveness for Various Soils Types*

| Soil | Infiltration Rate m/s (mm/hour) | Hydrological Effectiveness (%) |
|---|---|---|
| Sandy Clay | 1 x$10^{-5}$ (36) | >95 |
| Medium Clay | 1 x$10^{-6}$ (3.6) | 90 |
| Heavy Clay | 1 x$10^{-7}$ (0.36) | 50 |

### Principal Benefits of Retention Devices Over Detention Devices

The benefits of retention over detention are that retention uses water on site for recharging soil moisture, and thereby reduces peak flow rates and volumes of stormwater discharged to the marine environment. Importantly, retention devices will reduce the number of events during a year where water leaves a site. Whilst detention may, if properly implemented, have an effect on peak flow rates, it will not reduce the total volume of water discharged, nor will it reduce the number of times during a year that stormwater is discharged from the site. These latter two outcomes are particularly important from the perspective of receiving waters and only a policy that requires retention will achieve these outcomes.

### Water Sensitive Urban Design (Consultation Statement)

As described in Section 6.2.4, the Water Sensitive Urban Design Consultation Statement (Department for Water, 2012) promotes the concept of capture of 10 mm of rainfall from new development and either the reuse or infiltration of this volume. Notwithstanding that the modelling undertaken for this study indicates a need to increase this amount to 15 mm, the intent of the proposal above is in line with the Consultation Statements.

### Minimum Clearances to Adjacent Structures

This specification (Planning SA, September 2003) contains 'deemed to comply' requirements for the positioning of infiltration systems on a site.

These requirements are applicable to soil types A (Little or no ground movement), and S (Slightly Reactive Sites), or Class M-D (Moderately reactive sites) where the characteristic surface movement is equal to or less than 25 mm. Under these conditions, the Ministers Specification SA 78AA (Planning SA, September 2003) specifies that:

- retention devices shall be located a minimum of 3.0 m from all property boundaries, (excluding front boundaries and/or reserves) and 3.0 m from footings of all structures located on the allotment;
- a minimum clear spacing of 1.0 m between the sides of the retention device and any service trench is required.

Most of the catchment has soil types having a characteristic surface movement of more than 25 mm and therefore these 'deemed to satisfy' requirements cannot be used.

For 'Heavy Clay' soils, having permeabilities in the range between 1 x $10^{-6}$ and 1 x $10^{-8}$, Australian Runoff Quality recommends a minimum clearance to adjacent structures of at least 5 meters. Application of this requirement is likely to result in developments having either less site coverage or greater setback distances from the front boundary (where it is likely the infiltration trench would need to be located).

## 7.2.2 Retention Tanks

### Description

A retention tank is designed to capture and store runoff. The captured water is then available for uses such as garden irrigation or for internal domestic uses such as toilet flushing or laundry use. By plumbing the tank into the laundry and/or toilet, water is able to be used at a relatively constant rate allowing runoff from subsequent rainfall to refill the storage.

Due to the quality of water required for in the house use, these tanks are typically only used to capture runoff from roofs as runoff from paved areas is not considered to be of suitable quality.

Retention tanks are generally applied at the lot level as above ground tanks adjacent the house, although construction of underground tanks is possible provided that an appropriate overflow can be constructed.

It should be noted that it is currently a mandatory building requirement for new Class 1 buildings to have an alternative mains water supply. This requirement is often met by installation of a 1 kL retention tank plumbed into the dwelling.

In relation to the application of these devices within the study area, the limiting factor is expected to be the size of the tank required to meet the stormwater management objectives outlined in Section 6.

### Effectiveness of Retention Tanks in Reducing Volumes of Discharge

A substantial portion of the runoff from the roof must be reused in the household in order to approach the EPA target values for reduction of runoff from the catchment. It is assumed that the tank will be used for irrigation (in summer only) and plumbed into the laundry and toilet. The reuse water demand volumes are set out in Table 3.2.

The average daily water consumption per household in the SA Metropolitan area is 521L (ABS 2011). The breakdown of water usage within the household is shown in Table 7.5.

*Table 7.5 Household Water Use Breakdown (SA Water 2011)*

| Location | % | L |
|---|---|---|
| Garden and Outdoor | 40 | 209 |
| Bath and Shower | 20 | 104 |
| Laundry | 16 | 83 |
| Kitchen | 11 | 57 |
| Toilet | 11 | 57 |
| Other | 2 | 11 |
| | **TOTAL** | **521** |

If stored runoff is used for laundry, toilet flushing and irrigation in summer, the following daily demand volumes would be the approximate usage;

- Summer Months (December – February) = 350 L/day
- Other Months = 140 L/day

It should be noted that the above data is for an average household. The typical style of redevelopment that has been occurring within the catchment has involved replacement of single residences on a large allotment with multiple residences having substantially less outside space (particularly landscaping). As a result, it is likely that the consumption data presented above is likely to over-estimate house hold water use for garden irrigation within these types of development. However, in the absence of other data, the above statistics have been used to undertake a preliminary assessment of the likely reductions in flow volume brought about by installation of these tanks. The results obtained from this analysis, which was based on assessing runoff from a 100 $m^2$ roof area, are provided in Table 7.6 below.

*Table 7.6 Flow Reduction via Retention Tank Reuse*

| Tank Size for 100$m^2$ Roof Catchment (kL) | % of Reuse Demand Met | % Flow Reduction |
|---|---|---|
| 2 | 39 | 81 |
| 3 | 42 | 87 |
| 4 | 44 | 90 |
| 10 | 48 | 98 |
| 20 | 63 | 100 |

On allotments where there is little landscaping, outdoor use would be significantly reduced below the averages on which the above analysis is based. In these circumstances much larger tank volumes would be required.

Added to the above, where water is used indoors, flow from paved areas is not able to be utilised. Runoff from such areas comprises between 30 and 50% of the total impervious area on an allotment. It this runoff is not captured by the tank, the hydraulic effectiveness of these systems (considering the allotment as a whole) would be in the range between 40 and 57%. Such a system is unlikely to meet the requirements for flow volume reduction from the catchment set out by the EPA.

### Effectiveness of Retention Tanks as a Flow Rate Reduction Technique

Modelling of the impact of retention tanks on peak flows was not carried out as a part of this Study. However, an indication of their effect and likely sizing can be inferred from work carried out by Beecham (2003) and Coombes et al (2001) where the capacity of rainwater tanks to contribute to flood control was assessed. This work indicated that it can be assumed that approximately one third of the tank volume provided can be assumed to be available for flood control.

If retention tanks were to be adopted as the preferred management strategy, then based on the above criteria, a tank size well in excess of 10 kL would be required to ensure there is sufficient empty volume to capture the first 15 mm of rainfall from a 250 $m^2$ roof.

## 7.2.3 Preferred Management Strategy

A number of conclusions have been drawn from the investigation and modelling of on-site retention systems. These conclusions are presented separately below for infiltration systems and retention tanks.

For infiltration systems:

- The inclusion of these as an on-site retention technique has benefits for both flood mitigation and water quality improvement and if properly designed and implemented can comply with the relevant objectives set out in Section 6.
- The devices should be sized such that the first 15 mm of storm runoff is retained. If this is achieved then a hydrological efficiency of 95% can be expected
- Provision of an infiltration device sized to accept the first 15 mm of rainfall will reduce peak flows from storm events up to and including the 1 in 5 year ARI event to pre-development levels.
- The hydraulic conductivity of the soil throughout much of the study area has been identified as a ‘Heavy Clay’. This means that the emptying time for an infiltration device after a large storm event is considerably beyond the recommended time given in the guidelines. On these sites, provision of a ‘slow flow outlet’ and additional infiltration storage below this outlet will enable these devices to provide the required flood management and flow volume control.
- The classification of soils within the study area will mean that site layouts for new development will need to address appropriate setback distances from infiltration devices and possibly special footing designs where these setbacks cannot be achieved. This is likely to impact on the site coverage and dwelling densities that are able to be achieved on redeveloped allotments.

For retention tanks:

- The inclusion of retention tanks plumbed into the laundry, toilet and used for irrigation has benefits for flow volume reduction from redeveloped allotments. However only roof water (approximately 50% of the total allotment impervious area) can be reused in this way without the inclusion of additional treatment measures.
- In order to reduce flows in line with the target values given by the EPA, a retention tank in conjunction with additional water quality improvement devices intercepting flows from other impervious surfaces on the allotment would need to be used.
- To effectively retain enough runoff to approach the EPA target of retaining the first 10mm of rainfall, a large retention tank would be required.
- Flood storage has not been investigated in detail for retention tanks in this report. However previous studies have led to the assumption that one third of the storage can be assumed to be available for flood control. Tanks in excess of 10 kL in size are likely to be required to meet the peak flow reduction targets necessary for allotments within the study area.

On balance, a strategy involving the use of storm water retention via infiltration is the preferred approach for the catchment. However, some flexibility in this strategy is likely to be required on individual sites, provided that the over arching objectives of peak flow and volume management are achieved.

## 7.3 Strategy 1.3

**Implement joint audit programs and policies to ensure that planning policy and design techniques are built and maintained to ensure they continue to function as required.**

As discussed above, a key component of the proposed strategy involves the design and construction of devices on private allotments as part of redevelopment of these sites.

This approach will be driven through the planning process and will rely on appropriate changes being made to the current Development Plans of both Councils. This is discussed further in Section 11.

In addition, there will be an increased need to monitor compliance both in the construction and ongoing operation of these systems. Appropriate staffing and resources will need to be applied to these programs.

# 8 Description of Strategies – Runoff Quality

The strategies proposed to address runoff quality issues in the catchment are described in the following section.

## 8.1 Strategy 2.1

**To the extent that it is technically possible and financially viable, the road and drainage network should be progressively retrofitted with WSUD devices that strive to capture and treat road runoff to meet the water quality improvement requirements for stormwater required by the Adelaide Coastal Waters Study and other relevant state government policy.**

Environmental enhancement will occur through use of water sensitive urban design (WSUD) measures. WSUD is an approach to water management that includes all key aspects of stormwater management, but in particular, non-flood management issues. The overall benefit of WSUD is the opportunity to improve water quality and enhance the character of an area. This comes in the form of reduced mains water use for watering, reducing downstream export of pollutants and creating water elements as part of urban areas.

A WSUD approach to stormwater is primarily about using “soft engineering” approaches that provide a buffer for stormwater before reaching a receiving environment, in this case Gulf St Vincent.

The intent is to slow flows starting from the top of the catchment and to provide filtration and sediment removal from stormwater so as to prevent the majority of stormwater pollutants from leaving the area. This is particularly important in the study area, as this is a well built-up urban zone with space constraints. An end-of-line solution is in most cases not practical due to lack of space. Therefore, an approach to treatment distributed throughout the catchment is required. This should be retrofitted to the existing streetscape throughout the catchment.

With the inclusion of WSUD, flood conveyance will still remain the primary function of the stormwater management system. The effective inclusion of WSUD measures does not compromise the objectives of flood management. When WSUD stormwater quality treatments are implemented, these will be required to accommodate flood flows.

In many cases WSUD devices retrofitted to the existing streetscape can enhance the urban landscape while providing treatment and flood conveyance. These systems essentially become self irrigating allowing for some water to be contained within the site rather than disposing of it downstream.

### 8.1.1 Proposed WSUD Measures

Both Councils have commenced construction of WSUD techniques in their Council areas. Examples include Harbrow Grove in the City of Marion and the foreshore works in the City of Holdfast Bay.

The Councils will consider the implementation of WSUD techniques during all asset upgrades, and as a part of the ongoing business of Council on a project-by-project basis in the context of Council’s overarching objective of progressing towards becoming “Water Sensitive Cities”.

The following WSUD measures have been considered appropriate for consideration in this catchment.

#### Wetlands

The study area is well built up and urbanised, and there is limited open space available (particularly close to the coast). Open space that is available is already valued for its current use such as sporting grounds. The area of wetlands required to achieve the target reduction levels for pollutants from the entire catchment is approximately 48 ha, or 2% of the study area discharging to the Gulf.  This would be required to be spread over the various catchment outlets along the coast and include approximately another half of this area again as open space.

There is insufficient open space in the catchment to make this a realistic option on a broad scale, and no specific projects are proposed as a part of this plan.  However, the Councils will consider wetlands as opportunities as sites and projects suitable for small-scale wetland developments are developed.

### Biofiltration

A biofiltration system is a vegetated soil filtration system that provides efficient sediment and nutrient removal from stormwater.  The system consists of a vegetated swale or basin over a porous filter medium with a drainage pipe at the bottom.  Stormwater is diverted from a kerb or pipe into the biofiltration system, where it flows through the vegetation and temporarily ponds on the surface before infiltrating down through the filter media.  Treated water is then collected in a perforated pipe at the base of the filter media.

Pollutants are retained in the system through enhanced sedimentation, fine filtration and biological uptake in the vegetation.

Healthy vegetation plays a key role in maintaining the function of biofiltration systems.  The maintenance of a biofiltration system requires promoting healthy vegetation, removing excess collected sediments, ensuring the surface remains free draining and removing any material that blocks hydraulic structures.  Following establishment, inspections are required approximately every 6 months to ensure operation is maintained.

Biofiltration systems also receive significant runoff during rainfall events; therefore the vegetation often grows more vigorously than other street-side vegetation.  They can also include a “wet zone” for water storage and reuse during times of less rainfall.  They essentially self irrigate and can become green nodes within streetscapes.

The design of biofiltration systems is flexible, and they can be distributed fully throughout the catchment by retrofitting to the existing streetscape.  Road reconstruction or drainage system upgrades provide an ideal opportunity to progressively implement biofiltration within the catchment.

Ultimately to meet the water quality targets, modelling suggests that all streets will need to be fitted with biofiltration devices.

### Pervious Pavement

Pervious pavement is a load bearing pavement structure that is permeable to water.  The common features of pervious pavements include a permeable surface layer overlying an aggregate storage layer.  The reservoir storage layer consists of crushed stone or gravel which is used to store water before it is infiltrated to the underlying soil or discharged towards a piped drainage system.

Pervious pavements effectively strip a proportion of the runoff from urban areas and infiltrate this to underlying soils and groundwater.  They also provide limited water quality control, primarily through mechanical filtration processes.  Pervious pavements can improve the water quality of runoff through several processes, including:

- filtration of runoff through the pavement media and underlying material
- potential biological activity within the base and sub-media
- an overall reduction of pollutants discharging from site through reduced runoff volumes.

In the study area, pervious paving could potentially be used in streets with low traffic volumes and light traffic weight, and areas such as parking bays.  This would be most effective in areas close to the coast with underlying sandy soils with high infiltration rates.

## 8.2 Strategy 2.2

**Redevelopment of open spaces and other community facilities should incorporate WSUD devices that strive to meet the water quality improvement requirements for stormwater required by the Adelaide Coastal Waters Study and other relevant state government policy.**

The Councils will consider WSUD techniques as part of the redevelopment of open spaces and community facilities. These will be assessed on a project-by-project basis in the context of Council's overarching objective of progressing towards becoming "Water Sensitive Cities".

## 8.3 Strategy 2.3

**Develop statutory planning policy to require that all new development incorporates WSUD techniques that assist in meeting pollutant reduction targets.**

The Development Plans for both the City of Marion and City of Holdfast Bay contain many Objectives and Principles of Development Control that support the need for responsible stormwater management from the perspective of quantity, quality and reuse and that encourage WSUD techniques.

As set out in Section 7.2.1, infiltration is considered to be a suitable WSUD technique with multiple benefits including reduction of peak flows, reduction of the volume and frequency of stormwater discharges, and also improvements to soil moisture. As noted in that section, a number of issues need to be dealt with in designing infiltration systems as a part of all new development.

Proposed changes to the existing Development Plans are further discussed in Section 11.3.

## 8.4 Strategy 2.4

**Wherever technically feasible and without compromising flood protection objectives, all stormwater outlets discharging to the Adelaide beaches should be fitted with GPTs.**

GPTs are devices for the removal of litter and debris conveyed by runoff primarily by screening and rapid sedimentation. There are a variety of GPTs currently suitable for use in urban catchments including gully baskets, in-ground GPTs, trash racks and pipe nets.

GPTs are designed to collect solid material flowing in stormwater. For most catchments the proportion of debris that is litter is very small. The vast majority of material will be organic (leaves and twigs etc.). Trapping leaves and debris will tend to fill up the GPT and require maintenance, while the proportion of litter will commonly be less than 5%.

GPTs can be installed underground within an existing pipe network, in open channels or at outfalls. GPTs are typically the first line of defence when addressing stormwater quality because they target the coarsest pollutants.

There are several GPTs in the study area which are primarily retrofitted to the outlet systems.

New GPTs are proposed in the locations shown on Figure 8.1.

## 8.5 Strategy 2.5

The amount of pollutants entering the drainage network should be minimised by maintaining effective programs for:

- cleaning and maintenance of GPT
- street sweeping
- installing and regular emptying of rubbish bins, particularly in commercial precincts
- management of unpaved road verges to minimise sediment transport.

## 8.6 Strategy 2.6

The amount of pollutants entering the drainage network should be minimised by maintaining enforcement programs to ensure compliance with:

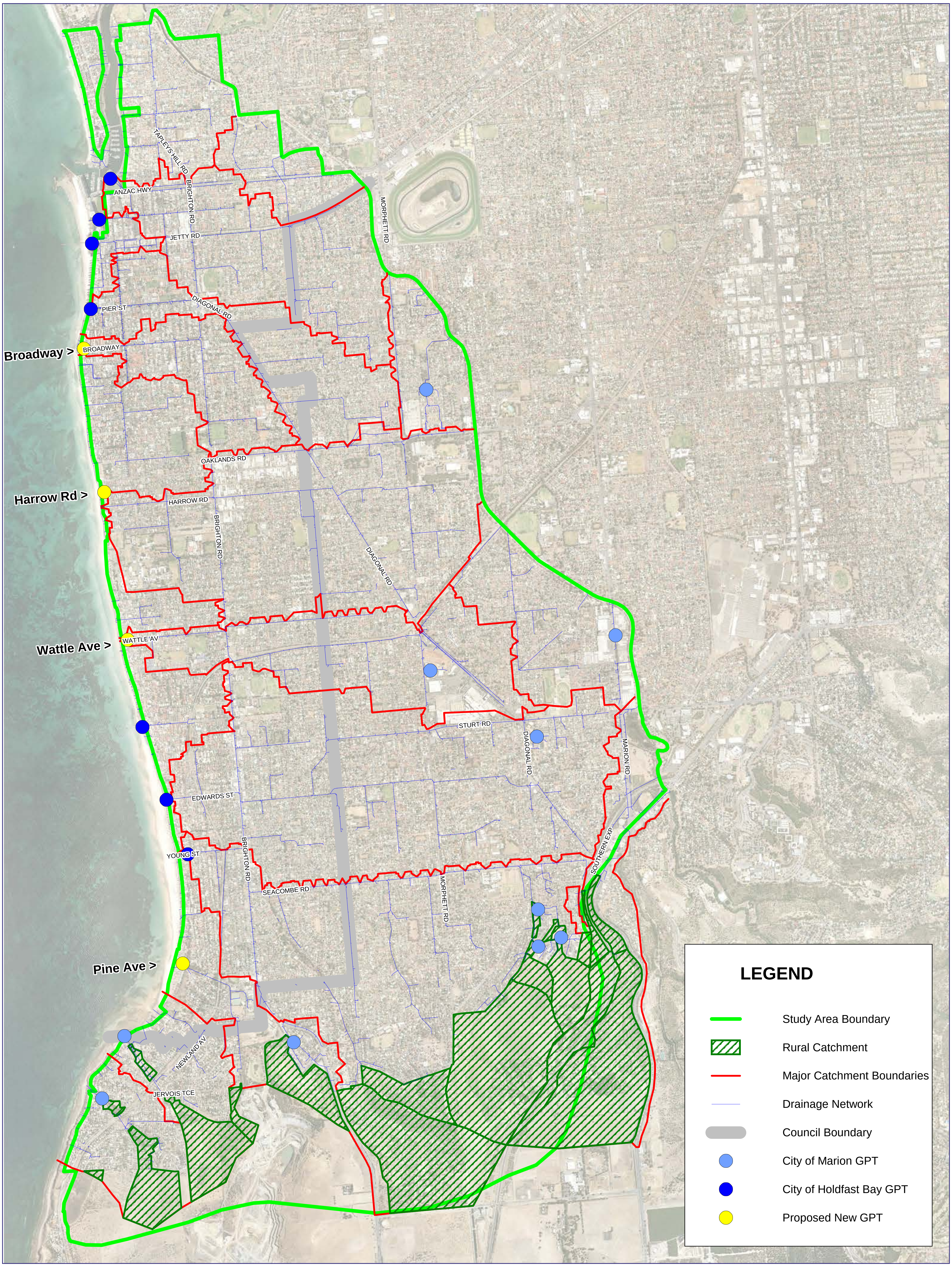
- anti-littering laws
- codes of practice for the building sites and the construction industry
- responsible chemical use.

## 8.7 Strategy 2.7

Soil erosion from hillsides and gullies should be managed by adopting appropriate land management practices and revegetation and stabilisation of gullies.

## 8.8 Strategy 2.8

Cooperate with other agencies to develop and conduct stormwater quality monitoring and reporting programs.



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**Cities of Marion and Holdfast Bay**

**PROPOSED GROSS POLLUTANT TRAPS**

**Figure 8.1**

# 9 Description of Strategies – Stormwater Reuse

The Councils will continue to explore opportunities for reuse as they arise and work with State Government agencies, private interests and others to promote opportunities. In particular the following existing initiatives are noted:

### *Potential for Large-Scale Wetland and MAR Schemes*

The study area has very little open space available for wetlands, but notable opportunities immediately adjacent to the study area include Westminster College, Warriparinga Wetlands and the Warradale Army Barracks.

### *Oaklands Park Wetland MAR*

Plans have been developed for the Oaklands Park MAR scheme which is outside of this catchment, but proposals exist to reticulate stormwater from this scheme into parks and other development nodes within the catchment in the City of Marion.

### *Recycled Water Use*

The City of Holdfast Bay currently uses recycled water to irrigate foreshore reserves in Glenelg. A southern extension of the recycled water pipeline is proposed to irrigate additional foreshore reserves.

It should be noted that it is currently a mandatory building requirement for new Class 1 buildings to have an alternative mains water supply. This requirement is often met by installation of a 1 kL retention tank which is plumbed into the dwelling and used for toilet flushing. It would be desirable to increase the minimum tank size provided on residences to increase reuse.

# 10 Description of Strategies – Watercourses

There are a limited number of open watercourses in the catchment and those that exist are in the southern portion. The gullies in the hills face area at the southern end of the catchment are generally in private ownership and responsibility for maintenance of these watercourses rests with the landowners.

Two other watercourses of note are Gilbertson Gully and Pine Gully. These are discussed below.

## 10.1 Gilbertson Gully

The downstream end of Gilbertson Gully above Seacombe Road is owned by Council. This watercourse is to be retained and development not permitted on this land. An opportunity exists to improve water quality, low flow detention, and potentially minor water harvesting through the implementation of some WSUD techniques on the watercourse land.

A detailed stormwater management plan incorporating WSUD principles is required for the Gilbertson Gully catchment and the Councils will seek funding opportunities to develop and implement a plan for the gully.

## 10.2 Pine Gully

Pine Gully is downstream of Linwood Quarry, and whilst it is currently in poor condition, it would provide an opportunity to provide local open space to restore the watercourse and to better manage stormwater by providing water quality improvements, low flow detention and minor water harvesting.

A land agreement with the current landowner is required for these outcomes to be realised and a detailed stormwater master plan incorporating WSUD principles is required for the whole of the catchment including Linwood Quarry and the Lorenzin site (refer Section 7.1.3). The Councils will seek funding opportunities to develop and implement a plan for the catchment.

# 11 Description of Strategies – Planning

## 11.1 Background

Sections 2.4 and 2.5 of this Plan describe the current development trends in the catchment and the likely impacts on future development densities and impervious area that would result if these trends continue. The impact of this development has been identified as a significant increase in the peak and volume of runoff from the catchment unless strategies are implemented to address these issues.

Strategies involving physical works to address the impact of increased runoff from development have been described in Sections 7 to 9. These strategies have considered works on public land and have also considered works on privately owned land that could be required as a part of the development process.

The ability to cater for the increased quantity of runoff that will generated by the predicted levels of development is constrained by the existing built-up nature of the catchment and by the costs that would be involved in upgrading infrastructure across the catchment (nominally a doubling in size of the existing underground systems). Much of this infrastructure has a significant remaining life and would not be due for replacement in the normal course of events for at least another 40 to 50 years, based on a 100 year asset life. The built up nature of the catchment has also removed much of the open space that could have been utilised to provide regional scale detention or retention systems to mitigate the impacts of this change.

As a result of the above, implementation of works on privately owned land and careful control of the manner in which re-development is undertaken form a key part of the ongoing management of stormwater in the catchment. Both of these items will need to be driven through the Development Planning process and will require some changes to the Development Plans of both the City of Marion and City of Holdfast Bay to reinforce existing provisions in both Plans that are focussed on stormwater management from new development.

The key issues that need to be addressed are:

- Form of Development
- Site Coverage Requirements
- Performance Requirements Governing Discharge Peaks and Volumes
- Flood Protection Requirements

Each of these issues is addressed by the Strategies outlined below.

## 11.2 Strategy 5.1

**The Cities of Marion and Holdfast Bay will work co-operatively to promote a development form in which higher density precincts are established with stormwater management infrastructure in planned open space**

The 30 year Plan for Greater Adelaide envisages a significant increase in the number of dwellings within the catchment. As discussed in Section 2.4 while the Plan describes a focus for this development around transport routes and within high density Transit Oriented Developments (TODs) the current Development Plans for both Councils do not currently reflect this vision.

From a stormwater management perspective, a future development scenario based on piecemeal subdivision of existing allotments to achieve the target increases in number of dwellings within the catchment is the least desirable outcome for a number of reasons as follows:

- The impacts of re-development are spread throughout the catchment and this does not lend itself to a strategy whereby investment in public infrastructure is able to occur to cater for this development in an efficient and planned manner.

- The form of development on the subdivided allotments provides little pervious area and limited space for the construction of on-site measures to manage runoff, specifically retention of runoff.
- Allotment scale redevelopment leads to a storm water management strategy which relies on numerous on-site devices. This will produce an impost on local government in relation to monitoring their correct installation and ongoing function.

By contrast, a future development scenario in which development is focussed within a number of Transit Oriented Developments or within defined higher density development precincts (with no residential subdivision elsewhere) would provide the most desirable outcome due to:

- The ability to properly plan for the impact of increased runoff from the development and to incorporate storm water management infrastructure within planned open space in these developments.
- The likelihood that such developments will have a low impervious area fraction due to the need to incorporate areas of open space to cater for the higher housing densities.
- The ability to provide a management strategy which relies on larger devices within public land, leading to a lower ongoing maintenance impost on local government.

A dialog needs to be commenced within both Councils and with State Government that focuses on the desirability or otherwise of future development only being in the form of TODs or within planned precincts. The most desirable outcome from a storm water management perspective, would be for re-zoning of specific precincts in the catchment for high density development and for subdivision of existing residential allotments in the remainder of the area to become non-complying development.

## 11.3 Strategy 5.2

**The Cities of Marion and Holdfast Bay will work co-operatively on changes to their respective Development Plans focussing on aligning stormwater provisions and reinforcing the objectives and strategies of this Stormwater Management Plan**

### 11.3.1 Site Coverage

The design of underground drainage systems serving the catchment was undertaken as part of the South Western Suburbs Drainage Scheme. Reference to calculations undertaken for the design indicate that a runoff coefficient of 0.33 (33% effective imperviousness) was assumed for residential areas within the catchment.

The Development Plans of both Councils contain provisions related to site coverage for residential areas, which are the predominant land use. The City of Holdfast Bay Plan indicates permissible site coverages (which exclude paving) in the range of 40 to 55% of residential site area. The City of Marion Plan indicates an allowable Site Coverage of up to 80% (refer comment below) including driveways and car parking areas.

Clearly, the current Development Plans are driving towards an ultimate imperviousness of the catchment that is well above that which was assumed in the design on the drainage system serving the area. While it is beyond the scope of this Plan to address the planning implications associated with adopting a lower site coverage, clearly there is a mismatch between the currently allowable levels of development and the capacity of the existing stormwater systems to cater for this development.

The City of Marion Development Plan currently contains a Site Coverage requirement for Residential Development. The wording of Item 26 under this provision is incorrect. The word impervious should be changed to pervious within this section.

### 11.3.2 Performance Requirements Governing Discharge Peaks and Volumes

Work carried out for the development of this Stormwater Management Plan has recommended an outcome whereby the first 15 mm of rainfall within each 24 hour period is retained on site for properties that are redeveloped. This retention may occur either by infiltration or by on site use.

Testing carried out as part of the preparation of this Plan has indicated that soils across much of the catchment may not have sufficient permeability to allow such a requirement to be met by infiltration. It is therefore proposed that where testing of site soils shows that disposal by infiltration is not possible, that discharge of water from the trench to the street water table be permitted provided that this discharge occurs over a period of at least 2 hours and no greater than 24 hours.

Neither the City of Marion nor City of Holdfast Bay Development Plans contains such a provision, although both Plans promote the retention and use of water within new developments.

The City of Marion Plan contains provisions under Item 29 of the 'Stormwater' subheading of 'Natural Resources' which govern the discharge of stormwater from sites involving new buildings or building extensions greater than 40 square metres in size. These provisions should be replaced with wording similar to the following:

*On land north of Seacombe Road, all new buildings and building extensions of 20 square metres or more in floor area, shall incorporate on-site stormwater retention systems which ensure that the first 15 mm of rainfall within any 24 hour period is retained on site. Where such retention systems rely on the use of infiltration, and testing shows that site soils will not permit infiltration of retained stormwater within a 24 hour period, provision of additional storage shall be provided either within an infiltration trench or tank which has sufficient capacity to contain runoff from 15 mm of rainfall and discharges over a period of at least 2 hours and no greater than 24 hours.*

The City of Holdfast Bay contains provisions and descriptions of design techniques for management of site storm water runoff from development within various zones. These provisions and descriptions will require modification within each of the zones to align with the requirements set out above.

### 11.3.3 Flood Protection Requirements

As a part of this project, 1 in 100 year ARI floodplain maps have been developed for the catchment. These maps show the properties that are likely to be affected by flooding and extent and depth of this flooding in a 1 in 100 year ARI event.

Section 6.3.2 contains objectives for the protection of new development from flooding. These objectives require that new development has adequate freeboard above the 1 in 100 year flood level and that it does not impede the flow of floodwaters thereby impacting flood levels on adjacent properties.

The current development plans of both Councils contain provisions relating to flood protection.

The City of Marion Plan contains these requirements within the Section under 'Hazards'. The wording of this Section appropriately addresses the requirements set out above.

The City of Holdfast Bay Plan addresses flood protection in a number of places. The provisions set out in the Plan have differing requirements between zones and are not addressed in a number of the zones. There is a need to amalgamate the flood protection requirements to be under a Council wide policy using wording similar to that used in the Marion Development Plan.

# 12 Description of Strategies – Management

## 12.1 Strategy 6.1

**Development of a strategy that sets out the timeframe for achieving the objectives set out in the Stormwater Management Plan and which integrates all elements of Council business to ensure that WSUD initiatives are embedded in all Council activities.**

The Stormwater Management Plan sets out a direction for the sustainable management of stormwater from the catchment. This strategy is largely based on implementation of WSUD. It is important that this philosophy flows through to Council activities and that WSUD is considered in works that both Councils undertake within the catchment.

## 12.2 Strategy 6.2

**Implementation of an Asset Management Plan for Stormwater that ensures that the objectives of the Stormwater Management Plan are achieved.**

Each of the Councils actively undertakes planning for management of their assets in a sustainable manner, including stormwater infrastructure. It is proposed that the current processes be continued.

## 12.3 Strategy 6.3

**The Cities of Holdfast Bay and Marion will work cooperatively on the development and implementation of stormwater management objectives to ensure a 'whole of catchment' approach is achieved.**

A number of the strategies described in the Plan will require an ongoing process of consultation and liaison between both Councils to either undertake further investigations or in the implementation of works.

## 12.4 Strategy 6.4

**Implement an ongoing monitoring program to measure catchment flows as a means of assessing the impact of proposed strategies for management of runoff from redevelopment**

There are currently no operational flow gauges within the catchments covered by this plan.

A rainfall and flow monitoring station was set up within the catchment of Drain 18 at Glenelg in the early 1990's. This station provided valuable data on the response of urban catchments in Adelaide to rainfall and information collected from this gauge was fundamental in changing parameters used in the modelling of runoff from Adelaide urban catchments. This station was closed due to lack of funding.

Hydrological modelling undertaken for this investigation has been based an assessment of model parameters based on experience with other catchments in Adelaide, but was not able to be calibrated due to a lack of current data for this catchment.

It is proposed that this gauge be reopened and rainfall, peak flow and volume data be collected. This data would be extremely valuable as a means of verifying the modelling process and to assess the impacts that may have already occurred as a result of changes to the catchment since the 1990's. Such data could also be used to verify the effectiveness of the strategies proposed as part of this plan if monitoring is continued over a period of time.

Establishment of an additional rainfall and flow gauging station on one of the major catchments discharging to the coast would be desirable to provide additional data for a second catchment in the study area. It is proposed that this catchment be the Edward Street drain catchment which contains substantial areas of residential land which is likely to be subject to ongoing development.

# 13 Costs, Benefits and Funding Arrangements

## 13.1 Cost Estimates

### 13.1.1 Major Drainage Outfalls

Cost estimates are provided in Table 13.1 for construction of the major drainage outfalls. These estimates are based on a preliminary design assessment of these outfalls carried out for the City of Holdfast Bay (Tonkin, 2013). These estimates are subject to detailed design of the drains and will need to be confirmed once this design work is carried out.

*Table 13.1 Major Drainage Outalls – Cost Estimates*

| Location | Responsible Council | Cost Estimate ($2014) |
|---|---|---|
| Tarlton Street Outfall | Holdfast Bay | $4,800,000 |
| Jetty Road Outfall | Holdfast Bay/Marion | $6,900,000 |
| Edward Street Outfall | Holdfast Bay/Marion | $6,800,000 |
| Minda / Harrow Road Outfall | Holdfast Bay/Marion | $3,300,000 |

As these works serve a catchment that is well in excess of 40 ha, funding for up to 50% of the investigation, design and construction of the various drains may be available through the Stormwater Management Authority. The remaining funding will need to be sourced from the catchment councils under a cost sharing arrangement (see Section 13.2).

### 13.1.2 Minor Drainage System Extensions

Cost estimates are provided in Table 13.2 below for extension of the minor drainage system. These estimates have been based on the layouts shown in Figure 7.3 and on the assumption that the drains have an average diameter of 450 mm. The estimates include an allowance for service alterations (10% of the construction cost) and for contingencies (20% of the construction cost).

Table 13.2 also identifies the responsible Council. Most of the drains and their associated catchments lie entirely within either the City of Holdfast Bay or the City of Marion. The only exceptions to this are the Whiteleaf Crescent drain (Location 2) and the Byre Avenue drain (Catchment 16). Cost apportionment for these drains will need to be determined in accordance with the principles set out in Section 13.2.3.

*Table 13.2 Minor Drainage System Extension – Cost Estimates*

| Location | Responsible Council | Cost estimate ($2014) |
|---|---|---|
| 1. Francis Crescent, Glengowrie | Marion | $270,000 |
| 2. Whiteleaf Crescent, Glengowrie | Marion/Holdfast Bay | $520,000 |
| 3. Bombay Street, Oaklands Park | Marion | $310,000 |
| 4. Soho Street, Warradale | Marion | $350,000 |
| 5. Crozier Terrace, Oaklands Park | Marion | $630,000 |
| 6. Dwyer Road, Oaklands Park | Marion | $390,000 |
| 7. Township Road, Marion | Marion | $390,000 |
| 8. Grandview Grove, Sturt | Marion | $ 380,000 |
| 9. Travers Street, Sturt | Marion | $490,000 |
| 10. Glamis Avenue, Seacombe Gardens | Marion | $380,000 |

| Location | Responsible Council | Cost estimate ($2014) |
|---|---|---|
| 11. Wilga Street, Seacombe Gardens | Marion | $270,000 |
| 12. Laurence Street, Dover Gardens | Marion | $370,000 |
| 13. Quintus Terrace, Dover Gardens | Marion | $700,000 |
| 14. Walkers Road, Somerton Park | Holdfast Bay | $590,000 |
| 15. Moore Street, Somerton Park | Holdfast Bay | $260,000 |
| 16. College Road, Somerton Park | Holdfast Bay | $520,000 |
| 17. Byre Avenue, Somerton Park | Marion/Holdfast Bay | $620,000 |
| 18. Cecelia Street, North Brighton | Holdfast Bay | $480,000 |
| 19. Caroona Street, Hove | Holdfast Bay | $310,000 |
| 20. Alfreda Street, Brighton | Holdfast Bay | $430,000 |
| 21. McCoy Street, Brighton | Holdfast Bay | $290,000 |
| 22. Rudford Street, Brighton | Holdfast Bay | $270,000 |
| 23. Yarmouth Street, South Brighton | Holdfast Bay | $390,000 |
| 24. High Street, South Brighton | Holdfast Bay | $320,000 |
| 25. Ophir Crescent, Seacliff Park | Holdfast Bay | $420,000 |
| 26. Wheatland Street, Seacliff | Holdfast Bay | $300,000 |
| 27. Walsh Street, Hove | Holdfast Bay | $700,000[1] |
| **Total** | | **$11,350,000** |

Note 1: The cost for the Walsh Street Drain has been taken from a separate investigation and is for providing a 1 in 20 year ARI design standard.

Responsibility for funding these works will rest with the Council in which they are located. Where the works lie within two Councils, costs will be apportioned using a cost sharing arrangement as described in Section 13.2.

The catchment areas served by each of the individual projects described above do not exceed 40 ha and as a result none of the above works are eligible for part funding through the Stormwater Management Authority.

### 13.1.3 Gross Pollutant Traps

Indicative construction costs for GPTs described in Section 8.4 are provided below.

*Table 13.3 Gross Pollutant Trap Costs*

| Location | Responsible Council | Cost ($2014) |
|---|---|---|
| The Broadway | Holdfast Bay | $190,000 |
| Harrow Road | Holdfast Bay/Marion | $380,000 |
| Wattle Avenue | Holdfast Bay/Marion | $370,000 |
| Pine Avenue | Holdfast Bay/Marion | $170,000 |
| Marine Street | Holdfast Bay | $175,000 |

Part funding for investigation, design and construction of these gross pollutant traps is available through the NRM Board. Responsibility for meeting the remaining cost will rest with the catchment Councils under a cost sharing arrangement as described in Section 13.2.

#### 13.1.4 Flow Monitoring

The indicative cost for construction of each proposed rainfall and flow gauging station is likely to be in the order of $20,000 to $30,000, depending on their final location. Ongoing annual maintenance costs are likely to be of the order of $10,000 per station.

The costs for establishing and operating the proposed stations should be shared between each Council. It is possible that part funding for construction may be available from the NRM Board or Stormwater Management Authority as the data these stations provide will have a wider community benefit.

### 13.2 Cost Apportionment Between the Councils

#### 13.2.1 Background

In 2004, KBR undertook a study for the Local Government Association of South Australia and the State Government of South Australia entitled Metropolitan Adelaide Stormwater Management Study (KBR, July 2004). Part C of the report prepared for that study dealt with the issue of apportionment of Council costs and explored a number of options for allocating the cost of stormwater infrastructure where the catchments extend across more than one Council.

The report includes a comprehensive discussion about the complexity of the factors that could be taken account of in the determination of equitable cost apportionment between Councils for stormwater infrastructure. Fundamentally, the report concludes that all areas that contribute stormwater as a result of urbanisation, bear some responsibility for the cost of the stormwater infrastructure required to convey that stormwater safely to the sea, but it also recognises the benefits that reducing flood risk in downstream areas has on the ability of those areas to allow urbanisation.

The report considers three cost apportionment models being:

- Option 1: A simple model where the costs are apportioned simply on the basis of contribution to flows measured in terms of impervious areas within each Council area. This option is suitable where the costs and the benefits are relatively uniformly distributed across the catchment, but is less attractive when most of the benefits fall in one area only.
- Option 2: A more complex model that attributes a part of the cost (x%) based on the proportion that each Council contributes to the flows, and attributes the balance (1-x%) based on proportion that each Council benefits from future costs avoided by the reduction in damage as a result of flood risk reduction.
- Option 3: A yet more complex model that builds on Option 2 by introducing a third factor (y%) that takes account of the proportion of local benefits such as opportunities for water re-use, aesthetic and recreational outcomes that each Council enjoys as a result of infrastructure.

KBR's report does not make any recommendation in relation to the quantum of the percentages x or y.

The works proposed in this management plan fall into the following categories:

1. Upgrades to the major drainage system to drain trapped low spots behind the coastal dune system.
2. Upgrades to the minor drainage system where overland flow paths are long and/or sub-catchments are too large for the existing drainage inlets.
3. WSUD measures on private developments.
4. WSUD measures on Council roads and open spaces.
5. Water quality improvement devices (GPTs) on coastal outfalls.

### 13.2.2 Cost Share for Major Drainage System Upgrades

The upgrades proposed for the major drainage system are exclusively to reduce the flood risk to properties in trapped low points behind the coastal dune system. The benefits are limited to flood risk reduction at those locations. It is proposed that the Cities of Holdfast Bay and Marion will apportion costs for upgrades to the major drainage system in accordance with Option 2 set out previously with a 50/50 weighting applied to contribution/benefit. As an example, if 60% of the stormwater was sourced in the City of Marion, and 40% in the City of Holdfast Bay, and all the benefit accrued in the City of Holdfast Bay for a project with a combined Council contribution of $C, the contributions from the Councils would be calculated as follows:

*Contribution by the City of Marion = (0.5 x 0.6 + 0.5 x 0.0) x C = 0.3 x C*

*Contribution from the City of Holdfast Bay (0.5 x 0.4 + 0.5 x 1.0) x C = 0.7 x C*

Note: It is anticipated that the State Government may partially fund this project and therefore the combined Council contributions would be the cost of the project less any other contributions.

Given that the two Council areas are relatively homogenous and that they have similar development policies in place, it is proposed that the calculation of contribution to flows be based on impervious areas calculated with reference to current zoning provisions in relation to minimum allotment sizes but not including land zoned as open space. This avoids the need to actually quantify the current imperviousness and anticipates a fully developed catchment.

### 13.2.3 Cost Share for Minor Drainage System Upgrades

In principle, the methodology set out above for major drainage system upgrades should be applied for all drainage upgrades, however in the case of upgrades to the minor system, it is more likely that the costs and the benefits will both occur in one or other of the Council areas.

### 13.2.4 Cost Share for WSUD Measures on Private Developments

It is proposed that WSUD measures on private property be funded by the developers of private property without any Council contribution.

### 13.2.5 Cost Share for WSUD Measures on Council Roads and Open Spaces

It is proposed that the cost of the Council contributions to WSUD measures on Council roads and open spaces be funded in proportion to the contribution of flows to the device. The logic for this is that the benefits of improved stormwater quality are shared across the broader community, and that the burden for implementing WSUD should similarly be shared in proportion to contribution. This is consistent with the cost share set out for major drainage system, but simply apportions benefits in the same proportion as contributions. As for the minor drainage system, it is considered that in most cases the costs and the benefits will both occur in Council areas only.

### 13.2.6 Cost Share for Water Quality Improvement Devices (GPTs) on Coastal Outfalls

As for WSUD measures, more generally it is considered that the benefits of improved water quality on the Adelaide beaches are a benefit to the broader community and costs ought to be shared in proportion to contribution.

### 13.2.7 Cost Share for Flow Monitoring

The data derived from the flow monitoring stations will be of benefit to both Councils. The costs ought to be shared in proportion to the area of each Council in the overall area covered by this Plan.

# 14 Priorities and Timeframes

Implementation of the works described in this Plan is subject to the availability of funding from the various Councils, the State Government through the NRM Board and Stormwater Management Authority and other funding partners that may be found.

For the purposes of setting priorities, the actions have been divided into short term (nominally within the next two years) medium term (2 to 5 years) and long term (up to 10 years) actions.

## 14.1 Short Term Actions (0 to 2 years)

The following actions have been identified as being of the highest priority (in no particular order):

- Prepare a stormwater management plan for the Linwood / Lorenzin / Pine Gully catchment
- Establish rainfall and flow monitoring stations to measure changes in the catchment characteristics and climate change
- Establish a joint Council Working Group to:
  - Develop catchment proposals to integrate WSUD into drainage designs, streetscape, public open space and development
  - Review stormwater management provisions within current Development Plans
  - Commence discussions in relation to TODs and development zones with State Government agencies
  - Integrate planning of development, public open space, infrastructure and stormwater management
  - Investigate feasibility of proposed major outfalls to reduce hazardous flooding
  - Formalise joint Council agreements for capital works undertakings
  - Investigate stormwater design solutions for WSUD, reuse and retention/detention and prepare guidelines for development
- Complete current program of lateral drainage system upgrades within the City of Marion
- Each Council to include the Stormwater Management Plan in the review of their Long Term Financial Plan, Strategic Plan and Infrastructure and Asset Management Plan and upgrade these Plans accordingly.

## 14.2 Medium Term Actions (2 to 5 years)

The following actions have been identified as being of medium term priority (in no particular order):

- Prepare a stormwater management plan for the Gilbertson Gully Catchment
- Review this Stormwater Management Plan and update accordingly
- Implement WSUD proposals
- Implement changes to the Development Plan
- Construct first of the four major outfalls (subject to feasibility and funding)
- Commence program of gross pollutant trap construction (subject to funding)
- Commence program of lateral drainage upgrades (subject to funding)

  Within the City of Marion, the highest priority catchments are considered to be:

  - Wilga Street, Seacombe Gardens

- Crozier Terrace, Oaklands Park
- Dwyer Road Oaklands Park
- Whiteleaf Crescent, Glengowrie
- Francis Street, Glengowrie

Within the City of Holdfast Bay, the highest priority catchments are considered to be:

- Walkers Road, Somerton Park
- Moore Street, Somerton Park
- Ceclia Street, North Brighton
- Walsh Street, Hove

## 14.3 Long Term Actions (5 to 10+ years)

The following actions have been identified as being of longer term priority (in no particular order):

- Construct remaining major outfalls (subject to feasibility and funding)
- Complete program of gross pollutant trap construction (subject to funding)
- Continue the program of lateral drainage system upgrades (most likely extending past the 10 year time horizon)
- Continue the introduction of WSUD proposals

# 15 Responsibilities

Responsibilities for implementation of the various strategies set out in this plan are set out below. These responsibilities have been described under the following categories:

- Capital works
- Planning
- Investigations
- Management

## 15.1 Capital Works

Proposed capital works to be implemented within the catchment are described Sections 7 and 8. Costs associated with these works are provided in Section 13 which also describes the responsibilities for implementing these works.

Broadly, lead responsibility for construction of these works has been allocated to the Council within which the works are to be undertaken.

## 15.2 Planning

Actions relating to planning that are described in this document include:

- Changes to provisions within the current Development Plans of both Councils to address the quantity of stormwater discharged from new development
- A more fundamental change to zoning within each Council to concentrate development within planned zones rather than across the catchment.

Responsibility for making changes to the current Development Plans rests with each Council and it is suggested that this occur as part of the next review of each plan.

The proposed changes to zoning are more complex and are likely to require liaison and consultation with State Government. It is suggested that a working party comprising appropriate personnel from each Council is created, to form a view as to the merits of such an approach, to identify barriers and to progress these discussions if considered appropriate.

## 15.3 Investigations

A number of investigations are proposed to advance various strategies described in this plan. Responsibilities for undertaking these investigations are set out in Table 15.1 below.

***Table 15.1 Responsibility for Further Investigations***

| Investigation | Lead Organisation Responsible | Related Organisation Responsible |
|---|---|---|
| Major Outfalls | City of Holdfast Bay | City of Marion, SMA, NRM Board |
| Linwood/Lorenzin/Pine Gully | City of Holdfast/City of Marion | NRM Board |
| Gilbertson Gully | City of Holdfast Bay | City of Marion, NRM Board |
| Minda Homes Development | Developer | City of Holdfast Bay |

## 15.4 Management

It is expected that each Council will take responsibility for management of the stormwater system within its area.

The establishment and operation of flow monitoring stations should be a joint responsibility of both Councils as the benefit derived from any data collected will be valuable to each organisation.

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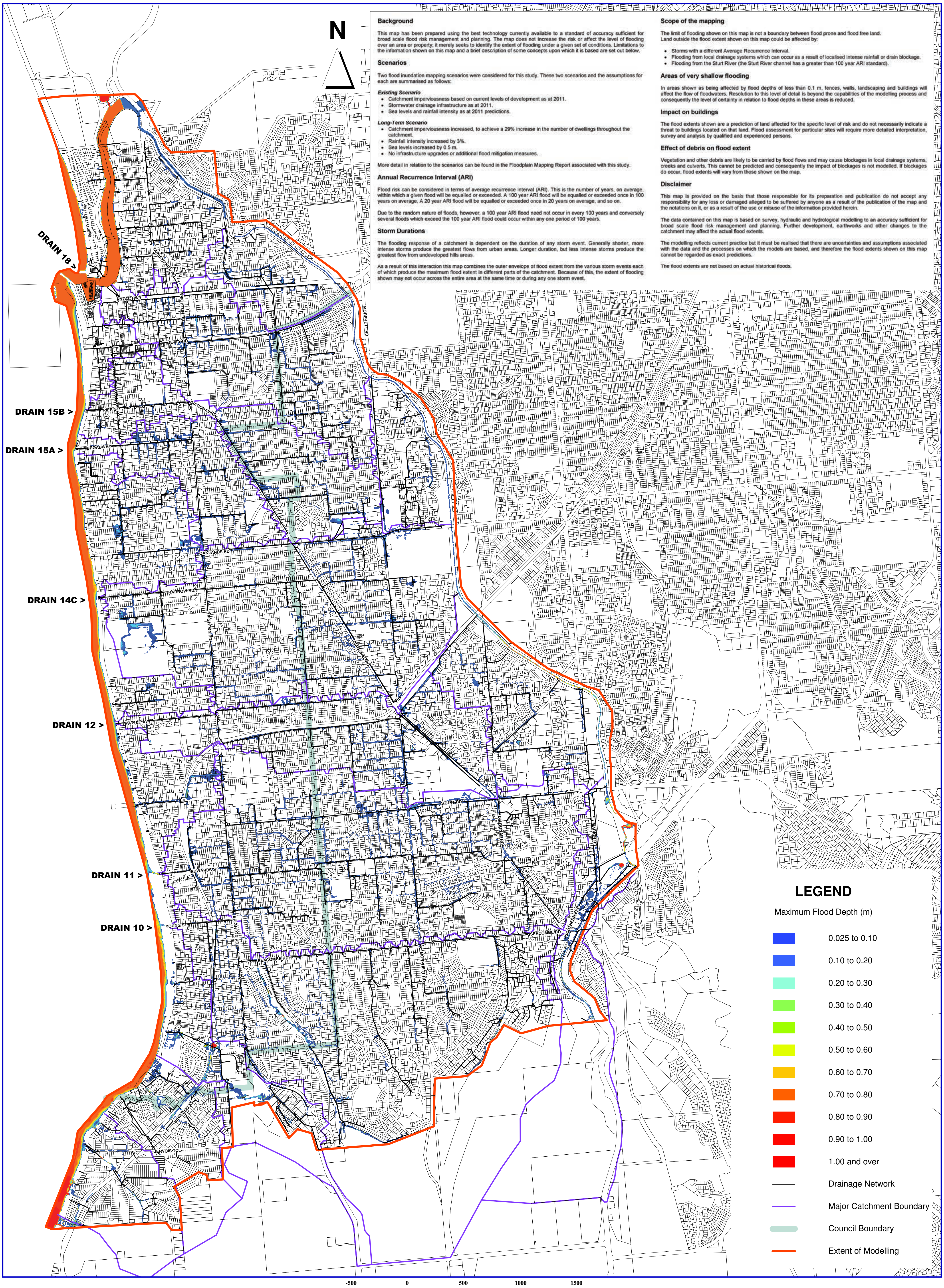
# Appendix A

# 5 Year and 100 Year Inundation Maps for the Existing Scenario




## Background

This map has been prepared using the best technology currently available to a standard of accuracy sufficient for broad scale flood risk management and planning. The map does not increase the risk or affect the level of flooding over an area or property; it merely seeks to identify the extent of flooding under a given set of conditions. Limitations to the information shown on this map and a brief description of some concepts upon which it is based are set out below.

## Scenarios

Two flood inundation mapping scenarios were considered for this study. These two scenarios and the assumptions for each are summarised as follows:

***Existing Scenario***

- Catchment imperviousness based on current levels of development as at 2011.
- Stormwater drainage infrastructure as at 2011.
- Sea levels and rainfall intensity as at 2011 predictions.

***Long-Term Scenario***

- Catchment imperviousness increased, to achieve a 29% increase in the number of dwellings throughout the catchment.
- Rainfall intensity increased by 3%.
- Sea levels increased by 0.5 m.
- No infrastructure upgrades or additional flood mitigation measures.

More detail in relation to the scenarios can be found in the Floodplain Mapping Report associated with this study.

## Annual Recurrence Interval (ARI)

Flood risk can be considered in terms of average recurrence interval (ARI). This is the number of years, on average, within which a given flood will be equalled or exceeded. A 100 year ARI flood will be equalled or exceeded once in 100 years on average. A 20 year ARI flood will be equalled or exceeded once in 20 years on average, and so on.

Due to the random nature of floods, however, a 100 year ARI flood need not occur in every 100 years and conversely several floods which exceed the 100 year ARI flood could occur within any one period of 100 years.

## Storm Durations

The flooding response of a catchment is dependent on the duration of any storm event. Generally shorter, more intense storms produce the greatest flows from urban areas. Longer duration, but less intense storms produce the greatest flow from undeveloped hills areas.

As a result of this interaction this map combines the outer envelope of flood extent from the various storm events each of which produce the maximum flood extent in different parts of the catchment. Because of this, the extent of flooding shown may not occur across the entire area at the same time or during any one storm event.

## Scope of the mapping

The limit of flooding shown on this map is not a boundary between flood prone and flood free land.
Land outside the flood extent shown on this map could be affected by:

- Storms with a different Average Recurrence Interval.
- Flooding from local drainage systems which can occur as a result of localised intense rainfall or drain blockage.
- Flooding from the Sturt River (the Sturt River channel has a greater than 100 year ARI standard).

## Areas of very shallow flooding

In areas shown as being affected by flood depths of less than 0.1 m, fences, walls, landscaping and buildings will affect the flow of floodwaters. Resolution to this level of detail is beyond the capabilities of the modelling process and consequently the level of certainty in relation to flood depths in these areas is reduced.

## Impact on buildings

The flood extents shown are a prediction of land affected for the specific level of risk and do not necessarily indicate a threat to buildings located on that land. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

## Effect of debris on flood extent

Vegetation and other debris are likely to be carried by flood flows and may cause blockages in local drainage systems, creeks and culverts. This cannot be predicted and consequently the impact of blockages is not modelled. If blockages do occur, flood extents will vary from those shown on the map.

## Disclaimer

This map is provided on the basis that those responsible for its preparation and publication do not accept any responsibility for any loss or damaged alleged to be suffered by anyone as a result of the publication of the map and the notations on it, or as a result of the use or misuse of the information provided herein.

The data contained on this map is based on survey, hydraulic and hydrological modelling to an accuracy sufficient for broad scale flood risk management and planning. Further development, earthworks and other changes to the catchment may affect the actual flood extents.

The modelling reflects current practice but it must be realised that there are uncertainties and assumptions associated with the data and the processes on which the models are based, and therefore the flood extents shown on this map cannot be regarded as exact predictions.

The flood extents are not based on actual historical floods.



**Scale: 1:15,000**

MAP DETAILS

| | |
|---|---|
| Projection : | Transverse Mercator, GDA 94<br>MGA, Zone 54 |
| Elevation : | Australian Height Datum (metres) |
| Cadastre : | PBBI Australia |
| Drainage Data : | Tonkin Consulting |
| Flood Mapping : | Tonkin Consulting |
| Job Number : | 2010.0878 |
| Filename : | MH_5yrARI_Flood_Inundation_A1.wor |
| Drawn : | Tim Cresswell |
| Date : | 29/11/2011 |

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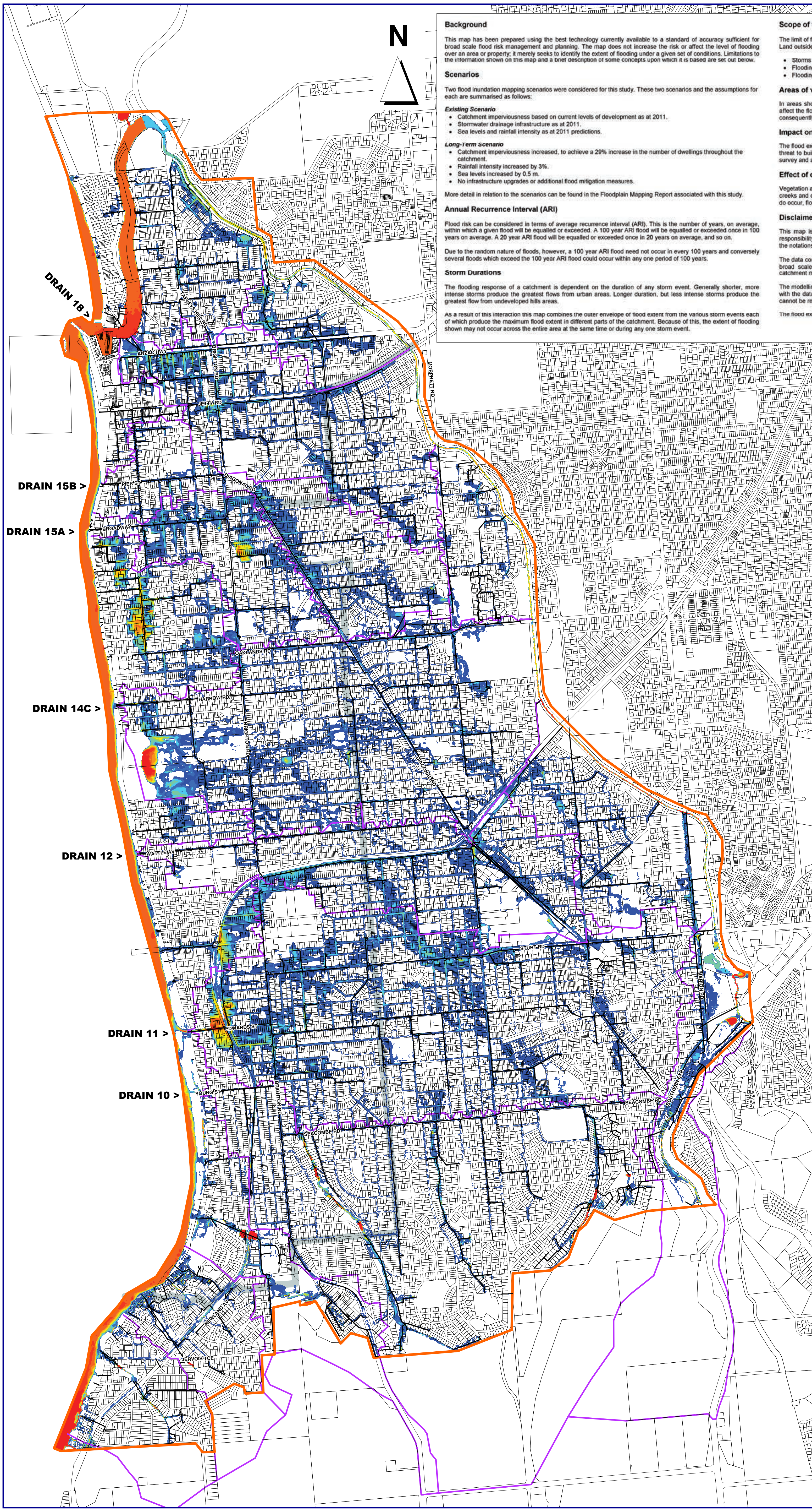
*City of Marion and City of Holdfast Bay*

5 YEAR ARI FLOOD INUNDATION MAP

EXISTING SCENARIO

## MARION & HOLDFAST BAY




## Background

This map has been prepared using the best technology currently available to a standard of accuracy sufficient for broad scale flood risk management and planning. The map does not increase the risk or affect the level of flooding over an area or property; it merely seeks to identify the extent of flooding under a given set of conditions. Limitations to the information shown on this map and a brief description of some concepts upon which it is based are set out below.

## Scenarios

Two flood inundation mapping scenarios were considered for this study. These two scenarios and the assumptions for each are summarised as follows:

***Existing Scenario***

- Catchment imperviousness based on current levels of development as at 2011.
- Stormwater drainage infrastructure as at 2011.
- Sea levels and rainfall intensity as at 2011 predictions.

***Long-Term Scenario***

- Catchment imperviousness increased, to achieve a 29% increase in the number of dwellings throughout the catchment.
- Rainfall intensity increased by 3%.
- Sea levels increased by 0.5 m.
- No infrastructure upgrades or additional flood mitigation measures.

More detail in relation to the scenarios can be found in the Floodplain Mapping Report associated with this study.

## Annual Recurrence Interval (ARI)

Flood risk can be considered in terms of average recurrence interval (ARI). This is the number of years, on average, within which a given flood will be equalled or exceeded. A 100 year ARI flood will be equalled or exceeded once in 100 years on average. A 20 year ARI flood will be equalled or exceeded once in 20 years on average, and so on.

Due to the random nature of floods, however, a 100 year ARI flood need not occur in every 100 years and conversely several floods which exceed the 100 year ARI flood could occur within any one period of 100 years.

## Storm Durations

The flooding response of a catchment is dependent on the duration of any storm event. Generally shorter, more intense storms produce the greatest flows from urban areas. Longer duration, but less intense storms produce the greatest flow from undeveloped hills areas.

As a result of this interaction this map combines the outer envelope of flood extent from the various storm events each of which produce the maximum flood extent in different parts of the catchment. Because of this, the extent of flooding shown may not occur across the entire area at the same time or during any one storm event.

## Scope of the mapping

The limit of flooding shown on this map is not a boundary between flood prone and flood free land.
Land outside the flood extent shown on this map could be affected by:

- Storms with a different Average Recurrence Interval.
- Flooding from local drainage systems which can occur as a result of localised intense rainfall or drain blockage.
- Flooding from the Sturt River (the Sturt River channel has a greater than 100 year ARI standard).

## Areas of very shallow flooding

In areas shown as being affected by flood depths of less than 0.1 m, fences, walls, landscaping and buildings will affect the flow of floodwaters. Resolution to this level of detail is beyond the capabilities of the modelling process and consequently the level of certainty in relation to flood depths in these areas is reduced.

## Impact on buildings

The flood extents shown are a prediction of land affected for the specific level of risk and do not necessarily indicate a threat to buildings located on that land. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

## Effect of debris on flood extent

Vegetation and other debris are likely to be carried by flood flows and may cause blockages in local drainage systems, creeks and culverts. This cannot be predicted and consequently the impact of blockages is not modelled. If blockages do occur, flood extents will vary from those shown on the map.

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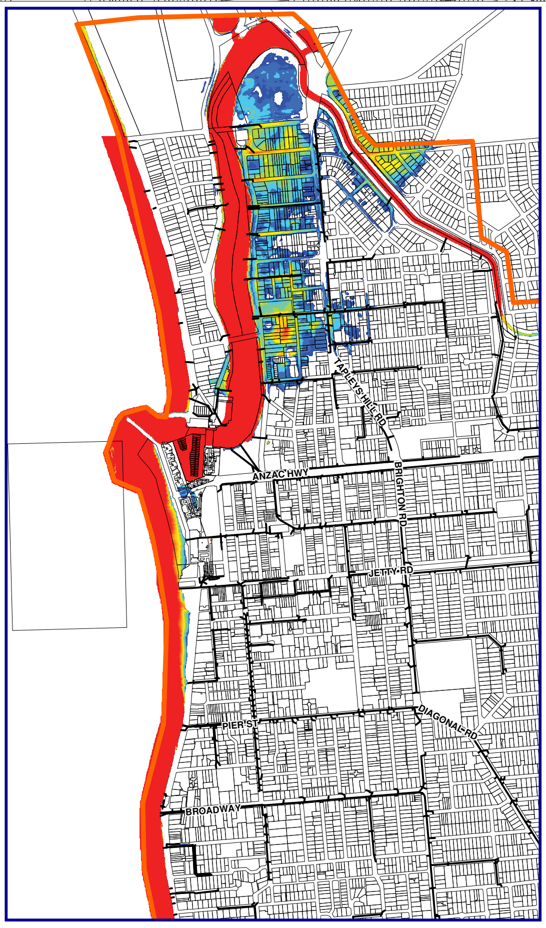
The data contained on this map is based on survey, hydraulic and hydrological modelling to an accuracy sufficient for broad scale flood risk management and planning. Further development, earthworks and other changes to the catchment may affect the actual flood extents.

The modelling reflects current practice but it must be realised that there are uncertainties and assumptions associated with the data and the processes on which the models are based, and therefore the flood extents shown on this map cannot be regarded as exact predictions.

The flood extents are not based on actual historical floods.




**100 YEAR ARI HIGH TIDE INUNDATION MAP**
**EXISTING SCENARIO**

This map represents the depth of inundation that would occur if a 100 year ARI tide (including storm surge) (EL 2.38 mAHD) were to inundate land surrounding the Patawalonga Lake. For this to occur, coincident with a 100 year ARI tide, either of the following would need to occur:

- Failure of the Glenelg Gates to hold back the high tide;
- Sufficient rainfall within the Brown Hill Creek or Sturt River catchments to cause the lake level to rise at the same rate as the tide.

Consequently the combined probability of this scenario occurring would be less likely than the 100 year ARI event.



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MAP DETAILS

Projection : Transverse Mercator, GDA 94
MGA, Zone 54
Elevation : Australian Height Datum (metres)
Cadastre : PBBI Australia

Drainage Data : Tonkin Consulting
Flood Mapping : Tonkin Consulting
Job Number : 2010.0878
Filename : MH_100yrARI_Flood_Inundation_A1.wor
Drawn : Tim Cresswell
Date : 29/11/2011

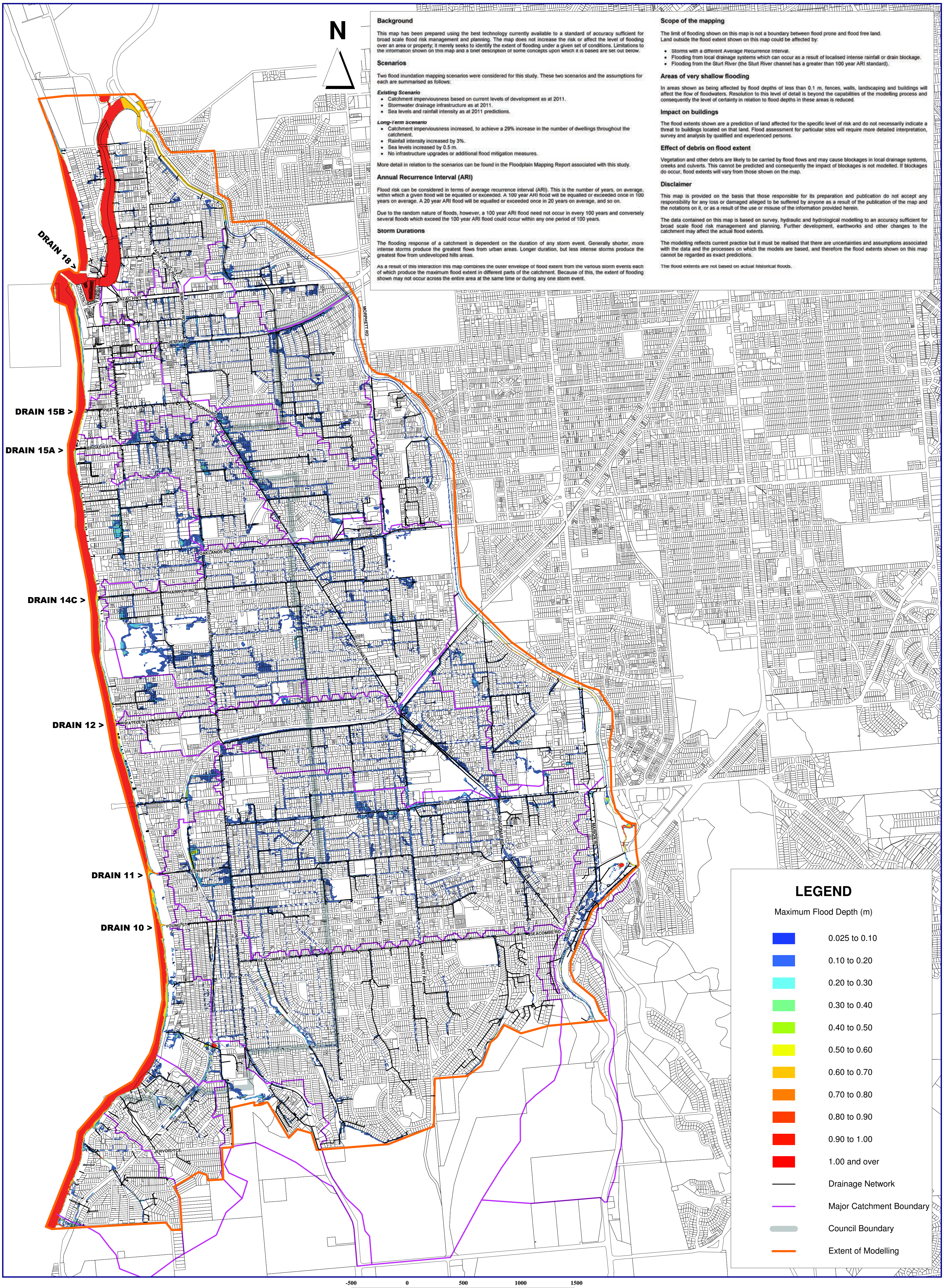
*City of Marion and City of Holdfast Bay*

**100 YEAR ARI FLOOD INUNDATION MAP**
**EXISTING SCENARIO**

# MARION & HOLDFAST BAY

# Appendix B

# 5 Year and 100 Year Inundation Maps for the Long-Term Scenario




## Background

This map has been prepared using the best technology currently available to a standard of accuracy sufficient for broad scale flood risk management and planning. The map does not increase the risk or affect the level of flooding over an area or property; it merely seeks to identify the extent of flooding under a given set of conditions. Limitations to the information shown on this map and a brief description of some concepts upon which it is based are set out below.

## Scenarios

Two flood inundation mapping scenarios were considered for this study. These two scenarios and the assumptions for each are summarised as follows:

***Existing Scenario***
- Catchment imperviousness based on current levels of development as at 2011.
- Stormwater drainage infrastructure as at 2011.
- Sea levels and rainfall intensity as at 2011 predictions.

***Long-Term Scenario***
- Catchment imperviousness increased, to achieve a 29% increase in the number of dwellings throughout the catchment.
- Rainfall intensity increased by 3%.
- Sea levels increased by 0.5 m.
- No infrastructure upgrades or additional flood mitigation measures.

More detail in relation to the scenarios can be found in the Floodplain Mapping Report associated with this study.

## Annual Recurrence Interval (ARI)

Flood risk can be considered in terms of average recurrence interval (ARI). This is the number of years, on average, within which a given flood will be equalled or exceeded. A 100 year ARI flood will be equalled or exceeded once in 100 years on average. A 20 year ARI flood will be equalled or exceeded once in 20 years on average, and so on.

Due to the random nature of floods, however, a 100 year ARI flood need not occur in every 100 years and conversely several floods which exceed the 100 year ARI flood could occur within any one period of 100 years.

## Storm Durations

The flooding response of a catchment is dependent on the duration of any storm event. Generally shorter, more intense storms produce the greatest flows from urban areas. Longer duration, but less intense storms produce the greatest flow from undeveloped hills areas.

As a result of this interaction this map combines the outer envelope of flood extent from the various storm events each of which produce the maximum flood extent in different parts of the catchment. Because of this, the extent of flooding shown may not occur across the entire area at the same time or during any one storm event.

## Scope of the mapping

The limit of flooding shown on this map is not a boundary between flood prone and flood free land.
Land outside the flood extent shown on this map could be affected by:

- Storms with a different Average Recurrence Interval.
- Flooding from local drainage systems which can occur as a result of localised intense rainfall or drain blockage.
- Flooding from the Sturt River (the Sturt River channel has a greater than 100 year ARI standard).

## Areas of very shallow flooding

In areas shown as being affected by flood depths of less than 0.1 m, fences, walls, landscaping and buildings will affect the flow of floodwaters. Resolution to this level of detail is beyond the capabilities of the modelling process and consequently the level of certainty in relation to flood depths in these areas is reduced.

## Impact on buildings

The flood extents shown are a prediction of land affected for the specific level of risk and do not necessarily indicate a threat to buildings located on that land. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

## Effect of debris on flood extent

Vegetation and other debris are likely to be carried by flood flows and may cause blockages in local drainage systems, creeks and culverts. This cannot be predicted and consequently the impact of blockages is not modelled. If blockages do occur, flood extents will vary from those shown on the map.

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The modelling reflects current practice but it must be realised that there are uncertainties and assumptions associated with the data and the processes on which the models are based, and therefore the flood extents shown on this map cannot be regarded as exact predictions.

The flood extents are not based on actual historical floods.



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**Scale: 1:15,000**

MAP DETAILS

Projection : Transverse Mercator, GDA 94
MGA, Zone 54
Elevation : Australian Height Datum (metres)
Cadastre : PBBI Australia

Drainage Data : Tonkin Consulting
Flood Mapping : Tonkin Consulting
Job Number : 2010.0878
Filename : MH_LT_5yrARI_Flood_Inundation_A1.wor
Drawn : Tim Cresswell
Date : 29/11/2011

*City of Marion and City of Holdfast Bay*

5 YEAR ARI FLOOD INUNDATION MAP

LONG-TERM SCENARIO

**MARION & HOLDFAST BAY**

## Background

This map has been prepared using the best technology currently available to a standard of accuracy sufficient for broad scale flood risk management and planning. The map does not increase the risk or affect the level of flooding over an area or property; it merely seeks to identify the extent of flooding under a given set of conditions. Limitations to the information shown on this map and a brief description of some concepts upon which it is based are set out below.

## Scenarios

Two flood inundation mapping scenarios were considered for this study. These two scenarios and the assumptions for each are summarised as follows:

*Existing Scenario*

- Catchment imperviousness based on current levels of development as at 2011.
- Stormwater drainage infrastructure as at 2011.
- Sea levels and rainfall intensity as at 2011 predictions.

*Long-Term Scenario*

- Catchment imperviousness increased, to achieve a 29% increase in the number of dwellings throughout the catchment.
- Rainfall intensity increased by 3%.
- Sea levels increased by 0.5 m.
- No infrastructure upgrades or additional flood mitigation measures.

More detail in relation to the scenarios can be found in the Floodplain Mapping Report associated with this study.

## Annual Recurrence Interval (ARI)

Flood risk can be considered in terms of average recurrence interval (ARI). This is the number of years, on average, within which a given flood will be equalled or exceeded. A 100 year ARI flood will be equalled or exceeded once in 100 years on average. A 20 year ARI flood will be equalled or exceeded once in 20 years on average, and so on.

Due to the random nature of floods, however, a 100 year ARI flood need not occur in every 100 years and conversely several floods which exceed the 100 year ARI flood could occur within any one period of 100 years.

## Storm Durations

The flooding response of a catchment is dependent on the duration of any storm event. Generally shorter, more intense storms produce the greatest flows from urban areas. Longer duration, but less intense storms produce the greatest flow from undeveloped hills areas.

As a result of this interaction this map combines the outer envelope of flood extent from the various storm events each of which produce the maximum flood extent in different parts of the catchment. Because of this, the extent of flooding shown may not occur across the entire area at the same time or during any one storm event.

## Scope of the mapping

The limit of flooding shown on this map is not a boundary between flood prone and flood free land.
Land outside the flood extent shown on this map could be affected by:

- Storms with a different Average Recurrence Interval.
- Flooding from local drainage systems which can occur as a result of localised intense rainfall or drain blockage.
- Flooding from the Sturt River (the Sturt River channel has a greater than 100 year ARI standard).

## Areas of very shallow flooding

In areas shown as being affected by flood depths of less than 0.1 m, fences, walls, landscaping and buildings will affect the flow of floodwaters. Resolution to this level of detail is beyond the capabilities of the modelling process and consequently the level of certainty in relation to flood depths in these areas is reduced.

## Impact on buildings

The flood extents shown are a prediction of land affected for the specific level of risk and do not necessarily indicate a threat to buildings located on that land. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

## Effect of debris on flood extent

Vegetation and other debris are likely to be carried by flood flows and may cause blockages in local drainage systems, creeks and culverts. This cannot be predicted and consequently the impact of blockages is not modelled. If blockages do occur, flood extents will vary from those shown on the map.

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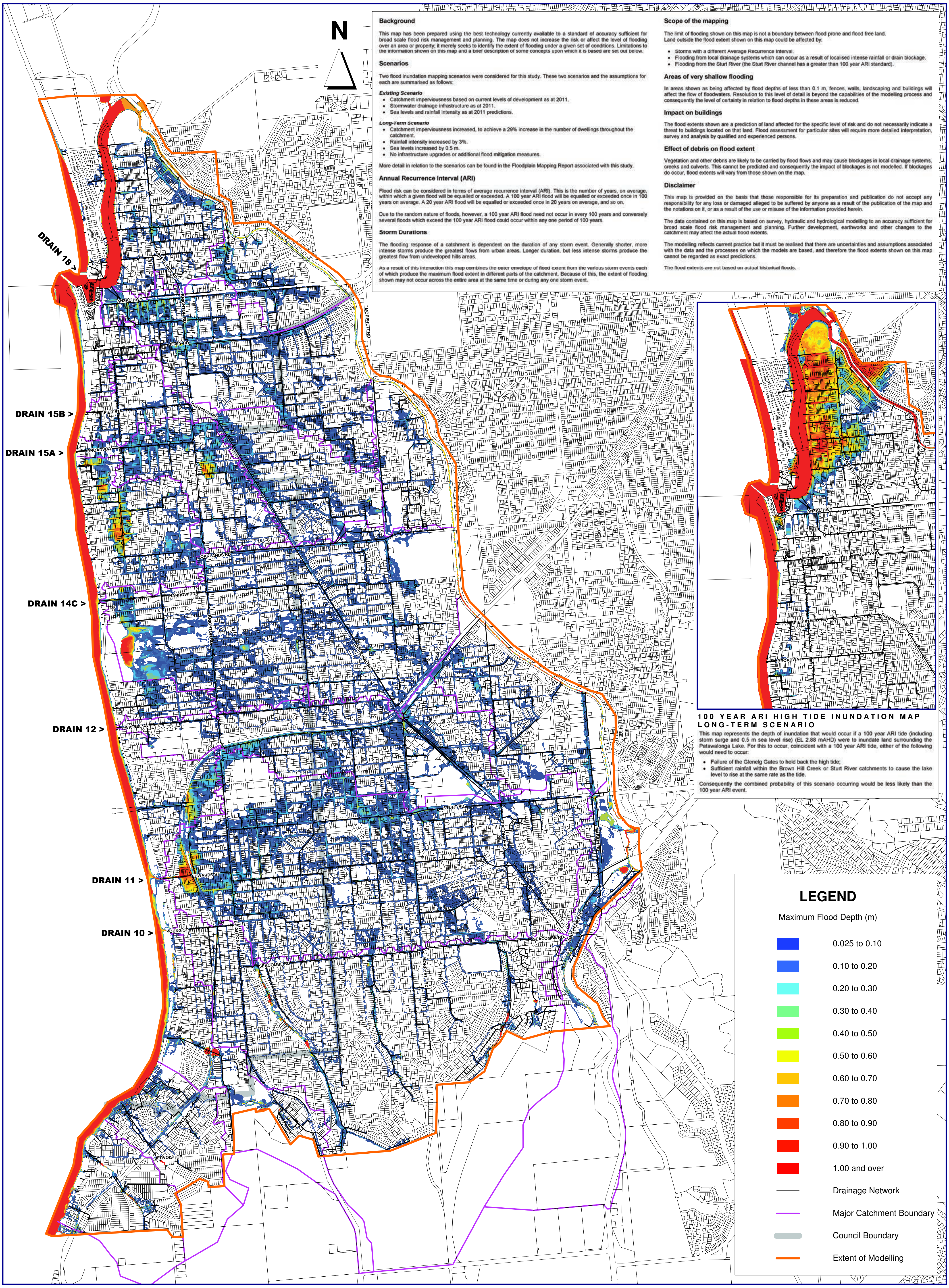
The modelling reflects current practice but it must be realised that there are uncertainties and assumptions associated with the data and the processes on which the models are based, and therefore the flood extents shown on this map cannot be regarded as exact predictions.

The flood extents are not based on actual historical floods.




**100 YEAR ARI HIGH TIDE INUNDATION MAP**
**LONG-TERM SCENARIO**

This map represents the depth of inundation that would occur if a 100 year ARI tide (including storm surge and 0.5 m sea level rise) (EL 2.88 mAHD) were to inundate land surrounding the Patawalonga Lake. For this to occur, coincident with a 100 year ARI tide, either of the following would need to occur:

- Failure of the Glenelg Gates to hold back the high tide;
- Sufficient rainfall within the Brown Hill Creek or Sturt River catchments to cause the lake level to rise at the same rate as the tide.

Consequently the combined probability of this scenario occurring would be less likely than the 100 year ARI event.

-500 0 500 1000 1500
Meters
**Scale: 1:15,000**

MAP DETAILS

| | | | |
|---|---|---|---|
| Projection : | Transverse Mercator, GDA 94<br>MGA, Zone 54 | Drainage Data : | Tonkin Consulting |
| Elevation : | Australian Height Datum (metres) | Flood Mapping : | Tonkin Consulting |
| Cadastre : | PBBI Australia | Job Number : | 2010.0878 |
| | | Filename : | MH_LT_100yrARI_Flood_Inundation_A1.wor |
| | | Drawn : | Tim Cresswell |
| | | Date : | 29/11/2011 |



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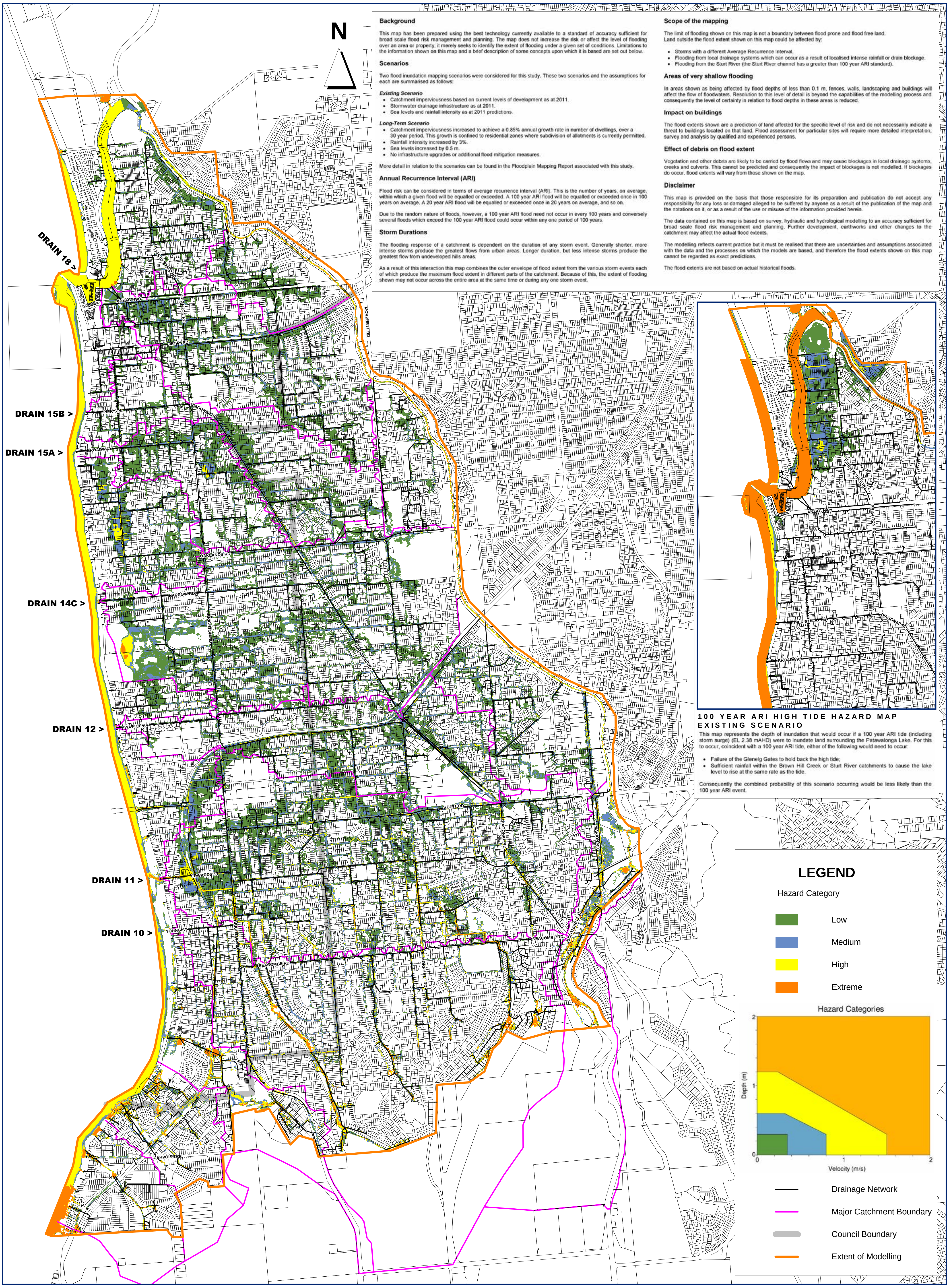
*City of Marion and City of Holdfast Bay*

100 YEAR ARI FLOOD INUNDATION MAP
LONG-TERM SCENARIO

**MARION & HOLDFAST BAY**

# Appendix C

# 100 Year Hazard Map for the Existing Scenario




## Background

This map has been prepared using the best technology currently available to a standard of accuracy sufficient for broad scale flood risk management and planning. The map does not increase the risk or affect the level of flooding over an area or property; it merely seeks to identify the extent of flooding under a given set of conditions. Limitations to the information shown on this map and a brief description of some concepts upon which it is based are set out below.

## Scenarios

Two flood inundation mapping scenarios were considered for this study. These two scenarios and the assumptions for each are summarised as follows:

***Existing Scenario***
- Catchment imperviousness based on current levels of development as at 2011.
- Stormwater drainage infrastructure as at 2011.
- Sea levels and rainfall intensity as at 2011 predictions.

***Long-Term Scenario***
- Catchment imperviousness increased to achieve a 0.85% annual growth rate in number of dwellings, over a 30 year period. This growth is confined to residential zones where subdivision of allotments is currently permitted.
- Rainfall intensity increased by 3%.
- Sea levels increased by 0.5 m.
- No infrastructure upgrades or additional flood mitigation measures.

More detail in relation to the scenarios can be found in the Floodplain Mapping Report associated with this study.

## Annual Recurrence Interval (ARI)

Flood risk can be considered in terms of average recurrence interval (ARI). This is the number of years, on average, within which a given flood will be equalled or exceeded. A 100 year ARI flood will be equalled or exceeded once in 100 years on average. A 20 year ARI flood will be equalled or exceeded once in 20 years on average, and so on.

Due to the random nature of floods, however, a 100 year ARI flood need not occur in every 100 years and conversely several floods which exceed the 100 year ARI flood could occur within any one period of 100 years.

## Storm Durations

The flooding response of a catchment is dependent on the duration of any storm event. Generally shorter, more intense storms produce the greatest flows from urban areas. Longer duration, but less intense storms produce the greatest flow from undeveloped hills areas.

As a result of this interaction this map combines the outer envelope of flood extent from the various storm events each of which produce the maximum flood extent in different parts of the catchment. Because of this, the extent of flooding shown may not occur across the entire area at the same time or during any one storm event.

## Scope of the mapping

The limit of flooding shown on this map is not a boundary between flood prone and flood free land.
Land outside the flood extent shown on this map could be affected by:

- Storms with a different Average Recurrence Interval.
- Flooding from local drainage systems which can occur as a result of localised intense rainfall or drain blockage.
- Flooding from the Sturt River (the Sturt River channel has a greater than 100 year ARI standard).

## Areas of very shallow flooding

In areas shown as being affected by flood depths of less than 0.1 m, fences, walls, landscaping and buildings will affect the flow of floodwaters. Resolution to this level of detail is beyond the capabilities of the modelling process and consequently the level of certainty in relation to flood depths in these areas is reduced.

## Impact on buildings

The flood extents shown are a prediction of land affected for the specific level of risk and do not necessarily indicate a threat to buildings located on that land. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

## Effect of debris on flood extent

Vegetation and other debris are likely to be carried by flood flows and may cause blockages in local drainage systems, creeks and culverts. This cannot be predicted and consequently the impact of blockages is not modelled. If blockages do occur, flood extents will vary from those shown on the map.

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The modelling reflects current practice but it must be realised that there are uncertainties and assumptions associated with the data and the processes on which the models are based, and therefore the flood extents shown on this map cannot be regarded as exact predictions.

The flood extents are not based on actual historical floods.

**100 YEAR ARI HIGH TIDE HAZARD MAP**
**EXISTING SCENARIO**

This map represents the depth of inundation that would occur if a 100 year ARI tide (including storm surge) (EL 2.38 mAHD) were to inundate land surrounding the Patawalonga Lake. For this to occur, coincident with a 100 year ARI tide, either of the following would need to occur:

- Failure of the Glenelg Gates to hold back the high tide;
- Sufficient rainfall within the Brown Hill Creek or Sturt River catchments to cause the lake level to rise at the same rate as the tide.

Consequently the combined probability of this scenario occurring would be less likely than the 100 year ARI event.

-500 0 500 1000 1500

Meters

**Scale: 1:15,000**

MAP DETAILS

| | |
|---|---|
| Projection : | Transverse Mercator, GDA 94<br>MGA, Zone 54 |
| Elevation : | Australian Height Datum (metres) |
| Cadastre : | PBBI Australia |
| Drainage Data : | Tonkin Consulting |
| Flood Mapping : | Tonkin Consulting |
| Job Number : | 2010.0878 |
| Filename : | MH_100yrARI_Flood_Hazard_A1.wor |
| Drawn : | Tim Cresswell |
| Date : | 19/09/2013 |

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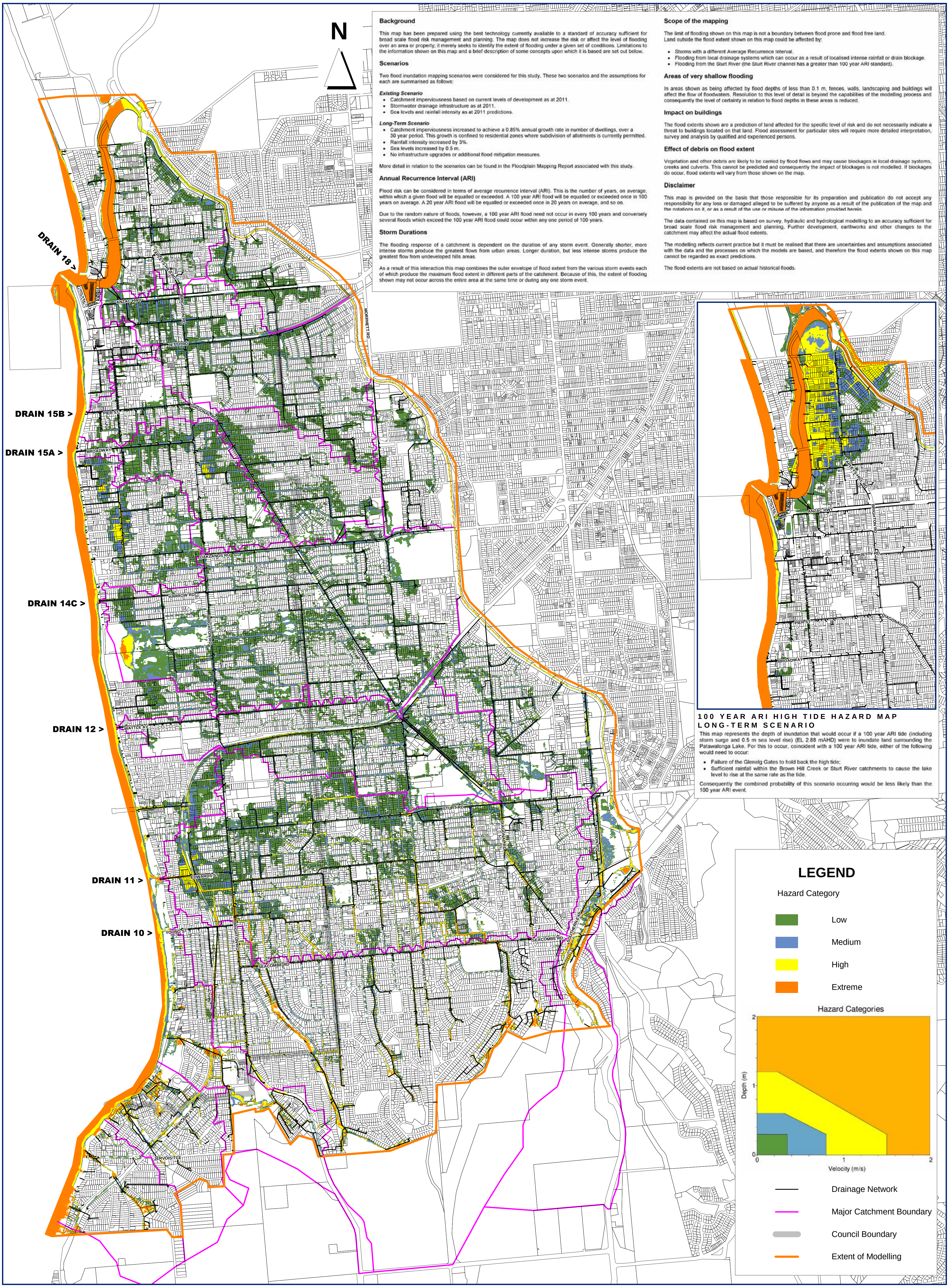
*City of Marion and City of Holdfast Bay*

**100 YEAR ARI FLOOD HAZARD MAP**
**EXISTING SCENARIO**

**MARION & HOLDFAST BAY**

# Appendix D

# 100 Year Hazard Map for the Long-Term Scenario




## Background

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## Scenarios

Two flood inundation mapping scenarios were considered for this study. These two scenarios and the assumptions for each are summarised as follows:

***Existing Scenario***

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***Long-Term Scenario***

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- No infrastructure upgrades or additional flood mitigation measures.

More detail in relation to the scenarios can be found in the Floodplain Mapping Report associated with this study.

## Annual Recurrence Interval (ARI)

Flood risk can be considered in terms of average recurrence interval (ARI). This is the number of years, on average, within which a given flood will be equalled or exceeded. A 100 year ARI flood will be equalled or exceeded once in 100 years on average. A 20 year ARI flood will be equalled or exceeded once in 20 years on average, and so on.

Due to the random nature of floods, however, a 100 year ARI flood need not occur in every 100 years and conversely several floods which exceed the 100 year ARI flood could occur within any one period of 100 years.

## Storm Durations

The flooding response of a catchment is dependent on the duration of any storm event. Generally shorter, more intense storms produce the greatest flows from urban areas. Longer duration, but less intense storms produce the greatest flow from undeveloped hills areas.

As a result of this interaction this map combines the outer envelope of flood extent from the various storm events each of which produce the maximum flood extent in different parts of the catchment. Because of this, the extent of flooding shown may not occur across the entire area at the same time or during any one storm event.

## Scope of the mapping

The limit of flooding shown on this map is not a boundary between flood prone and flood free land.
Land outside the flood extent shown on this map could be affected by:

- Storms with a different Average Recurrence Interval.
- Flooding from local drainage systems which can occur as a result of localised intense rainfall or drain blockage.
- Flooding from the Sturt River (the Sturt River channel has a greater than 100 year ARI standard).

## Areas of very shallow flooding

In areas shown as being affected by flood depths of less than 0.1 m, fences, walls, landscaping and buildings will affect the flow of floodwaters. Resolution to this level of detail is beyond the capabilities of the modelling process and consequently the level of certainty in relation to flood depths in these areas is reduced.

## Impact on buildings

The flood extents shown are a prediction of land affected for the specific level of risk and do not necessarily indicate a threat to buildings located on that land. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

## Effect of debris on flood extent

Vegetation and other debris are likely to be carried by flood flows and may cause blockages in local drainage systems, creeks and culverts. This cannot be predicted and consequently the impact of blockages is not modelled. If blockages do occur, flood extents will vary from those shown on the map.

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The modelling reflects current practice but it must be realised that there are uncertainties and assumptions associated with the data and the processes on which the models are based, and therefore the flood extents shown on this map cannot be regarded as exact predictions.

The flood extents are not based on actual historical floods.

**100 YEAR ARI HIGH TIDE HAZARD MAP**
**LONG-TERM SCENARIO**

This map represents the depth of inundation that would occur if a 100 year ARI tide (including storm surge and 0.5 m sea level rise) (EL 2.88 mAHD) were to inundate land surrounding the Patawalonga Lake. For this to occur, coincident with a 100 year ARI tide, either of the following would need to occur:

- Failure of the Glenelg Gates to hold back the high tide;
- Sufficient rainfall within the Brown Hill Creek or Sturt River catchments to cause the lake level to rise at the same rate as the tide.

Consequently the combined probability of this scenario occurring would be less likely than the 100 year ARI event.

## LEGEND

Hazard Category

- Low
- Medium
- High
- Extreme

Hazard Categories

Depth (m)

Velocity (m/s)

- Drainage Network
- Major Catchment Boundary
- Council Boundary
- Extent of Modelling

-500 0 500 1000 1500

Meters

**Scale: 1:15,000**

MAP DETAILS

| | |
|---|---|
| Projection : | Transverse Mercator, GDA 94 MGA, Zone 54 |
| Elevation : | Australian Height Datum (metres) |
| Cadastre : | PBBI Australia |
| Drainage Data : | Tonkin Consulting |
| Flood Mapping : | Tonkin Consulting |
| Job Number : | 2010.0878 |
| Filename : | MH_LT_100yrARI_Flood_Hazard_A1.wor |
| Drawn : | Tim Cresswell |
| Date : | 19/09/2013 |


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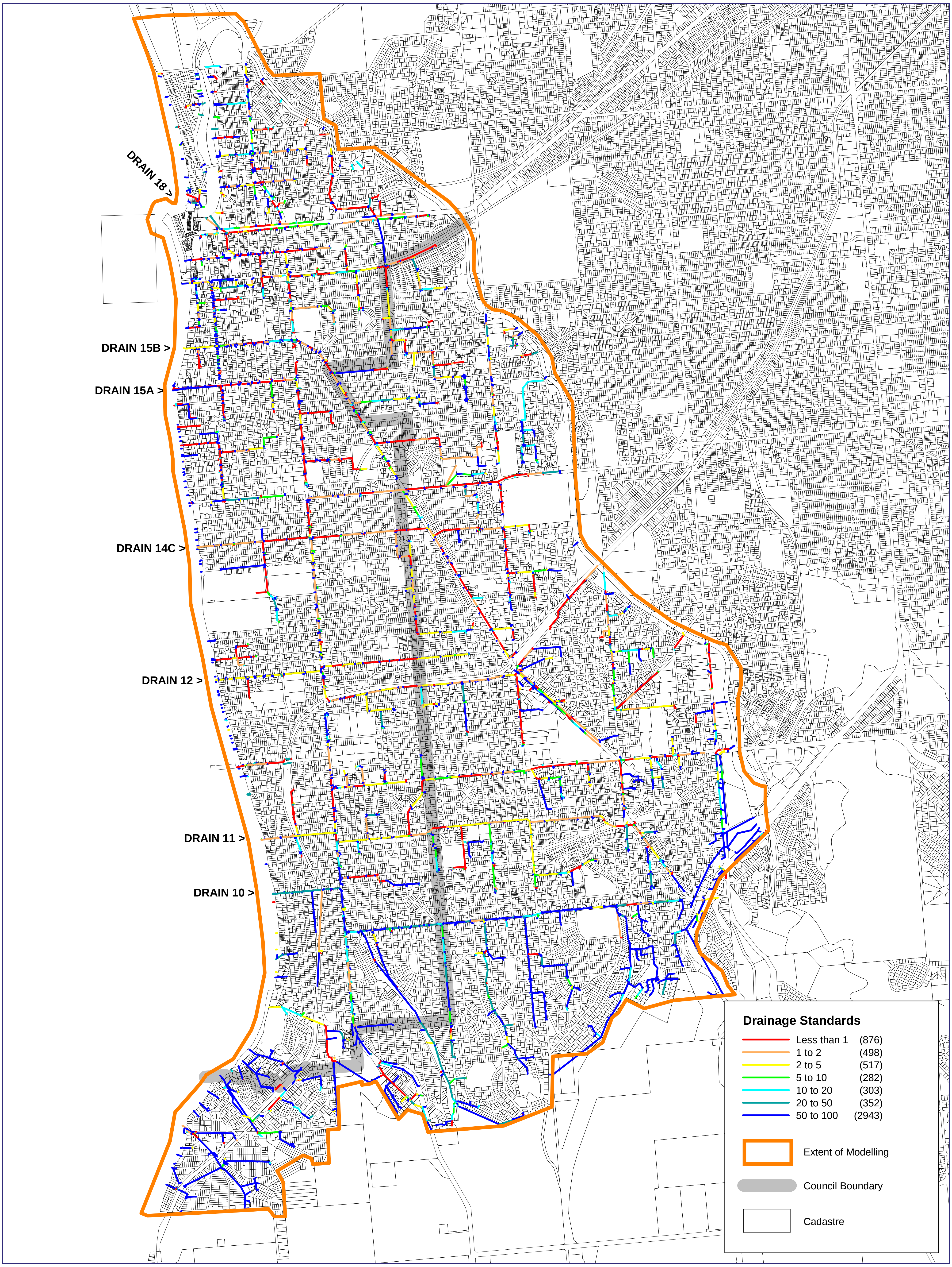
*City of Marion and City of Holdfast Bay*

**100 YEAR ARI FLOOD HAZARD MAP**
**LONG-TERM SCENARIO**

# MARION & HOLDFAST BAY

# Appendix E

# Drainage Standard Maps



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N
W E
S

Scale: 1:30,000

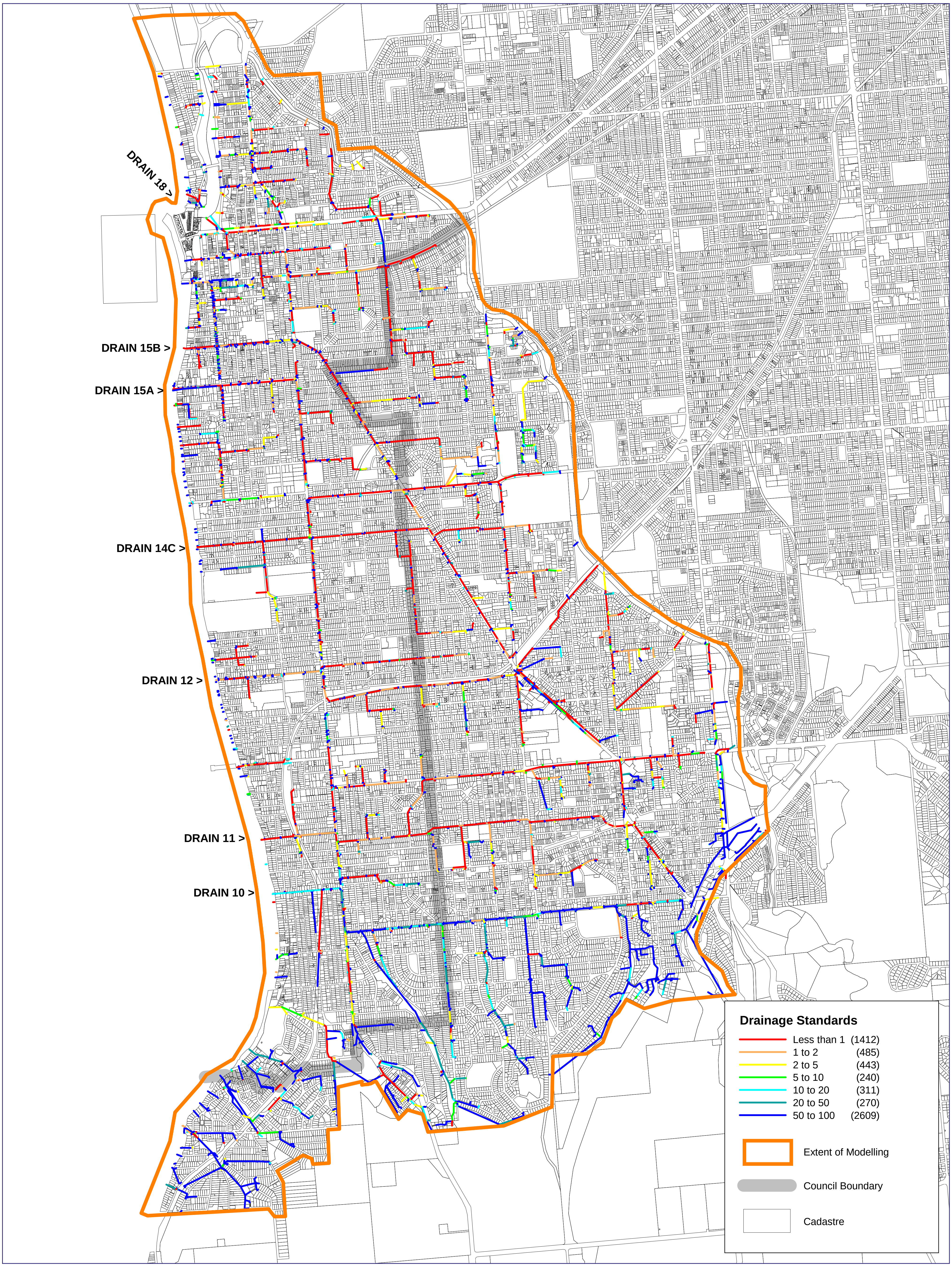
Job Number: 2010.0878
Filename: Holdfast_Exist_Drain_Std_A3.wor
Revision: B
Date: 17/01/2013
Drawn: TDC

Data Acknowledgement:
Cadastre supplied by PBBI Australia Pty Ltd

City of Marion and City of Holdfast Bay

## EXISTING DRAINAGE STANDARDS

MARION & HOLDFAST BAY



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Scale: 1:30,000

Job Number: 2010.0878
Filename: Holdfast_Future_Drain_Std_A3.wor
Revision: B
Date: 17/01/2013
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City of Marion and City of Holdfast Bay

# FUTURE DRAINAGE STANDARDS

MARION & HOLDFAST BAY