Final Report

Shoal Bay Drainage Study

81016106

Prepared for
Port Stephens Council

18 May 2016

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Executive Summary

Port Stephens Council have engaged Cardno to undertake a drainage study to identify the existing drainage deficiencies to the ongoing drainage problems being experienced at Shoal Bay and assess conceptual options to provide a cost effective solution.

The study reviews all background material and consults with stakeholders, including the community, to identify where historical flooding has occurred and the causes of this flooding. Data and previous studies were provided by Port Stephens Council to identify flooding issues in Shoal Bay. Additionally, an initial workshop was held for the community to be informed of the study and provide information regarding historical flooding in the area. Cardno collated and reviewed the information provided and established a combined one-dimensional and two-dimensional hydrological and hydraulic stormwater model using XPSWMM to reproduce flood behaviour for the existing conditions in Shoal Bay. Design rainfall events considered are the 1%, 10% and 20% annual exceedance probability (AEP) storm events. The modelling results were discussed with stakeholders and the community to identify any deficiencies in the findings and potential solutions to the flooding issues experienced.

Areas identified as experiencing significant, widespread flooding issues that could be potentially mitigated by public works were:

> The north end of Horace Street;

> Bullecourt Street;

> Passchendaele Park and the south end of Rigney Street, and;

> The residential properties between Fingal Street and Verona Road.

Five options were selected by Port Stephens Council and Cardno to be modelled to determine their imapct on flood behaviour. These options considered different combinations of strategies for reducing flooding, including catchment wide infiltration, upgrading of stormwater pipes, upstream diversion of runoff and detention basins.

The four options were subjected to a multi-criteria assessment based on:

> Flood level reduction;

> Net Present Value over 50 a year timeframe;

> Construction Impact, and;

> Potential environmental impact.

It is recommended that Option 2 be adopted as the preferred option and taken forward to detailed design and construction. This option involves the construction of large twin box culverts from Verona Road along Rigney Street, Messines Street and Government Road to Shoal Bay Beach. This option ranked first in the least cost muti-criteria weighting scenario and second in the remaining scenarios (community desire, best environmental outcomes, and highest safety). The Net Present Value of Option 2 is $6.02 M.

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1 Introduction

Cardno has been commissioned by Port Stephens Council to undertake a drainage study to identify existing drainage problems and assess options which provide a cost effective solution for alleviating these problems.

Urban development in Shoal Bay has intensified in recent years. However, the urban stormwater drainage system has not been upgraded to reflect this development resulting in unacceptable ponding and flooding issues for local residents, land owners and businesses.

1.1 Objective

The objective of this drainage study is to carry out hydrological and hydraulic analysis of the Shoal bay drainage system. The analysis will also investigate a number of different potential solutions to solve the drainage issues faced within Shoal Bay.

Specific objectives include:

- Identify specific problem areas and causes of these problems within the catchment, including consultation with the stakeholders and the public.
- Identify strategies and/or mitigation works to alleviate the current drainage and flooding issues. Solutions must consider the 1% AEP storm event and resulting stormwater drainage network should resemble a major/minor system, as defined in Australian Rainfall & Runoff (AR&R).
- Provide an itemised cost estimate for each potential solution.
- Recommend a cost effective solution to Council.

2 Background

2.1 Location

The town of Shoal Bay is located within the Port Stephens Local Government Area (LGA), along the eastern coastline of the New South Wales Hunter Region. To the immediate north is Port Stephens with headlands on the Pacific Ocean coast immediately northeast of the town. The closest major city is Newcastle, approximately 50 km southwest. **Figure 2-1** shows the location of Shoal Bay within the Port Stephens area.

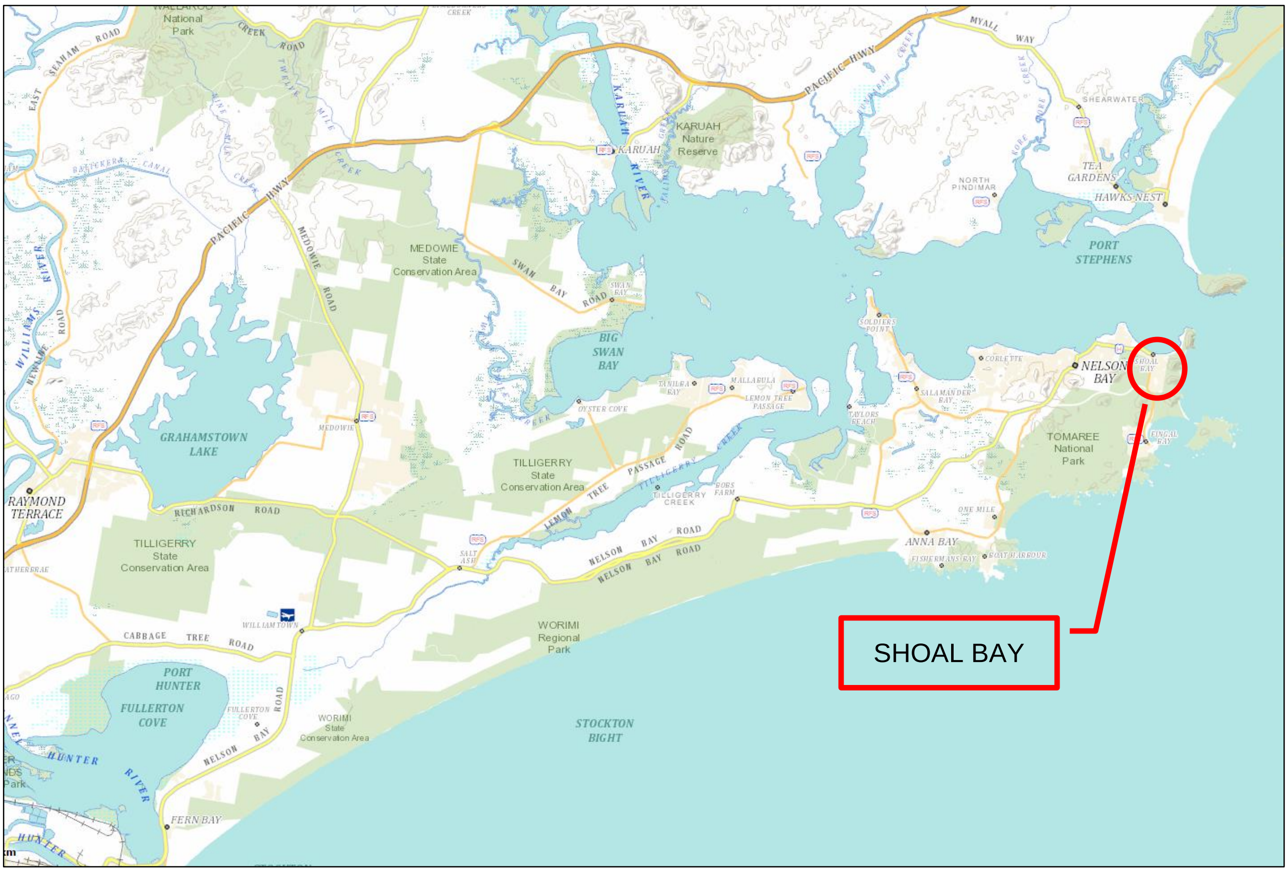


Figure 2-1 Study Area Location

According to the Australian Bureau of Statistics (ABS), there were 1,838 people in Shoal Bay at the time of the 2011 census. The town is a popular tourist destination and this is reflected by 39.9% of the 1,339 private dwelling being unoccupied at the census date.

The town is bordered by Tomaree National Park to the south, west and east.

The study area is restricted to Shoal Bay's immediate drainage network catchment area. This is bounded by mountains to the east and south, wetland to the west, and beach to the north. The total area is approximately 134 ha, as outlined in **Figure 2-2** below.

Figure 2-2 Study Area

The main transportation routes through town are Shoal Bay Road (towards Nelson Bay) and Government Road (towards Fingal Bay).

2.2 Climate

Shoal Bay experiences a warm and temperate climate due to its latitude and location adjacent to the Pacific Ocean. The closest Bureau of Meteorology (BoM) weather station is located at Nelson Head lighthouse, approximately 2 km northwest. This station has been in operation intermittently since 1881.

Monthly mean maximum temperatures range from 27.3°C to 17.4°C in January and July, respectively. Monthly mean minimum temperatures range from 18.9°C to 8.9°C in February and July, respectively. The mean annual rainfall is 1,352 mm with typically the driest monthly being October and the wettest month being June.

2.3 Topography

Generally, the town of Shoal Bay is located between the mountain ranges to the east and south, wetlands to the west and the beach to the north. Elevations slope from high in the east to low in the west. The maximum elevation of the mountains to the east is approximately 150 m AHD. Most of the western half of the town is very flat and the beach along the northern boundary is approximately 1 m to 2 m below Shoal Bay Road.

There are no defined watercourses within the study area, according to Land and Property Information (LPI) New South Wales.

2.4 Land Use

The Port Stephens Local Environmental Plan (LEP) 2013 indicates that land use within the study area consists of low (R2) and medium (R3) density residential, public recreation (RE1), local centre (B2), private recreation areas (RE2), Infrastructure (SP2), and national park and nature reserves (E1). **Figure 2-3** is an excerpt from the LEP showing the local area.

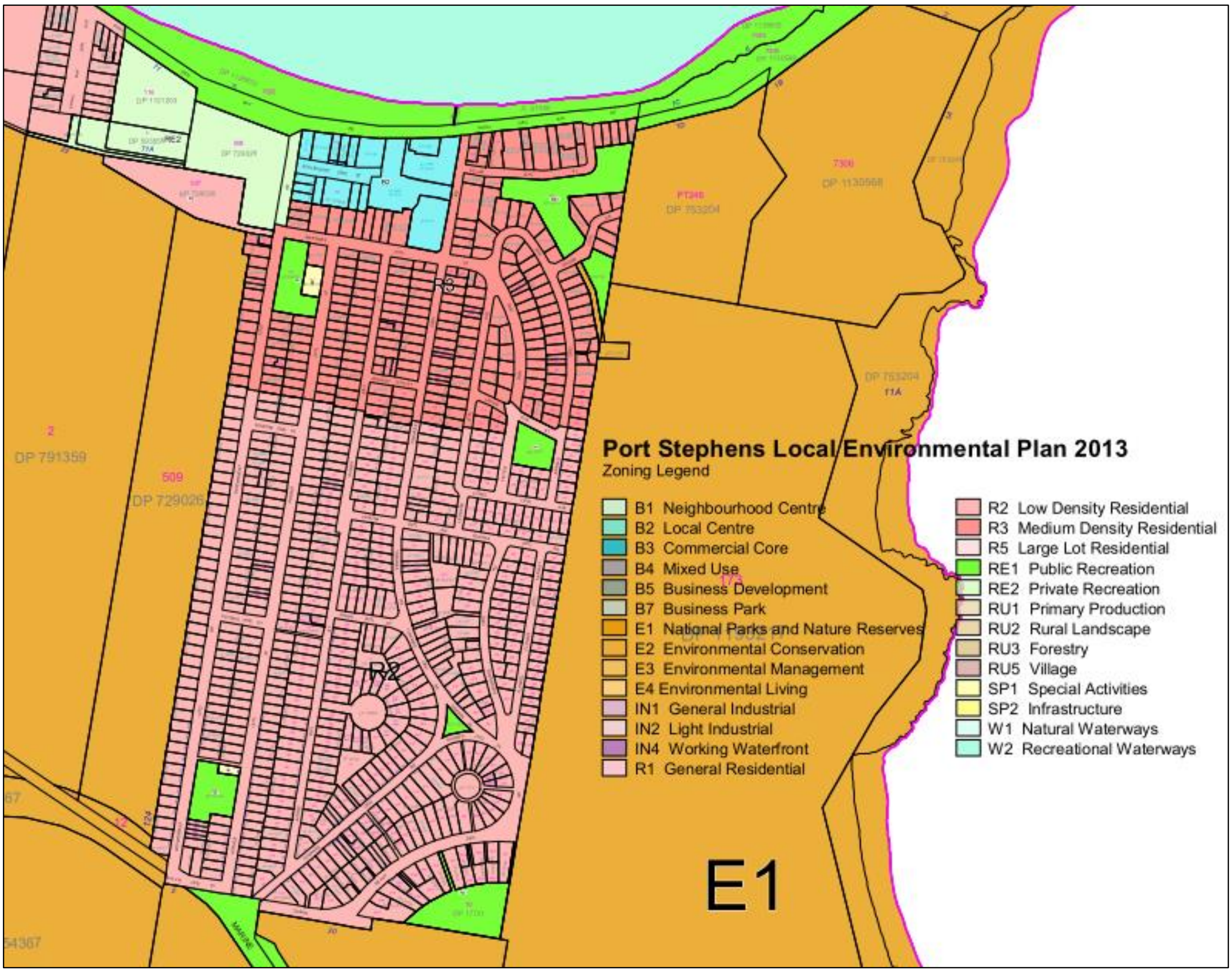


Figure 2-3 Land Use in Shoal Bay

2.5 Urban Drainage

Shoal Bay contains a network of formalised drainage infrastructure. Some of this infrastructure consists of roadside swales and in other areas, where recent road construction has been undertaken, kerb and gutter has been installed with lateral drainage stormwater pipes installed below ground. Generally, the system conveys runoff from urban areas towards one of three outlets on Shoal Bay beach, or allows it to infiltrate into the underlying sandy soil.

Due to the relatively recent increase in development in Shoal Bay, combined with the flat and sometimes locally undulating ground elevations, the existing stormwater drainage network is not ideal. There are many trapped low points along roadways where overland flows can only escape through private properties. Trunk drainage lines run through private properties with limited easement for future upgrading works.

Runoff from roads without kerb and gutter causes ponding in front yards and results in infrequent flooding of homes. Most of the existing pipe network is undersized, causing flooding in properties along a number of roadways in the study area.

There have been recent installations of pervious pipe within the study area. These are located in areas of known high infiltration rates; specifically along Leonard Avenue. These were designed as test case sections to evaluate the reduction of stormwater loads on infrastructure lower in the catchment where there were identified stormwater problems.

Port Stephens Council has provided Cardno with information from its GIS database which details of the entire existing stormwater infrastructure in the study area.

3 Data Review

3.1 GIS Data

3.1.1 Cadastre

Council have provided data to show the cadastre within Shoal Bay. This is used to identify individual properties and blocks which are affected by flooding.

3.1.2 Land Use Zoning

Council have provided adequate data for identification of land use zones within the study area. This information will be utilised to delineate roughness areas for 2D hydraulic modelling.

3.1.3 Stormwater Drainage Assets

Council's GIS-based stormwater asset data base is detailed and includes the pit and pipe network within the study area. The following applicable information was identified:

> Pipes – Size, material, joint type, length, number of cells, asset ID

> Pits – Location, depth, upstream and downstream invert depth, opening type, lintel length, asset ID

> Headwalls – Material, length, height, wing wall length, asset ID

> Gross Pollutant Traps – Type (manufacturer), internal dimensions, asset ID

> Detention/retention Basins – Location, type, size, asset ID

Not all information is available for every item in the database. In the absence of specific information for a network element, Cardno has made appropriate engineering judgement or, where necessary, verified the required information on site.

Shoal Bay Holiday Park and the aged care facility to the immediate south of the Holiday Park contain significant private drainage networks to manage runoff at local low points. These networks are connected to the public drainage network and assumptions have been made on pit locations, pit types, pit depths, pipe alignments and pipe sizes based on aerial photography and previous modelling (**Section 3.6**).

3.2 Aerial Photography

Council has supplied aerial photography of the study area. The image was taken in 2012 and will aid in the confirmation of location of elements in the stormwater drainage network and identifying any structures at risk of flooding.

Cardno has also utilised Nearmap and Google aerial imagery, including Street View photography, to identify individual pit locations and types.

3.3 Topography and Survey

The following information has been used to create a digital terrain model (DTM). The DTM is the surface information utilised by XPSWMM 2D hydraulics, as outlined below.

3.3.1 LiDAR

LiDAR (Light Detection and Ranging) is a survey tool used to create relatively accurate three dimensional ground maps of large areas. LiDAR data for this project was supplied as a 1 m digital elevation model (DEM) produced using a triangular irregular network method of averaging ground heights to formulate a regular grid. This data has a vertical accuracy of +/-300 mm and horizontal accuracy of +/-800 mm. The data has been filtered to remove buildings and the tree canopy.

3.3.2 Ground Surveys

Ground surveys are used where necessary to supplement the LiDAR data, as the triangulated network data produced by surveyors from measurements on site has higher accuracy than LiDAR.

Ground survey data has been provided for the following areas:

> Northern end of Horace Street

> Tomaree Road, Shoal Bay Road to Messines Street

> Sections of Ronald Avenue, Leslie Street, Victor Parade, Essendene Road, Flannel Flower Fairway and Fingal Street

Some of the survey information contains invert levels of stormwater infrastructure, particularly in Horace Street which is a known drainage problem area.

3.4 Geological and Geotechnical Information

Geotechnical investigations and reports provide critical information on interactions with rainfall and runoff produced in a catchment. Specifically, they can provide a reasonable indication of the infiltration rate the underlying soil provides.

3.4.1 Douglas Partners Geotechnical Investigation (2005)

This report was unavailable to Cardno for review. However, reference is made to it in other studies (**Section 3.5**), specifically infiltration rates into the sandy soils below.

3.4.2 Geological Survey of New South Wales

According to the Geological Survey of NSW, there are three distinctive geological zones underlying the study area:

> Qps – Sand, originating from the Quaternary period

> Qph – Sand with underlying bedrock, originating from the Quaternary period

> Clne – Volcanic rock from the Gilmore Volcanics Group, originating from the Visean epoch in the Carboniferous period

Figure 3-1 shows these geological zones overlaid on the study area mapping.

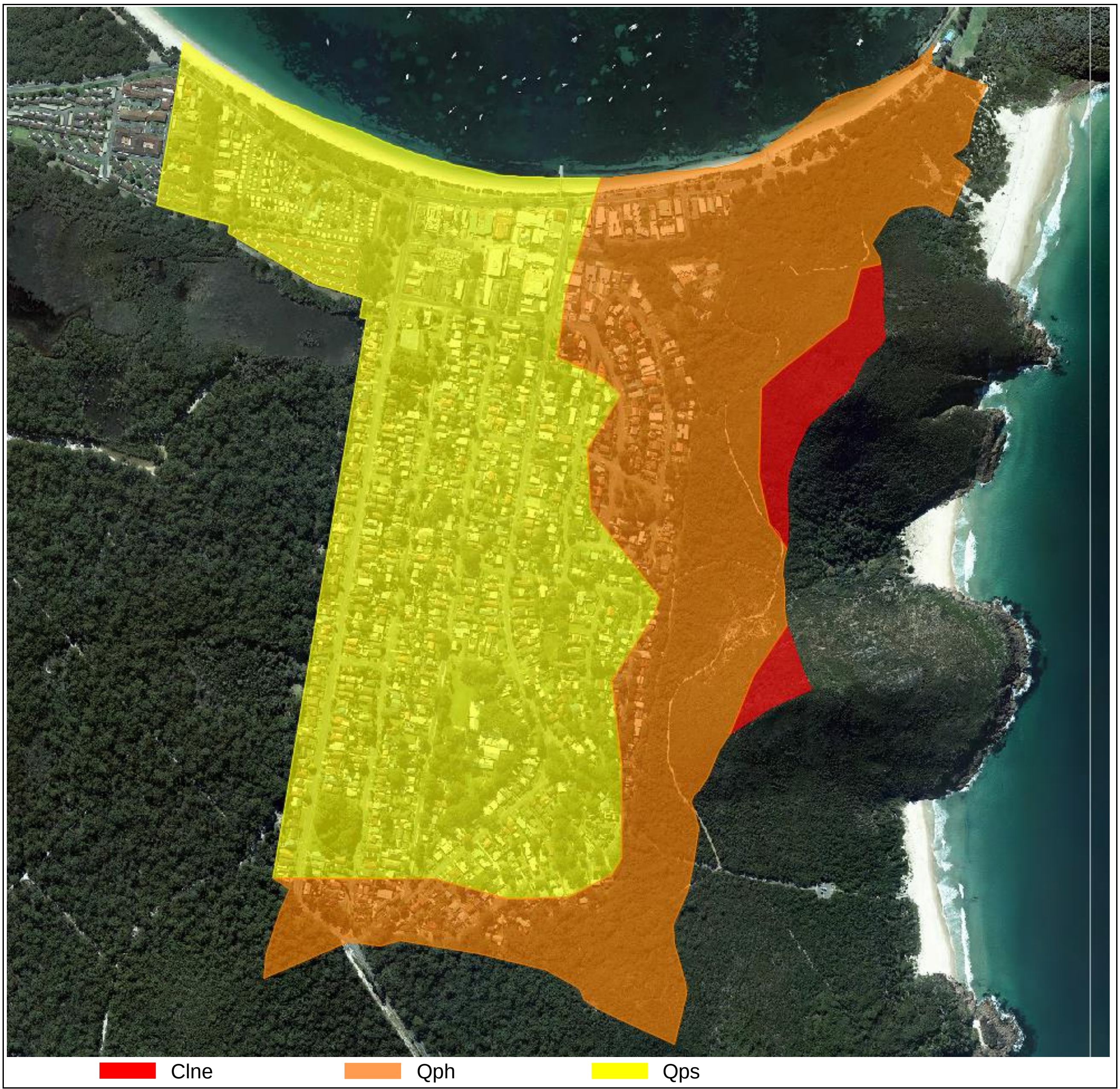


Figure 3-1 Geological Zones

This is consistent with what is observed on site and is reflected in infiltration rates used in previous modelling. To the east are steep sloped volcanic mountains, gradually become flatter while moving west and becoming predominantly sandy soils with relatively high infiltration rates.

The three geological zones will be used to spatially differentiate the effective infiltration rates of the underlying soils.

3.5 Previous Studies

3.5.1 Port Stephens Flood Study - Stage 2 (1996)

This report was produced by the Manly Hydraulics Lab (MHL) in 1996 for Port Stephens and Great Lakes Councils. It was undertaken to determine the nature and extent of flooding around the foreshore of Port Stephens and Tilligerry Creek.

The main outcome from this report, in regard to the current drainage study, is the definition of design water levels at Shoal Bay.

3.5.2 Shoal Bay Drainage Study (2009)

This report was produced by Port Stephens Council in response to the known drainage problems in the Shoal Bay area. It assessed the performance of the existing drainage network and identified possible solutions to alleviate flooding in the catchment.

The study included the construction of a stormwater model using the DRAINS software. Through modelling the existing condition, problem areas were identified and a range of options were evaluated using this model and costed including:

> Implementation of infiltration devices

> Rerouting of existing pipes

> Construction of new pipes

> Increasing pipe sizes

> Creation of new detention basins or expansion of existing basins

The report recommended that:

> An infiltration solution be adopted to reduce the outflow to Shoal Bay Beach

> All future development should incorporate a combination of detention and infiltration to reduce runoff at the lot level

As part of the current report, Cardno has reviewed the recommendations from Council's 2009 study and has considered the incorporation of some of the identified potential solutions.

3.5.3 Shoal Bay Infiltration Drainage Investigation (2013)

This study represents an extension of the previous study undertaken by PSC in 2009. It further investigates the design, assessment and construction of an infiltration solution to the drainage problems in Shoal Bay.

A range of different mitigation options were evaluated, including those utilising infiltration as those not.

The report concludes that overland flows are re-entering the existing system downstream in the catchment and overloading the already strained pipe system. Some of these overland flows can be reduced by 90% - 100% by implementing the proposed infiltration system upstream in the catchment.

The study recommends that:

> Improved routine maintenance should be implemented to optimise the efficiency of the system

> A program be prepared for the implementation of its proposals, monitor their effectiveness and further improve drainage design

Cardno has been able to incorporate some of the identified potential solutions and utilise the 2013 DRAINS model for the XPSWMM model calibration, as part of this report.

3.6 Previous Stormwater Modelling

3.6.1 Shoal Bay Drainage Study (2009)

A DRAINS model was constructed as part of this study, using DRAINS version 2009.01. PSC has provided to Cardno an existing conditions model and multiple options models.

This model does not extend to the eastern-most extents of Shoal Bay Road and only includes two of the three outlets to Shoal Bay Beach.

Rainfall input data was produced for this model by IFD analysis based on Australian Rainfall and Runoff (AR&R) 1987. In accordance with AR&R, ILSAX type hydrological modelling is used to determine runoff from catchments when the aforementioned rainfall is applied.

One soil type was used across the entire model and this utilised the Horton infiltration equation. It is considered that this approach can be limiting to the model as it may not accurately reflect the different degree of infiltration exhibited spatially across the study area. The initial and final infiltration rates

(250 mm/hr and 150 mm/hr, respectively) are considered to be slightly high and may only represent subcatchments in the upper study area where infiltration is greater.

One Antecedent Moisture Condition (AMC) was applied to all rainfall events. It was assumed that an Antecedent Rainfall Depth (ARD) of 100 mm was applied to the catchment before commencement of the model. This is a reasonable assumption, but Cardno considers that changing this for different storm event Annual Exceedence Probability (AEP) would result in more accurate modelling.

The chosen depression storage values are considered to be appropriate.

The DRAINS model utilises the kinematic wave equation to determine times of concentration for the subcatchments. This is considered more accurate than applying a constant time of concentration for all urban subcatchments, which is typical of most LGA engineering design guidelines.

Lot based infiltration systems and rainwater tanks were not considered for urban catchments. This is a conservative, but reasonable, assumption given that it is difficult to quantify the extent and effectiveness of these systems for each lot in the entire town.

In the standard DRAINS software, kerb inlet pits and other pits utilise a combination of predefined NSW Department of Housing kerb inlets and user defined 'Lintel only' and 'Letterbox inlet' inlet types. The rating curves for the user defined inlet are considered to be reasonable an accurate.

Inlet pit blockage factors differed throughout the model, ranging from 0 (unblocked) to 0.5 (50% blocked) and one instance of 0.75 (75% blocked). These values also varied between model scenarios (i.e. modelling of existing and potential options).

The infiltration value used (180 mm/hr) for the various permeable pipes and basins subject to infiltration was consistent across the entire model and may not reflect the differing infiltration rates across the study area and effective infiltration rates within different storm event AEPs. These rates are consistent throughout the storm event and not calculated using the Horton equation. A constant infiltration rate of 50 mm/hr was applied to the sides of the basin in Passchendaele Park whilst the floor is considered impermeable, or to be waterlogged. This is a reasonable assumption given the permanent water level within the basin is caused by the water table below. The basin near Box Beach Road did not have any infiltration rate applied.

The outfalls to Shoal Bay Beach assume a downstream hydraulic control which is consistent with sea water levels from the Port Stephens Flood Study (MHL, 1996).

3.6.2 Shoal Bay Infiltration Drainage Investigation (2013)

Similar to the accompanying report, the models associated with this study are an extension and update from the previous study by Port Stephens Council in 2009.

This model was constructed using a later version (but unspecified) of DRAINS. There are multiple options files and an existing conditions file.

Differences in the model compared to the 2009 study include a revised soil type within the ILSAX model. The model utilises the soil type 3 which is a conservative selection to account for blocking and silting up. This corresponds to an initial and final Horton infiltration rate of 125 mm/hr and 6 mm/hr, respectively. The AMC selected is the default 3, which corresponds to an ARD of 50 mm. The infiltration value used for infiltration devices (i.e. infiltration pipes and basins) is 300 mm/hr which is relatively very high.

The existing conditions DRAINS model (2013) has been used to calibrate Cardno's existing condition used in the XPSWMM 2D model. However, Cardno has recommended that the soil type for the study area be revised to 2 to reflect a more accurate representation of the underlying soils in Shoal Bay.

3.7 Site Investigation

A joint site investigation was undertaken by PSC and Cardno staff to identify previously-known drainage problem areas within the study area. Additionally, this investigation gave Cardno's modeller an appreciation of the multiple input parameters and controls involved in the establishment of an accurate and site-verified 2D hydraulic model.

4 Existing Conditions Assessment

4.1 Model Establishment

Cardno developed a series of integrated 1D/2D models to determine the extent and flow characteristics of stormwater within the town of Shoal Bay. The software utilised to accomplish this is XPSWMM, which includes the TUFLOW engine to compute 2D flow calculations.

Modifications to the existing stormwater network can be easily made to determine the impact of these works on the local stormwater flow regime. This can be in the form of larger pipes/channels, increase pit inlet capacities or dynamic elevation shapes to represent other structural above ground works (i.e. basins, levees, earthworks, etc.).

4.1.1 2D Modelling Advantages & Disadvantages

The main advantage of a 2D model is that it can provide a realistic description of the flows throughout the study area. According to AR&R Project 15: Two Dimensional Modelling in Urban and Rural Floodplains, when compared to a traditional 1D stormwater modelling, it can be said that 2D modelling has the following advantages:

> Floodplain flowpaths do not need to be predetermined by the modeller, as they are computed directly as a function of the model terrain and the applied flows.

> Flowpaths can change with changes in water level in much the same way as they do in reality.

> Within an urban context, cross-momentum of flow splits at road intersections is accounted for, thus providing a far more realistic representation of the dynamics of flow spreading across a pavement at a road junction. This can have a significant impact on flow splits in the road reserve downstream of an intersection.

> Losses due to two-dimensional effects such as bends and flow separations are automatically included within the computation, and do not need to be accounted for by increasing the roughness parameter or energy loss factors.

> Model results can provide details of the flow distribution within individual flowpaths.

> Model results can be used directly for mapping flood extents and inundation depths, velocities and safety hazard.

Conversely, a 2D model has the following disadvantages when compared with 1D models:

> Significantly more survey data input is required. The magnitude of this disadvantage has declined over recent time as aerial survey techniques have improved and become more cost-effective, allowing survey data to be collected over large areas. Aerial Laser Survey (ALS or LiDAR) is an example of such a technique.

> Significantly more computation time is required. Even with the power of modern desktop computers, 2D model simulations can take many hours and even days to complete, depending on the model extent, grid/mesh resolution and event time. This is simply due to the increase in the number of model elements (1D compared to 2D) requiring a longer computation time to resolve. Section 4.1.2.2 illustrates this effect further, as below.

> The 2D approach usually requires a trade-off between the total number of grid/mesh elements (determined by the grid/mesh resolution and model extent) and run time, particularly for fixed grid models. As the model domain increases in area, grid and average mesh resolution correspondingly needs to decrease in order to maintain the total number of grid/mesh elements and avoid excessive run times. Thus, the average topographic resolution in 2D models can diminish due to run time concerns.

4.1.2 Catchment Hydrology

Hydrological modelling is, in general, be consistent with the latest version of Australian Rainfall and Runoff (AR&R) and Port Stephens Council's Handbook for Drainage Criteria (2008).

4.1.2.1 Catchment Extents

The proposed catchment is defined by the study area and is approximately 134 hectates in area. This was determined using LiDAR data to establish contour data and delineate the catchment area contributing to stormwater flows in Shoal Bay. The western extent of the catchment area is very flat and the boundary of the catchment for the purposes of this study area was taken to be Shoal Bay Avenue, inclusive of the properties fronting this road. Similarly, the eastern extent of the catchment was taken to be the eastern cadastral boundary of Shoal Bay Road.

The catchment extent will cover all of the area expected to experience significant flooding from local stormwater based on previous rainfall events and complaints from the community.

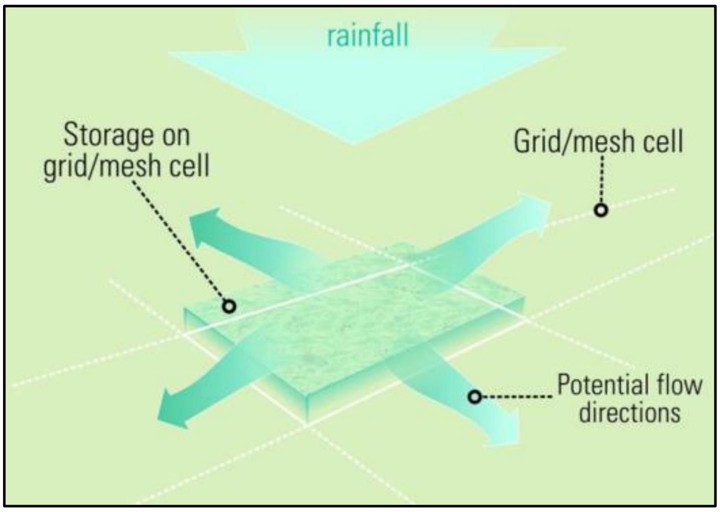
4.1.2.2 Rainfall

The rainfall applied to this catchment was consistent with Council's Handbook for Drainage Criteria (2008) for the Nelson Bay area.

XPSWMM is capable of undertaking hydrological modelling using traditional approaches with a separate hydrological model to determine flows at individual subcatchment outlets (or nodes) before routing them through a hydraulic model.

Additionally, XPSWMM is capable of utilising a rainfall on grid approach to hydrological modelling. The flowing has been extracted and paraphrased from AR&R Project 15: Two Dimensional Modelling in Urban and Rural Floodplains to describe the rainfall on grid approach.

> *This method works through the application of rainfall directly onto the 2D domain. The rainfall depth at a particular time step is applied to an individual grid cell, and the 2D model utilises its internal hydraulic calculations to determine the runoff from this grid cell. In a similar manner to traditional hydrological modelling, runoff from an individual grid/mesh cell will be dependent on:*
>
> \> *Grid cell area*
>
> \> *Rainfall depth*
>
> \> *Grid cell roughness*
>
> \> *Slope between neighbouring grid cells*
>
> \> *Rainfall losses*
>
> *In a 2D model runoff can flow in four directions, depending on the slope and water level in the neighbouring grid cells. In addition, not all rainfall may be converted to runoff as the 2D grid cell may provide storage.* ***Figure 4-1*** *below illustrates the concept of direct rainfall on an individual grid cell.*



(Image Source: AR&R Project 15)

Figure 4-1 Rainfall on Grid Conceptualisation

This approach can potentially be utilised for any flow analysis. Currently, the general reasons for using this technique are:

> *Flat terrain and catchment*

> *Cross catchment flows*

> *Detailed urban studies*

There are both advantages and disadvantages of using the direct rainfall method, discussed as follows.

Advantages:

> *Assumptions on catchment outlet locations are not required. When a traditional hydrological model is utilised, an assumption is required on where the application of catchment outflows are made to the hydraulic model.*

> *Assumptions on catchment delineation are not required. Flow movement is determined by 2D model topography and hydraulic principles, rather than on the subcatchment discretisation, which is sometimes based on best judgement and can be difficult to define in flat terrains.*

> *Cross catchment flow is facilitated in the model. In flat catchments, flow can cross a catchment boundary during higher rainfall events. This can be difficult to represent in a traditional hydrological model.*

> *Overland flow is incorporated directly. Overland flow models in traditional hydrological packages require a significant number of small sub-catchments, to provide sufficient flow information to be applied to a hydraulic model.*

Disadvantages:

> *Direct rainfall is a new technique, with limited calibration or verification to gauged data. Detailed checking is needed in the application of this approach.*

> *Potential significant increase in hydraulic model run times. Hydrological models on their own generate peak flows significantly faster than direct rainfall models.*

> *Requires digital terrain information. Depending on the accuracy of the results required, there may be a need for extensive survey data, such as aerial survey data.*

> *Insufficient resolution of smaller flowpaths may impact upon timing. Routing of the rainfall applied over the 2D model domain occurs according to the representation of the flowpaths by the 2D model. Higher in the catchment, these flowpaths become smaller and it is likely that they will not be as well-represented by the 2D model as they may exist on a sub-grid scale. This may affect timing of runoff routing. There are various methods for overcoming this.*

> *The shallow flows generated in the direct rainfall approach may be outside the typical range where Manning's 'n' roughness parameters are utilised. There are potential solutions to this including depth varying roughness.*

Cardno has utilised the rainfall on grid approach to hydrological modelling. A number of test subcatchments in Shoal Bay were identified to compare flows produced by this method to traditional hydrological calculation methods (refer **Section 4.2.3**).

4.1.2.2.2 Design Storm Events

In accordance with Council's brief, Cardno has analysed the 1%, 10% and 20% AEP storm events for the existing conditions scenario. The same storm events have been analysed for assessing potential stormwater impact mitigation options and sensitivity analysis.

4.1.2.2.3 Critical Storm Duration

Cardno has utilised the previous 1D DRAINS modelling to determine the critical storm durations for each design AEP storm event, as well as both the existing and future scenarios. This is advantageous as run times for this model last a few minutes compared to the hours required to run a 2D model and assess multiple storm event durations.

Cardno assessed durations ranging from 5 minutes to 24 hours. Critical storm durations were determined by reading maximum flow, water level and volume outputs at various critical areas in the model (e.g. headwall outlets, problem flooding areas, largest subcatchments). Not all subcatchments and model elements resulted in maximum values for one duration, so the critical storm selected exhibited maximum values for the significant elements in the model.

Generally, critical storm durations for the existing scenario are governed by the volume of water flooding significant areas in the study (e.g. the north end of Horace Street) as the discharge from the catchment at Shoal Bay beach is similar for each duration analysed because it is undersized to carry major system flows. However, for the proposed options, the critical storm is more accurately reflected in the maximum flows discharging to Shoal Bay beach as the infrastructure is more closely designed to the requirements of AR&R (i.e. having major/minor system controlling runoff). **Table 4-1** shows the critical storm durations selected for each design AEP event.

Table 4-1 Design Storm Critical Durations

Design Storm Annual Exceedance Probability (AEP)	Critical Duration Existing Scenario	Critical Duration Future Scenario
1%	3 hours	1 hour
10%	2 hours	1 hour
20%	2 hours	1 hour

Critical storm durations documented in the previous studies – Shoal Bay Drainage Study (PSC, 2009) and Shoal Bay Infiltration Investigation (PSC, 2013) – are reasonably consistent with these results.

4.1.2.3 Infiltration

Infiltration of rainfall and runoff in Shoal Bay is critical to the flow characteristics in the urban area. Therefore, applying the correct infiltration values to the correct areas of the 2D model is essential to replicating stormwater behaviour in reality.

Port Stephens Council does not have access to the previous Geotechnical Investigation (Douglas Partners, 2005) so it was not able to be directly referred to for infiltration values. Infiltration values can only be assumed based on the previous DRAINS modelling. **Table 4-2** details the different infiltration values found in the modelling undertaken in the previous studies along with a description of where the values apply.

Table 4-2 Infiltration Rates from Previous Modelling

Year of Study	Infiltration Value (mm/hr)	Type of Infiltration Model	Location	Notes
2009	150 – 250	Horton	Catchment wide	Applied to hydrological model
	190 – 200	Constant	Proposed infiltration pipes	No obvious difference between high and low areas of catchment
	50	Constant	Passchendaele Park	Infiltration only applied to basin walls
2013	6 – 125 *	Horton	Catchment wide	Applied to hydrological model
	300	Constant	Proposed infiltration pipes and basins	Applied only on high areas of catchment
	50	Constant	Passchendaele Park	Infiltration only applied to basin walls

* *It should be noted that catchment wide infiltration values from the 2013 study were for a DRAINS soil type 3 which are relatively low and not representative of actual infiltration from rainfall. When modifying this 1D model for 2D model calibration, Cardno has increased the infiltration to represent DRAINS soil type 2, 13 – 180 mm/hr.*

These values will be used as a basis to quantify infiltration across the study area.

2D modelling in XPSWMM allows infiltration to be applied to different soil types, varying spatially. Within the Shoal Bay study area, high infiltration rates are observed in the areas corresponding to higher elevations. Conversely, relatively lower infiltration rates are found in areas lower in the study area. This is caused by an elevated water table restricting the volume of rainfall able to infiltrate into the soil.

The Geological Survey of New South Wales identifies three distinctive geological areas within the Shoal Bay study area (refer **Section 3.4.2**). These areas roughly correspond to elevations in the study area and, by inference, surface infiltration rates. The exception to this is the high elevations of the hills and mountains in the eastern extent of the study area within Tomaree National Park. These are steep, rocky slopes of volcanic origin and do not allow much infiltration. An additional transition area was added between the Qps and Qph geological zones based on surface elevations to reflect the large difference in infiltration rates documented in the previous DRAINS models.

XPSWMM can apply multiples type of infiltration models to the soil in a 2D model. Cardno has employed the initial infiltration / continuous infiltration model, which allows an initial rainfall depth to infiltrate before rainfall can be converted into runoff. Following this initial occurrence, rainfall infiltrates at a constant rate with excess rainfall producing runoff.

Figure 4-2 and **Table 4-3** shows the adopted infiltration values for the study area and the regions areas they apply to.

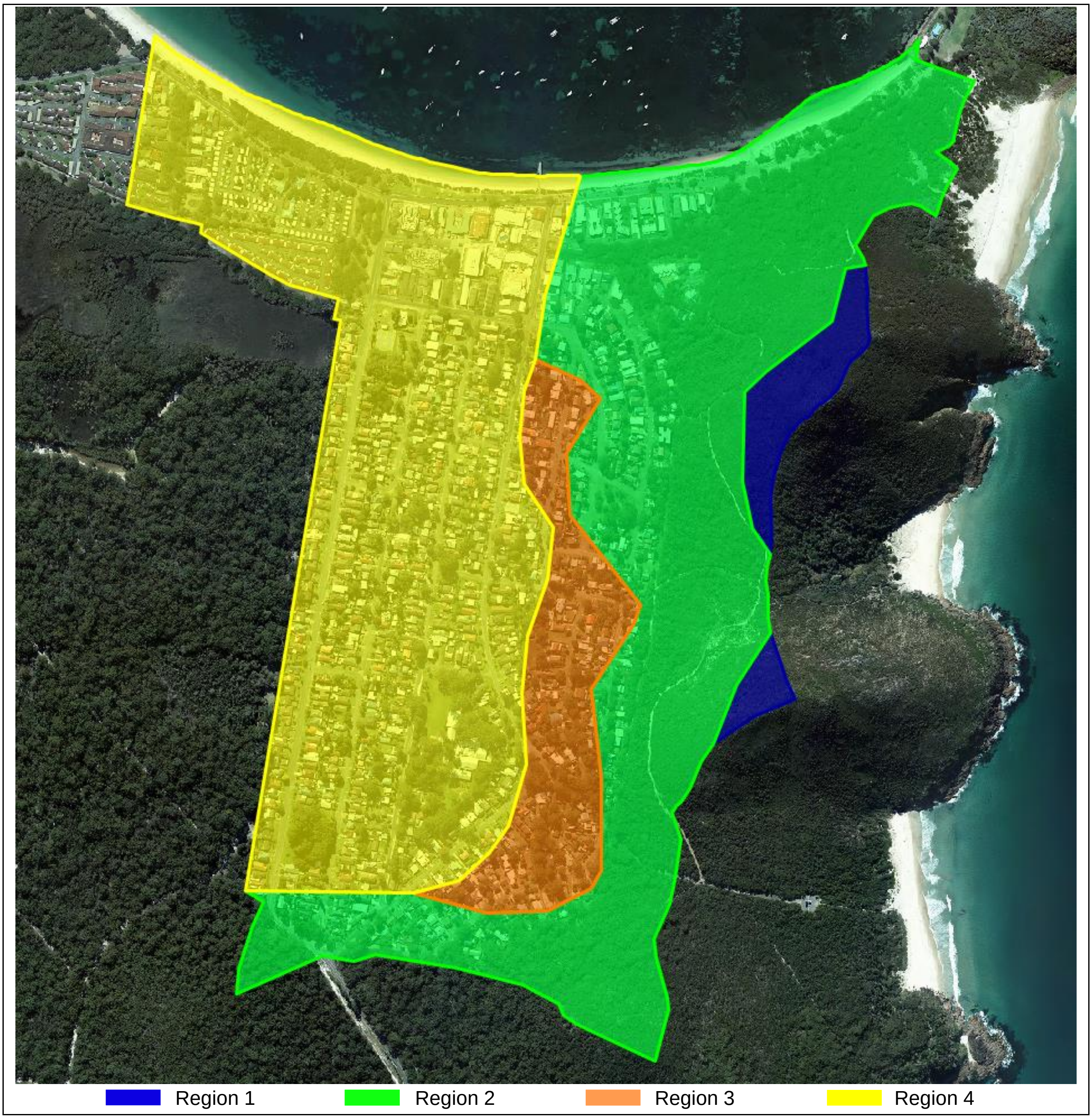


Figure 4-2 Infiltration Regions

Table 4-3 Infiltration Rates

Region	Initial Infiltration (mm)	Continuous Infiltration Rate (mm/hr)
1	10	5
2	10	40
3	7.5	12.5
4	2	7.5

An exception which was applied to the above infiltration strategy is Passchendaele Park where no infiltration has been applied to simulate a permanent water level.

4.1.3 Hydraulics

Hydraulic calculations in XPSWMM are split between the 2D domain (i.e. surface flow) and the 1D domain (i.e. underground flow in pits and pipes). Cardno has carried out modelling and parameter selection consistent with Council's Handbook for Drainage Criteria (2008).

4.1.3.1 1D Domain

4.1.3.1.1 Pits

Pits and headwalls are modelled as nodes in XPSWMM. For clarity, nodes in the model have been named according to the structure ID given in the Council GIS database. The structure type, location and sequence in a stormwater drainage line determine its structure ID. For example, a pit may be called *18saabs002* and headwall called *18saaah001* – the *s* and *h* identifies its structure type, the *saab* and *saaa* identifies the line it is located on and the *002* and *001* identifies it's sequence in the line (lower numbers are further downstream).

Inlet capacities and pressure change coefficients for pits are consistent with Clause D5.10 of Council's Handbook for Drainage Criteria (2008). Where non-standard pits are found in the study area, Cardno has used inlet rating curves based previous modelling experience and engineering judgement.

Blockage factors in the model are also consistent with Clause D5.10 Council's Handbook for Drainage Criteria (2008). These are shown in **Table 4-4**.

Table 4-4 Drainage Design Criteria Blockage Factors

Pit Type	Blockage Factor
On Grade	20%
Sag	50%
Headwall	50%

Sag and on grade pits are not identified in Council's GIS data. These have been determined primarily by contours produced with LiDAR data but also based on interrogation of aerial photography and on-site inspection.

4.1.3.1.2 Pipes

In Council's GIS database, pipes are generally named in accordance with their upstream pit name (e.g. if the pit name is *18saabs002*, the downstream pipe name is *18saabp002*). However, this is not consistent across the entire network within Shoal Bay. Some pipes are named based on their downstream pits. For simplicity, pipes in this model are named according to their upstream pits (e.g. if the pit is named *18saabs002*, the pipe is named *p_18saabs002*). Additionally, XPSWMM has a character limit of ten (10) for each 1D node and link in the network. Given that the first three characters of all nodes in the database are *18s*, these were removed from the naming to allow for downstream pipe naming. The final result, for example, is a pit being named *aabs002* and the downstream pipe being named *p_aabs002*.

All pipes and conduits modelled will be consistent with Clause D5.10 of Council's Handbook for Drainage Criteria (2008). Conduit roughness parameters will be applied according to **Table 4-5**.

Table 4-5 Conduit Roughness

Conduit Type	Manning's n Value
Reinforced Concrete Pipe	0.013
PVC pipe	0.010
Reinforced Concrete Box Culvert	0.015

There have been multiple installations of infiltration pipes within the study area following the previous studies completed by Council (refer **Section 3.5**). This includes the area of Leonard Avenue and the eastern end of Shoal Bay Road.

Designs of the infiltration pipes indicate they are surrounded with gravel backfill to increase the rate of infiltration out of the pipes. Cardno have assumed a constant infiltration rate of 200 mm/hr, consistent with the previous DRAINS modelling (refer **Section 3.6**).

4.1.3.2 2D Domain

4.1.3.2.1 Grid Resolution and Timestep

The DTM created for this project is converted into a DEM of square cells with fixed widths. The size of these cells is a significant factor in the efficiency of model runs and detail of model results. A larger grid cell size typically results in faster run times and less potential for model instabilities and error, but results are not as detailed. Smaller grid cell sizes, conversely, result in longer run times and difficulties creating a model without significant instabilities and error, but results are of a higher level of detail.

Given the size and nature of the study area, Cardno has recommended that a grid size ranging from 5 m x 5 m to 2 m x 2 m is recommended. **Figure 4-3** shows an example of what the aforementioned grid cell sizes look like along Bullecourt Avenue.

5m x 5m Grid Cell Size

3m x 3m Grid Cell Size

2m x 2m Grid Cell Size

Figure 4-3 Gird Cell Size Examples

All of the above examples will be able to adequately determine flow characteristics within the study area, but provided different levels of detail.

The grid orientation has roughly been aligned parallel and perpendicular to Government Road (and subsequently Horace Street, Rigney Street, Messines Street, etc.) as, generally, roads are the main overland flow paths in urban areas and this allows for better definition of these 2D overland flows.

The timestep selected has a major impact on model stability and error, as well as overall model run times. A general rule of thumb is to have the timestep in seconds equal to half to one quarter of the grid cell size in metres.

For this model Cardno has selected a grid cell size of 2m x 2m and a timestep of 0.5 seconds.

4.1.3.2.2 Hydraulic Roughness and Percentage Impervious

Cardno has utilised Council's GIS data to delineate land use zones in the study area. This zoning information is used to apply an appropriate surface roughness coefficient to the grid cells of the DEM. The roughness coefficients selected based on aerial imagery, site inspections, published guidelines, engineering experience and modelling experience.

Care has been exercised when interpreting published roughness values from these references for a 2D modelling application. The roughness values in these texts and similar are generally derived from in-channel flows and based on a 1D interpretation of those flows (Engineers Australia, 2012). Manning's n parameters selected were based on Table 10-1 of AR&R Project 15: Two Dimensional Modelling in Urban and Rural Floodplains.

Clause D5.06 of Council's Handbook for Drainage Criteria (2008) provides a guide for impervious percentages of certain land use zoning. Cardno has utilised this and the available aerial photography to determine the percentage impervious which was applied to each zoning.

Table 4-6 shows the adopted values of hydraulic roughness and impervious percentage for each land use zoning.

Table 4-6 Hydraulic Roughness and Impervious Percentage

Zoning	Manning's n	Percent Impervious (%)
B2 - Local Centre	0.250	100
E1 - National Parks and Nature Reserves	0.100	0
R2 - Low Density Residential	0.100	60
R3 - Medium Density Residential	0.200	75
RE1 - Public Recreation	0.040	0
RE2 - Private Recreation	0.030	30
SP2 - Infrastructure	0.040	20
Roads	0.025	90

Although roads are not considered to be a separate land use zone in the Port Stephens LEP, Cardno has separated these areas based on cadastral information. This will allow for a more accurate representation of flows parallel and perpendicular to roadways and allows for roughness values in residential areas to reflect the abundance of structures and buildings.

4.1.3.2.3 Effect of Buildings and Fences

There is no data available to identify in the study area the locations and sizes of buildings and fences and digitising this information from aerial photography is time consuming for a study area this size. These structures may have a significant effect on flow characteristics in 2D models, as expected from field observations. The Manning's n roughness values from **Table 4-6** have been reflect this approach.

4.1.3.3 Boundary Conditions

4.1.3.3.1 Outlets

The three main outlets to Shoal Bay Beach are subject to the effect of sea level, as defined in the Port Stephens Flood Study (MHL, 1996).

There is an additional headwall outlet at the southern boundary of the model, at the bottom of Ocean Beach Road. This outlet will assume a tailwater level equivalent to the minimum of the upstream conduit's critical and normal depths.

The other way stormwater exits the model is through infiltration (infiltration pipes and basins) described in **Sections 4.1.2.3** and **4.1.3.1.2**.

4.1.3.3.2 Sea Level

Sea levels at Shoal Bay Beach have a significant backwater effect on stormwater trying to discharge from the urban area, thereby limiting the capacity of the stormwater pipes immediately upstream of the three main outlets.

The Port Stephens Flood Study (MHL, 1996) defines sea levels for two locations near the study area: Shoal Bay East and Shoal Bay West. The Shoal Bay East location is closer to the study area and was selected to represent sea level for this study. Sea levels are reported for the 1% AEP, 2% AEP, 5% AEP and extreme events.

The sea levels defined in this study are influenced by many factors, including:

> Astronomical tidal effects;

> Ocean storm surges;

> Wave setup and runup from local and ocean waves;

> Catchment runoff and rainfall directly on Port Stephens, and;

> Bathymetry of Port Stephens.

Given that rainfall on the catchment is not the only factor in determining sea levels (and rainfall on the study area is even less important), Cardno has assumed a 5% AEP sea level of 1.53 m Australian Height Datum (AHD) for the design storm events.

4.2 Calibration

There are no adequate records of flooding from historical storm events for this study area. Therefore Cardno has conducted a model calibration based on the following information, in order of priority:

1. DRAINS model from the 2013 PSC Shoal Bay Infiltration Investigation
2. Anecdotal evidence from community consultation
3. Test catchment comparing rainfall on grid hydrology and traditional hydrological modelling

Cardno has adjusted the following model parameters within the respective upper and lower bounds to achieve a reasonable model calibration:

> Infiltration rates, and;

> Hydraulic roughness.

4.2.1 <u>2013 DRAINS Model</u>

Cardno has used the 2013 PSC Shoal Bay Infiltration Investigation DRAINS model as the main source of quantitative results for model calibration. Calibration attempted to achieve not only maximum results values but also matching the timing of these peak values if possible.

The model calibration locations are shown in **Figure 4-4**.

Figure 4-4 Calibration Locations

Calibration results are shown below in **Table 4-7**, **Table 4-8**, and **Table 4-9**.

Table 4-7 Calibration to 2013 DRAINS Model – 1% AEP

ID	Location	XPSWMM Piped Flow Rate (m^3/s)	DRAINS Piped Flow Rate (m^3/s)	XPSWMM Max Water Level (m AHD)	DRAINS Max Water Level (m AHD)	XPSWMM Time to peak (min)	DRAINS Time to peak (min)
1	Beach outlet west	4.38	3.38			145	60
2	Beach outlet jetty	1.61	0.961			50	50
3	Government Rd 2x900 mm RCP	2.24	2.25			40	165
4	Passchendaele Park basin			7.57	7.41	200	190
5	Box Beach Rd basin			12.12	12.59	45	90
6	Ocean Beach Rd outlet	0.247	0.172			45	50
7	600 mm RCP Culvert under Joleen Cres	0.713	0.387			45	75
8	Joleen Cres U/S basin			30.43	29.29	45	75
9	2x750 mm RCP between Fingal St & Verona Rd	0.964	0.824			170	50

Table 4-8 Calibration to 2013 DRAINS Model – 10% AEP

ID	Location	XPSWMM Piped Flow Rate (m^3/s)	DRAINS Piped Flow Rate (m^3/s)	XPSWMM Max Water Level (m AHD)	DRAINS Max Water Level (m AHD)	XPSWMM Time to peak (min)	DRAINS Time to peak (min)
1	Beach outlet west	3.06	3.26			40	50
2	Beach outlet jetty	1.14	0.839			45	45
3	Government Rd 2x900 mm RCP	2.27	2.24			60	50
4	Passchendaele Park basin			7.12	7.02	125	170
5	Box Beach Rd basin			12.11	12.43	40	110
6	Ocean Beach Rd outlet	0.254	0.150			40	40
7	600 mm RCP Culvert under Jolene Cres	0.294	0.198			45	65
8	Joleen Cres U/S basin			30.35	29.14	45	60
9	2x750 mm RCP between Fingal St & Verona Rd	1.17	0.819			140	50

Table 4-9 Calibration to 2013 DRAINS Model – 20% AEP

ID	Location	XPSWMM Piped Flow Rate (m^3/s)	DRAINS Piped Flow Rate (m^3/s)	XPSWMM Max Water Level (m AHD)	DRAINS Max Water Level (m AHD)	XPSWMM Time to peak (min)	DRAINS Time to peak (min)
1	Beach outlet west	2.99	3.18			80	50
2	Beach outlet jetty	0.850	0.718			40	45
3	Government Rd 2x900 mm RCP	2.27	2.24			70	50
4	Passchendaele Park basin			7.03	6.92	130	170
5	Box Beach Rd basin			12.09	12.32	45	100
6	Ocean Beach Rd outlet	0.218	0.134			45	40
7	600 mm RCP Culvert under Jolene Cres	0.099	0.138			70	70
8	Joleen Cres U/S basin			30.30	29.10	70	70
9	2x750 mm RCP between Fingal St & Verona Rd	1.14	0.791			100	50

Refer **Appendix A** for graphs comparing both models.

The two models could not be calibrated at some significant flooding points within the study area. This includes the northern end of Horace Street, Bullecourt Road, Rigney Street and Messines Street. The 1D DRAINS model loses water from the system at areas where significant flooding occurs (excluding defined basins such as Passchendaele Park, the Box Beach Road basin and upstream of Joleen Crescent) and is not able to provide water level elevation results in these areas.

As it does not model the actual flood water elevations for significantly flooded areas, the DRAINS model also does not take into account accurate pressure head values at the upstream and downstream ends of pipes within the significantly flooded areas. This problem is exacerbated in larger storm events because more water is lost from the system.

Some notes on the calibration results:

> At the large western beach headwall (ID 1), the 1% AEP event results in the XPSWMM model experienced a greater peak discharge and longer duration compared to the DRAINS model because of the effect of higher upstream flood levels at Horace Street and that flood volume requiring a longer time to drain from the system. The effect on peak discharge is not pronounced in the 10% and 20% AEP events.

> The basin created by the embankment of Joleen Crescent (ID 8) across an existing watercourse shows both higher water level results and subsequently higher peak discharge values in the 600 mm RCP basin outlet (ID 7) in the XPSWMM model. This is due to the nature of grid cell storage in 2D modelling compared to depth-area ratings curves used in 1D basin modelling. While some storage volume might be lost or gained in 2D modelling (the degree of this depends on the grid size used), defining the storage areas and depths is taken directly from the DEM and not subject to estimation. Additionally, as the basin catchment is entirely rural, the higher infiltration values used in the DRAINS model produced lower runoff into the basin. The results of each model at this basin has a larger effect on all of the downstream results including the discharge at the beach jetty outlet (ID 2). This is applicable to all AEP events.

> The peak flow results within the 2 x 900 mm RCP on Government Road (ID 3) in the XPSWMM model display similar maximum values and times to peak. The difference between the two hydrographs, for all

AEP events, is the duration of flow before the pipe discharge begins to fall. The XPSWMM model has a greater flow duration because it does not lose flood water from the system upstream at Horace Street.

> The minor differences in the flood water levels at Passchendaele Park and the basin at Box Beach Road are caused by the separate methods for definitions of basins in 2D and 1D modelling, as described above. Additionally, the basin at Box Beach Road has an overflow level of 12.4 m AHD in the DRAINS model; far above the adjacent low point along Ocean Beach Road (approximately 12.0 m AHD) resulting in higher modelled basin water levels.

> Both the peak discharge and time to peak are significantly greater in the XPSWMM model for all AEP events. This is caused by the upstream direct hydraulic connection of the pipe system connected to Passchendaele Park and the 900 mm RCP at the southern end of Rigney Street which alleviates flooding in this area. In XPSWMM the direct connection is made similar to how the pipes exist in reality (with two outlet pipes for one pit) where in DRAINS an overland flow path is used to connect the two systems resulting in only surcharged water being diverted along the stormwater pipes in Rigney Street. Additionally, at the local depression where water ponds between Fingal Street and Verona Road the DRAINS model loses water from the model.

4.2.2 Community Consultation

Good anecdotal evidence for most flood modelling calibration can usually be gathered from the people who witness flooding events, and the case at Shoal Bay is no different. During the first community workshop, the public were invited to provide Council and Cardno with their complaints and evidence of flooding in the study area. Numerous individual accounts of flooding on private and public property, both qualitative and quantitative, were identified during the workshop and recorded for comparison against the completed model.

A relatively accurate recreation of community identified flooding in the flood model provides both confidence of model accuracy for the modellers and trust of the community in the study in general.

Table 4-10 summarises most of the significant flooding experiences of the community reported in the first community workshop. Refer to **Section 5.1** for further information regarding community consultation.

Table 4-10 Community Consultation Verification

Location	Recorded Community Stormwater Issues	Results of XPSWMM Modelling
Horace St North	Major flooding in low point – pictures received indicate approximately 1 m.	1% AEP: depths up to 1.0 m 10% AEP: depths up to 0.8 m 20% AEP: depths up to 0.7 m
Messines St and Horace St	High velocity flows from Messines St around the corner into Horace St.	In all events, higher water velocities were shown from Messines St south into Rigney St compared Horace St. High water depths at Horace St reduced velocities of approaching runoff.
Government Rd and Shoal Bay Rd	Flooding around building approximately 200mm.	DEM has building removed showing large hole in place of building. Private drains removing water are not modelled. 1% AEP: depths up to 1.3 m 10% AEP: depths up to 1.0 m 20% AEP: depths up to 0.8 m
Lillian St	Debris flowing from hillside to the south into property.	High velocities coming off hillside in all AEP events, and increasing along Lillian St to the northeast.
Rigney St and Edward St	Overland flows through properties to east of intersection – depth approximately 300 mm. Pit at rear of 26 Rigney St surcharges.	1% AEP: depths up to 0.7 m at rear of 36 Rigney 10% AEP: depths up to 0.5 m at rear of 36 Rigney 20% AEP: depths up to 0.4 m at rear of 36 Rigney Pit surcharges in all events.

Location	Recorded Community Stormwater Issues	Results of XPSWMM Modelling
Rigney St and Verona Rd	Water ponds approximately 150 mm in open lot with water running into house to the north.	1% AEP: depths up to 0.6 m in open lot 10% AEP: depths up to 0.5 m in open lot 20% AEP: depths up to 0.4 m in open lot Water generally flows east to west. Some flow goes south across Verona Rd at low point, some flows go north and cross Rigney St approximately 100 m north.
64 Horace St & 79 Government Rd	Water frequently passing through the surrounding properties.	1% AEP: depths up to 0.8 m 10% AEP: depths up to 0.6 m 20% AEP: depths up to 0.4 m There is a low point in the DEM at 75 Government Rd which appears to be from the house constructed in 2010. Surrounds are still a low point excluding this lot. Depths reported above represent surrounding depths, not the depth in the house excavation.
21 Flannel Flower Fairway	Water flows between 21 and 23, across roadway and into properties.	High velocity flows between houses and north along road gutter. Model limited in that it cannot replicate flows in the small concrete channel between houses.
13 Leonard Ave & 56 Tomaree Rd	Flooding experienced around nearby houses.	1% AEP: depths up to 0.2 m 10% AEP: depths up to 0.2 m 20% AEP: depths up to 0.1 m Greater flooding depths and water velocities in properties to the south.
Ronald Ave & Joleen Cres	Overland flows flowing across Ronald Ave from Joleen Cres and through properties.	Water mostly flows northwest along Ronald Ave with some crossing Ronald Ave in all AEP events. Minor flood depths in front of houses across the street – approximately 0.1 m.
36 Rigney St	Pit at road surcharges. Water in street approximately 300 mm deep. Nearby homes subject to flooding.	1% AEP: depths up to 0.3 m in property and pit not surcharging 10% AEP: depths up to 0.3 m in property and pit not surcharging 20% AEP: depths up to 0.2 m in property and pit not surcharging More significant flooding north of this property all the way downstream to major flooding at Horace St. Surcharging likely to have different cause (e.g. downstream blockage).

Some notes on the community reporting flooding and the model results:

> Generally, good verification was achieved in the model

> It is not very likely that the debris coming downhill from Joleen Crescent to the properties on Lilian Street was caused by stormwater runoff. The more probable cause of this is land slippage from the hillside. The established vegetation here reduces the risk of erosion from stormwater runoff, especially to the degree witnessed by the community.

4.2.3 Test Catchments

The rainfall on grid approach to hydrology is a relatively new method for determining catchment and subcatchment flows as well as rainfall lost to infiltration. Subsequently, AR&R Project 15: Two Dimensional Modelling in Urban and Rural Floodplains recommends setting up individual catchments to test the results of the rainfall on grid method against hydrology methods widely accepted across Australia. The method selected are:

> Probabilistic Rational Method;

> Laurenson Method (utilised in the XPRAFTS modelling program), and;

> Time-Area Method (utilised in the DRAINS modelling program).

Where appropriate, the time of concentration method used is the kinematic method. This is considered to be most appropriate and accurate for this type of study area as it can allow for both urban and rural catchment types.

The test catchments were generally selected in accordance with the following criteria:

> Easily defined;

> Large enough to significantly contribute to runoff in the study area;

> Individually cover a single infiltration region;

> Represent steep and flat areas, and;

> Represent urban and rural catchments.

Figure 4-5 shows the four catchments used to test the rainfall on grid hydrology method and **Table 4-11** shows the results compared to other hydrological methods.

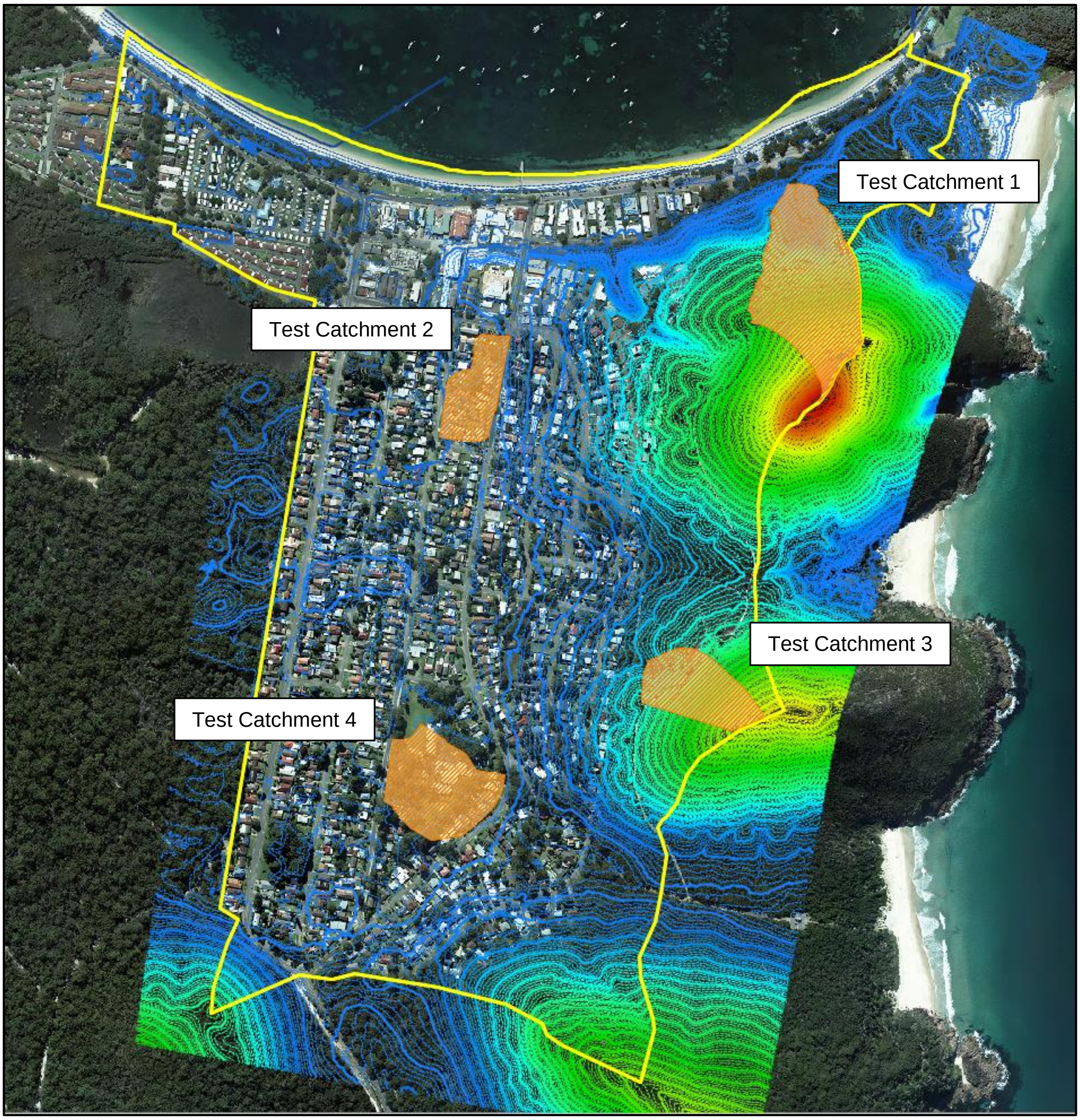


Figure 4-5 Test Catchment Locations

Table 4-11 Test Catchment Results

Catchment	Area (ha)	Probabilistic Rational Method peak discharge (m^3/s)	Laurenson Method (XPRAFTS) peak discharge (m^3/s)	Time-Area Method (DRAINS) peak discharge (m^3/s)	Rainfall on Grid Method (XPSWMM2D) peak discharge (m^3/s)
1	4.138	0.985	0.799	0.800	0.941
2	1.226	0.630	0.368	0.343	0.277
3	1.815	0.506	0.351	0.435	0.393
4	2.329	0.759	0.608	0.450	0.327

In general, the rainfall on grid method results in peak catchment discharges similar in magnitude to the other hydrological methods modelled.

The two rural catchments exhibit peak discharges from the rainfall on grid method that are consistent with the other methods. The two urban catchments produce lower flows using the rainfall on grid method when compared to the other hydrological methods. The Probabilistic Rational Method does not take into account partial area effects which normally produces a 'two peaked' hydrograph (the first peak for impervious area runoff and the second for pervious area runoff) which explains the higher peak discharge compared to all other methods. The Laurenson Method and the Time-Area Method both assume that impervious areas in developed catchment are directly connected to the pit and pipe system (i.e. downpipes and private drainage are directly connected to the road drainage system); however, in Shoal Bay most roofs and other impervious surfaces discharge directly onto the ground which is better represented in this case by the rainfall on grid method.

4.3 Model Results

Maps illustrating existing conditions model results are found in **Appendix B,** Figures B1 to B9. Note that all results are filtered to only display areas where flooding is greater than 150 mm in depth.

Mapping shows the following results:

> Maximum water elevation;

> Maximum water depth, and;

> Maximum water velocity.

The existing conditions results highlighted a number of localised low points within the study area. Some are restricted to front yard along the existing roadways (e.g. Horace Street, north of Peterie Street) while others are located in back yards and within existing building footprints (e.g. two locations between Siddons Street and Peterie Street, between Fingal Street and Verona Road, between Rigney Street and Horace Street, west of Edward Street). The severity and extent of flooding in both of these areas can be reduced to a certain degree, but cannot be eliminated by works within public property.

4.4 Sensitivity Analysis

To test the sensitivity of the established model, certain significant parameters are adjusted to determine their effect on the model results. The following parameters have been selected for sensitivity analysis:

> Infiltration rates;

> Rainfall, and;

> 2D surface roughness.

Each parameter is adjusted +20% and -20%. This provides an indication of the potential difference in the model compared to an actual similar design rainfall event due to modelling error, environmental changes, incorrect assumptions, etc.

Refer to **Appendix B**, Figures B10 to B15, for mapping results showing water depth differences compared to the existing conditions results for each analysis. Only the 1% AEP storm event was modelled to determine the effect of changing these parameters on model results.

Results illustrate the following observations:

> Changes in the rainfall applied to the study area had the greatest effect on flood depths. A 20% increase in the total volume of rainfall resulted in an increased flood depth of approximately 100 mm along overland flow paths and an approximate maximum 200 mm at low points in the study area. A 20% decrease in total rainfall depth resulted in decreased flood depths of approximately 100 mm along overland flow paths and an approximate maximum of 300 mm at low points in the study area.

> Adjustment of the infiltration rate had less of an impact on flooding depths. Adjusting the infiltration rates of +/- 20% resulted in +/- 100 mm, respectively, in some of the study area low points.

> Changes in 2D surface roughness had very little significant impact on model results – a maximum of +/- 100 mm in a few small areas.

5 Consultation

Cardno undertook consultation with community and key stakeholders over the course of the Study. The purpose of the consultation component of the Study was to:

> Consult with relevant government and stakeholder groups in relation to mitigation measures and benchmarks to be achieved;

> Gain information from local sources within the catchment (primarily affected residents) in relation to the nature and extent of recurring local flooding including the documentation of observations as to the behaviour of flood waters during such events;

> Seek opinions as to appropriate mitigation options and feedback on proposed measures to address the flooding problems.

Consultation was undertaken through the use of letters to stakeholders, letters to the community, two (2) community workshops and an Expression of Interest/Comments register.

5.1 Community

5.1.1 Workshop 1

Cardno convened a community participation workshop in consultation with staff from Port Stephens Council between 4.00pm and 6.00pm on 28 October 2015. The workshop was conducted at the conference room at the Shoal Bay Holiday Park, Shoal Bay.

The workshop targeted local residents, land owners and business personnel. Approximately 30 community members were in attendance.

The purpose of the workshop was to:

> Inform the community of the study;

> Gather community feedback;

> Identify any community concerns;

> Obtain evidence from community members, and;

> Identify any stormwater management ideas.

Cardno staff gave a presentation to attendees outlining the purpose of the study, the purpose of attaining community input, known problem areas and issues, and results and findings from past studies/reports. Attendees were given the opportunity to provide comment and raise issues/concerns following the presentation.

The final phase of the workshop involved separating community members for the purpose of group round table discussions. Attendees were encouraged to identify site specific areas of concern on A1 maps and record any problem areas.

Community identified problem areas were identified as:

> Low points in Rigney Road and adjacent properties;

> Passchendaele Park;

> Corner of Leonard Avenue and Messines Street;

> Bullecourt Street;

> Government Road from Shoal Bay Road to Messines Street;

> Land slippage at Joleen Crescent;

> Water flows (velocities and volumes) from higher land to low points, particularly in Horace Street and Rigney Road;

> Lack of kerb and guttering at Horace Street, and;

> Bushland to the west of the study area.

A copy of the minutes of the meeting were sent to all attendees via email or post. A copy of the minutes are found in **Appendix C**.

5.1.2 Workshop 2

Cardno undertook a second community participation workshop on 10 December 2015. The workshop was held between 3.00pm and 5.00pm and was conducted in consultation with staff from Port Stephens Council. The workshop was undertaken at the conference room at the Shoal Bay Holiday Park, Shoal Bay.

Local residents, land owners and business personnel were invited to the workshop. Approximately 13 community members were in attendance.

The purpose of the workshop was to:

> Present results from flood modelling;

> Present stormwater drainage options, and;

> Consider a priority list of works.

The workshop involved a presentation given by Cardno. The presentation gave an overview of the project, findings of Workshop 1, an explanation of the hydrologic and hydraulic modelling prepared by Cardno, results of the modelling, a list of potential stormwater management options and an overview of each option.

Following the presentation, attendees were invited to suggest other options not considered by Cardno that may assist in managing stormwater and flooding in Shoal Bay. One option was identified as:

> Construct a trench under Tomaree Road. One of the meeting attendees suggested this in order to intercept the water before flowing down hill and divert it to flow out to the beach. The attendee recommended building a trench and reinstating the road once the works are complete.

A copy of the minutes of the meeting were sent to all attendees via email or post. Additionally, the PowerPoint presentation was sent to all attendees who provided email addresses. For attendees who provided postal address only, a copy of the presentation was sent if requested. A copy of the minutes are found in **Appendix C.**

5.1.3 EOI and Comments Register

Cardno invited community members and interested persons to register their interest in the study and provide any comments. Invitation was sought through letter, advertisement in the local newspaper (Port Stephens Examiner) and notification on Council’s website.

Expression of Interests (EOI) and comments were received via email and phone conversation. Cardno provided a point of contact for all enquiries. Comments generally reflected those received at workshops and often included photos and videos to show particular issues/concerns.

Any persons that registered an interest in the study were provided continued updates throughout the study including a copy of workshop minutes and further notifications of upcoming workshop events. All persons registered will receive notification of the exhibition period of the study.

5.2 Stakeholders

Key stakeholders were identified in consultation with Council. Stakeholders were contacted by letter and email and requested to provide any comment in relation to any requirements to be addressed as part of the study.

An example of the letter sent to stakeholders is provided in **Appendix C.**

Table 5-1 Stakeholder Responses

Company	Contact Name & Position	Response Summary
NSW State Emergency Services		Nil Response
NSW Maritime Park Authority	Alison Collaros – Senior Water Regulation Officer	• Study to incorporate requirements of DPI *Guidelines for Controlled Activities*, • Study to include consideration of water licensing and approval requirements under the *Water Management Act 2000*.
NSW Office of Water		Nil Response
NSW Department of Planning and Environment		Nil Response
NSW Office of Environment and Heritage		Nil Response
NSW Department of Primary Industries (Fisheries Institute)		Nil Response
Hunter Water Corporation	Mr. Malcolm Withers – Senior Developer Services Engineer	• Study to quantify the effect of the proposed flood mitigation options on groundwater levels.
Shoal Bay Resident Association		Nil Response

Issues raised by NSW Maritime Park Authority, NSW Office of Water and Hunter Water Corporation are outside of the scope of this study. It is recommended, depending on the outcome of this study, that these be addressed in subsequent design and development stages.

6 Options Assessment

6.1 Overview

The stormwater management objectives of the options investigated are:

> decrease the depth of flooding experienced by residents;

> decrease the duration of flooding experienced by residents

> decrease the frequency of flooding experienced by residents;

> where possible, upgrade the stormwater system to meet the requirements of AR&R with regards to a minor/major stormwater system;

> where possible, restrict flooding to public land, and;

> minimise the cost of option implementation as much as practical.

Each option was agreed upon in consultation with PSC. Consideration was also given to ideas from the Shoal Bay community during consultation workshops.

Proposed options are limited to works within public property.

6.2 Options

6.2.1 <u>Option 1</u>

This option focuses on the implementation of infiltration measures to reduce the volume of runoff flowing down the catchment to problem flooding areas.

Specifically, this option includes:

> Installation of permeable pipes;

> Construction of a storage/infiltration basin at Leslie Street;

> Construction of an underground storage/infiltration device at Garden Place, and;

> Construction of kerb and gutter along roadways directly affected by the above measures which currently do not have kerb and guttering.

Given the nature of this option, the objective is not to infiltrate runoff to a level that would enable the existing stormwater infrastructure to provide a major/minor system level of service (as described in AR&R), but to provide maximum opportunity for infiltration of runoff in the study area.

Ideally, infiltration devices are located higher up in the catchment where the potential infiltration rates are greater and less likely to be influenced by high water tables. However, there are some locations lower in the catchment which are not subject to inundation during the existing 1% AEP storm event and therefore can also accommodate permeable pipe installations. **Figure 6-1** shows the locations of the proposed infiltration devices for this option.

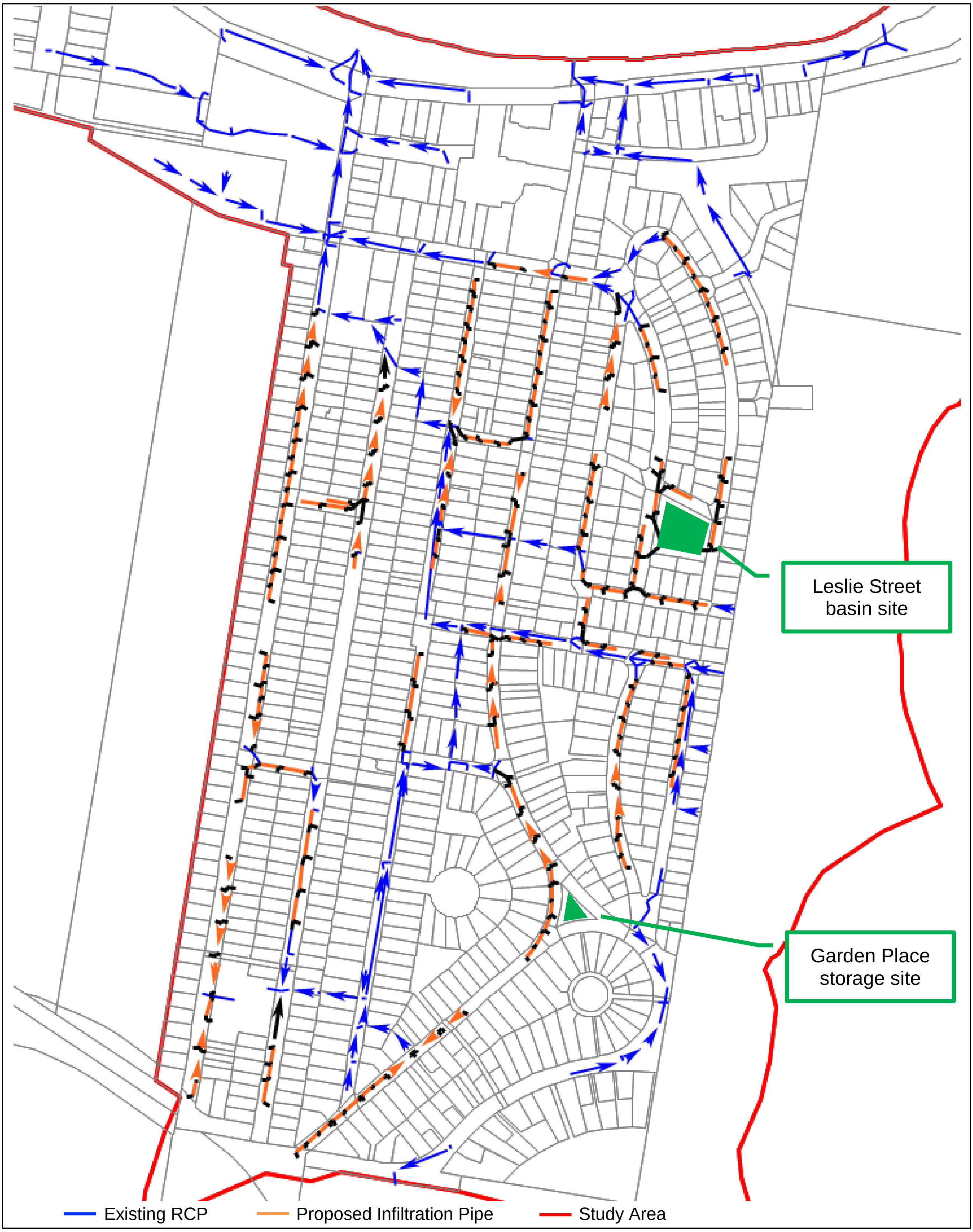


Figure 6-1 Proposed Option 1 Infiltration Device Locations

Permeable pipes within the catchment provide a means for runoff to infiltrate into the groundwater regime before it can contribute to overland flooding. The pipes are installed in a similar way to traditional stormwater pipes beneath roadways and overland flow is transferred to the permeable pipes via kerb inlet pits.

Each permeable pipe is backfilled with single sized stone in a 1.5 m wide trench in order to increase surface area for infiltration and volume of water retained. Additionally, a weir is provided in the downstream pit of each pipe to retain water and allow it to slowly infiltrate into the groundwater. Flows overtopping the weir continue downstream to the next permeable pipe thus giving runoff another chance to infiltrate. This continues until flows reach the existing stormwater piped network. **Figure 6-2** shows the section for a typical permeable pipe with a grated lintel inlet pit.

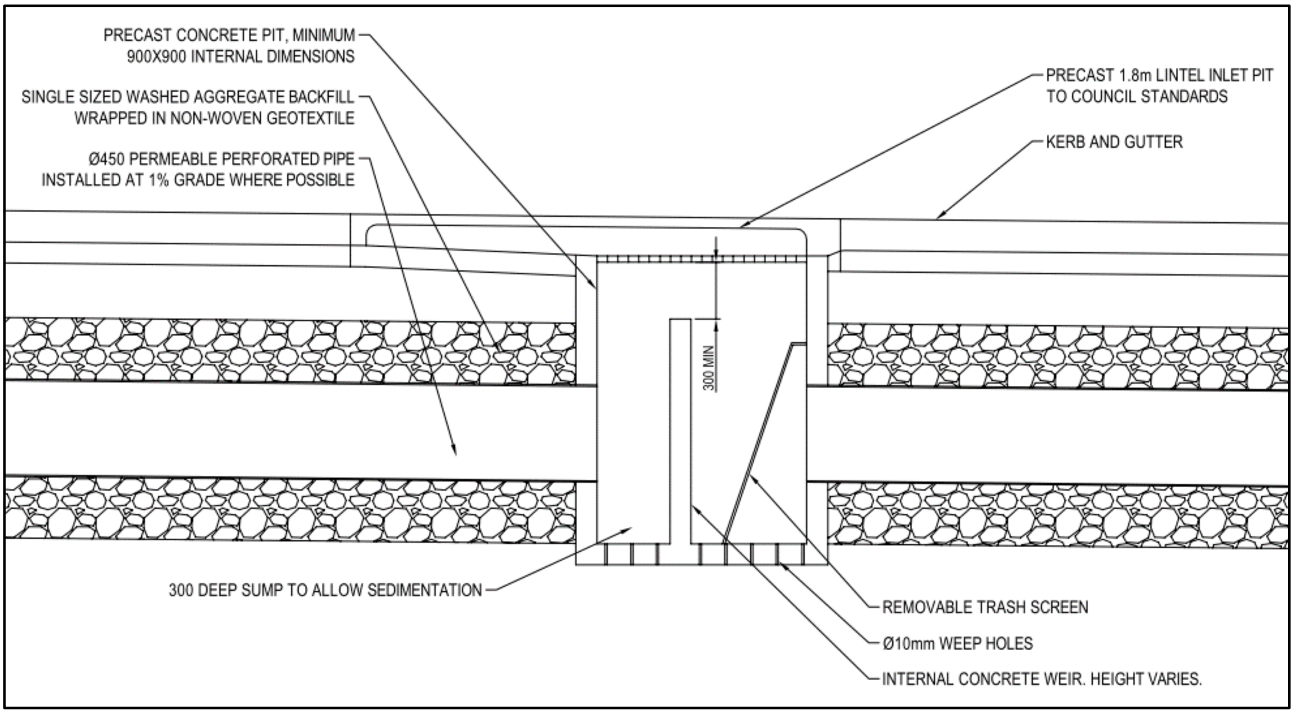


Figure 6-2 Proposed Permeable Pipe Typical Section

Design of the permeable pipes allows a maximum of 1% grade to increase the efficiency of the infiltration system. Although this results in some fairly deep trenches (up to 3 m deep in short sections) and short sections of pipe with relatively short distanced between kerb inlet pits, this allows for permeable pipes to be installed on steep grades located at higher elevations in the study area thereby maximising infiltration.

The proposed infiltration/storage device in the open space at Leslie Street involves the construction of a bund at the lower end of the open space to detain runoff and allow it to drain/infiltrate over a period of time. Water enters the new basin via overland flow in the catchment upstream and headwalls in the open space from the proposed stormwater drainage system in the surrounding streets. At the bottom of the basin is a grated inlet pit to drain the ponded water. **Figure 6-3** shows the location of the park and proposed alignment of the bund. The top of the bund (red line) is proposed to be at 18.4 m AHD, giving the bund a maximum height of approximately 1.5 m.

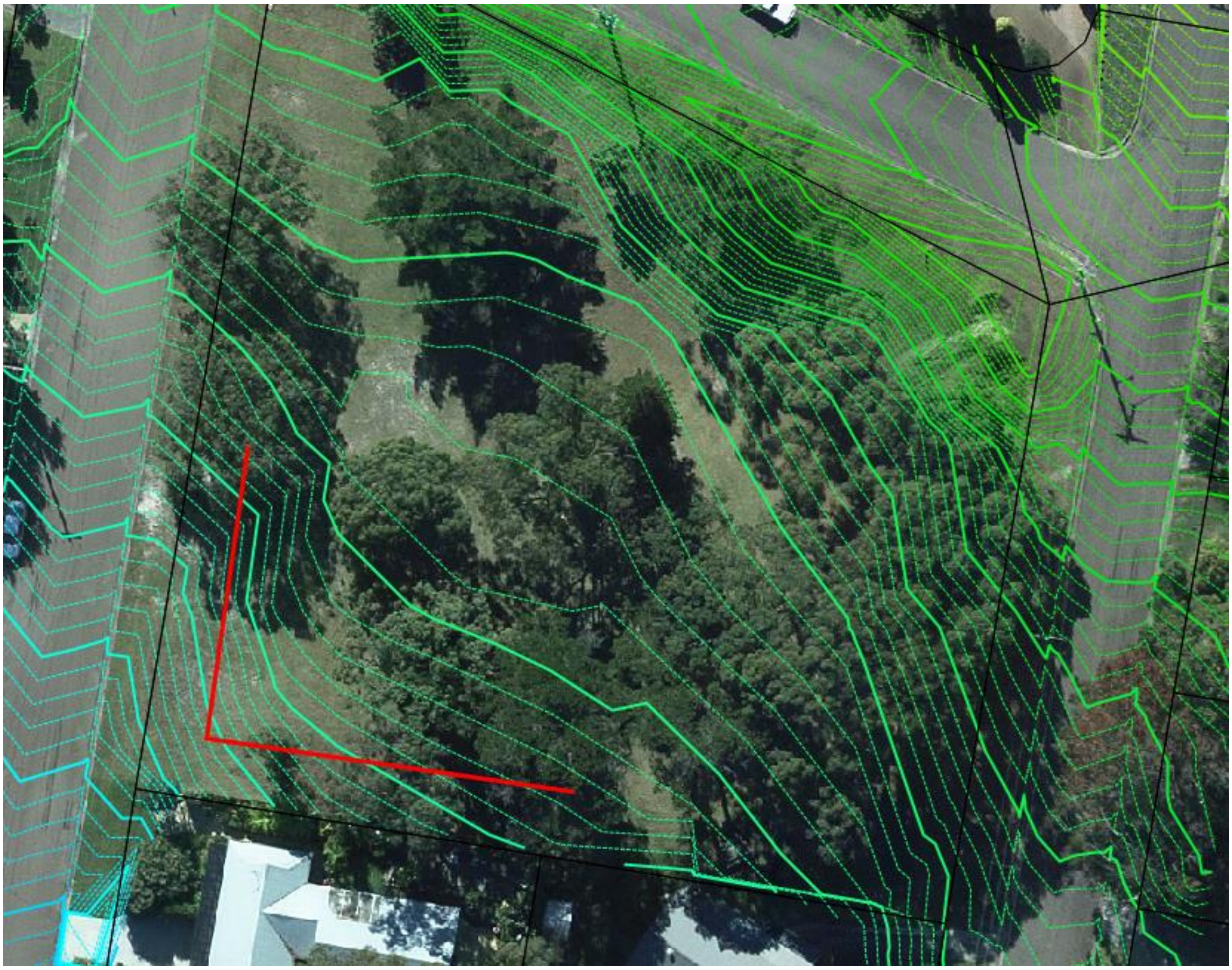


Figure 6-3 Basin at Leslie Street

At Garden Place, it is proposed to construct a storage system which allows runoff entering it to be stored and infiltrate into the groundwater regime. This storage system could take many forms, including an underground tank, set of permeable pipes with gravel backfill, or an above ground basin. For the purposes of this study, Cardno have modelled an underground storage tank with a permeable base to allow for runoff entering the tank to infiltrate. The tank is approximately 100 m^2 and 2.0 m deep. Runoff enters the tank through a grated inlet pit located at the low point of the open space.

A significant length of the roadways in Shoal Bay do not have kerb and gutter and the associated minor stormwater drainage system. Installation of permeable pipes along these roads provides an opportunity to construct kerb and gutter at these locations for a reduced capital cost compared to them being constructed separately. This will also assist with directing as much overland flow as possible to the proposed permeable pipes and maximising infiltration potential in the study area. **Figure 6-4** illustrates the proposed alignments for new kerb and gutter associated with Option 1.

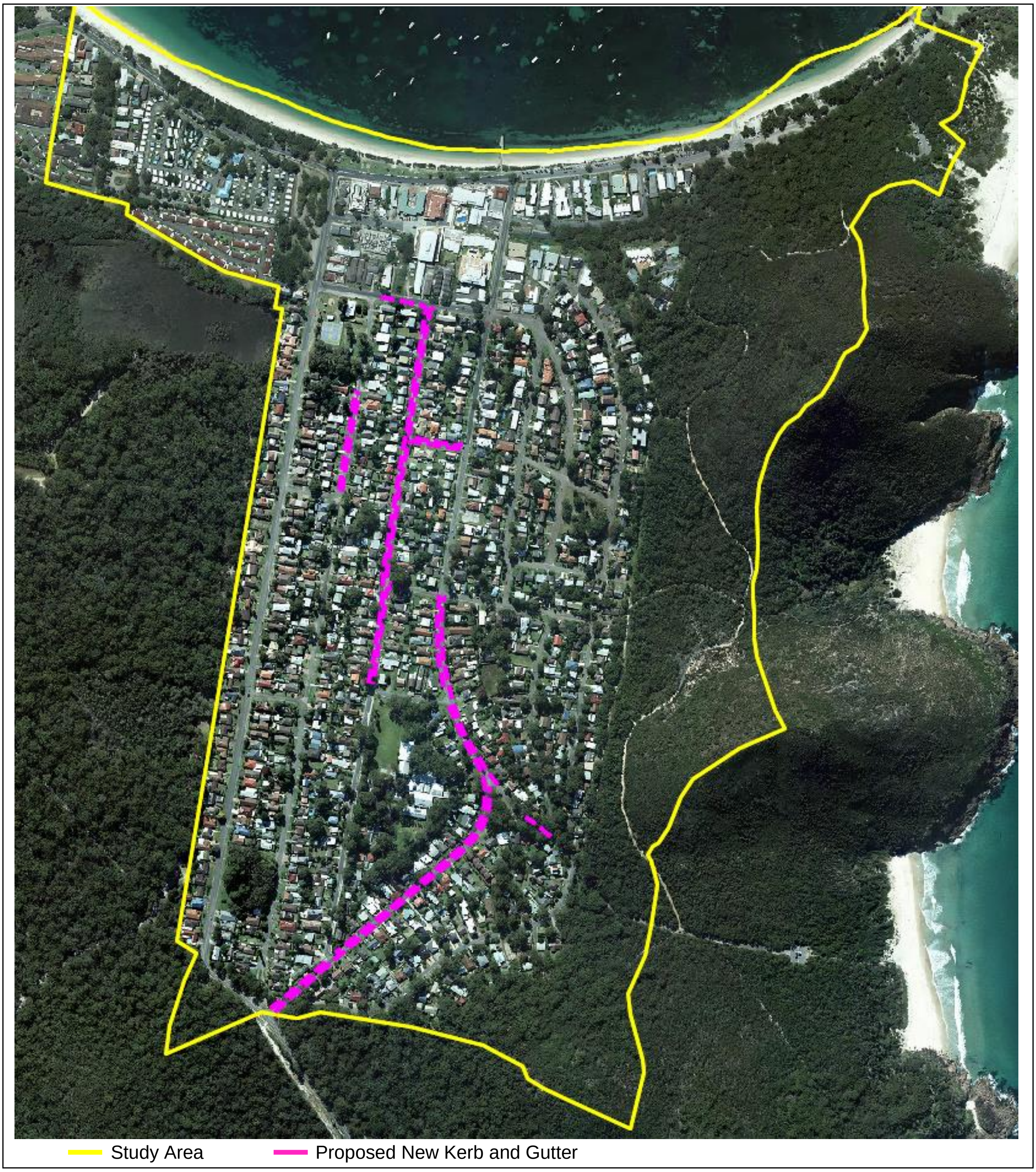


Figure 6-4 Proposed New Kerb and Gutter for Option 1

Kerb and gutter is proposed to be constructed at the following locations:

> Garden Place, near Tomaree Road;

> Tomaree Road, from Marine Drive to Verona Road;

> Rigney Street, from Verona Road to Messines Street;

> Edward Street;

> Horace Street, from Siddons Street to approximately 6 Horace Street, and;

> Messines Street from Rigney Street to Horace Street (south kerb).

These locations represent the best options for construction of new kerb and gutter for Option 1 taking into account the estimated capital costs and effectiveness of controlling stormwater runoff.

6.2.1.1 Advantages and Disadvantages

Option 1 has the following advantages:

> Reduces the volume of flooding in lower elevations of the study area;

> Combines new kerb and gutter with permeable pipes to increase the total kerb and gutter in Shoal Bay;

> Can be implemented over time when budget becomes available;

> No change to infrastructure at Shoal Bay beach;

> Reduces flows and pollutant loads at beach outlets, and;

> Existing drainage system remains untouched.

The following disadvantages are associated with Option 1:

> Will take significant time for complete implementation;

> More effective for short duration and/or intensity rainfall events where the total rainfall volume is relatively small;

> Infiltration potential of permeable pipes and basins will decrease over time as the system suffers from siltation;

> Requires significant maintenance regime for the system to be effective in the long term;

> Some risk for clashes with other underground services, and;

> Will not be able to reduce runoff enough to achieve the objective of having a major/minor drainage system, as described in AR&R.

6.2.1.2 Costs

The overall cost of this option takes into account both the capital costs and the maintenance cost associated with the proposed infrastructure over a period of 50 years.

A summary of assumptions and costs for Option 1 are as follows:

> Capital cost = $5.12 M

> Maintenance cost for first year = $30,000

> Assumed increased cost of maintenance per year = 3%

> Assumed interest rate for Net Present Value = 5%

> Life cycle of permeable pipes = 50 years

> Life cycle of reinforced concrete pipes (RCP) = 100 years

> Period of analysis = 50 years

The net present value for Option 1 is $6.10 M. This cost assumes that all capital cost works are completed in the first year.

Costing details of this option can be found in **Appendix D**.

6.2.1.3 Flood Results

Refer to **Appendix B**, Figures B16 to B24, for maps showing the impact Option 1 has on flooding in Shoal Bay.

It should be noted that contrary to **Table 4-1**, the critical storm duration modelled for this option is the same duration as the existing conditions scenario (i.e. 1% AEP 3 hours, 10% AEP 2 hours, and 20% AEP 2 hours). This duration is selected because there are no changes to the capacity of the outlets at Shoal Bay beach and the critical storm duration is still dictated by the volume of flood waters in the significantly flooded locations within the study area (e.g. the north end of Horace Street, Passchendaele Park, etc).

The following are notes of significance on the results for areas of major flooding:

> Flooding at the low point at the north end of Horace Street was reduced by up to 200 mm in the 1% AEP, 300 mm in the 10% AEP, and 300 mm in the 20% AEP. In the 1% AEP event there is still significant flooding to private property, although is it a lesser extent compared to the exiting scenario. In both the 10% and 20% AEP events, flooding to private property was not eliminated, but the depth and extent were significantly reduced compared to the existing scenario.

> Flooding on Bullecourt Street was reduced up to 500 mm in the 1% AEP event resulting in a maximum depth of 500 mm. Flooding in the 10% and 20% AEP was reduced to a maximum of 300 mm in Bullecourt Street. In all AEP events, there was still flooding in the private properties to the south of Bullecourt Street as it is lower than the road.

> Flooding between Fingal Street and Verona Road was reduced between 100 mm and 200 mm for all AEP events. Maximum depths remain at 800 mm, 600 mm and 500 mm in the 1%, 10% and 20% AEP events, respectively.

> Flooding in Passchendaele Park and the surrounding properties, including up to Rigney Street, were reduced between 200 mm (1% and 10% AEP) and 300 mm (20% AEP).

6.2.2 Option 1A

Similar to Option 1, this option also focuses on the implementation of infiltration measures to reduce the volume of runoff flowing down the catchment to problem flooding areas.

Specifically, this option includes:

> Installation of permeable pipes;

> Construction of a storage/infiltration basin at Leslie Street;

> Construction of an underground storage/infiltration device at Garden Place

> Construction of a storage/infiltration basin at the north end of Horace Street, immediately south of the tennis courts;

> Upgrading stormwater pipes at Bullecourt Street and realigning their discharge directly to Shoal Bay Beach (they currently discharge to the main trunk line in Government Road), and;

> Construction of kerb and gutter along roadways directly affected by the above measures which currently do not have kerb and guttering.

Figure 6-5 shows the proposed works associated with Option 1A.

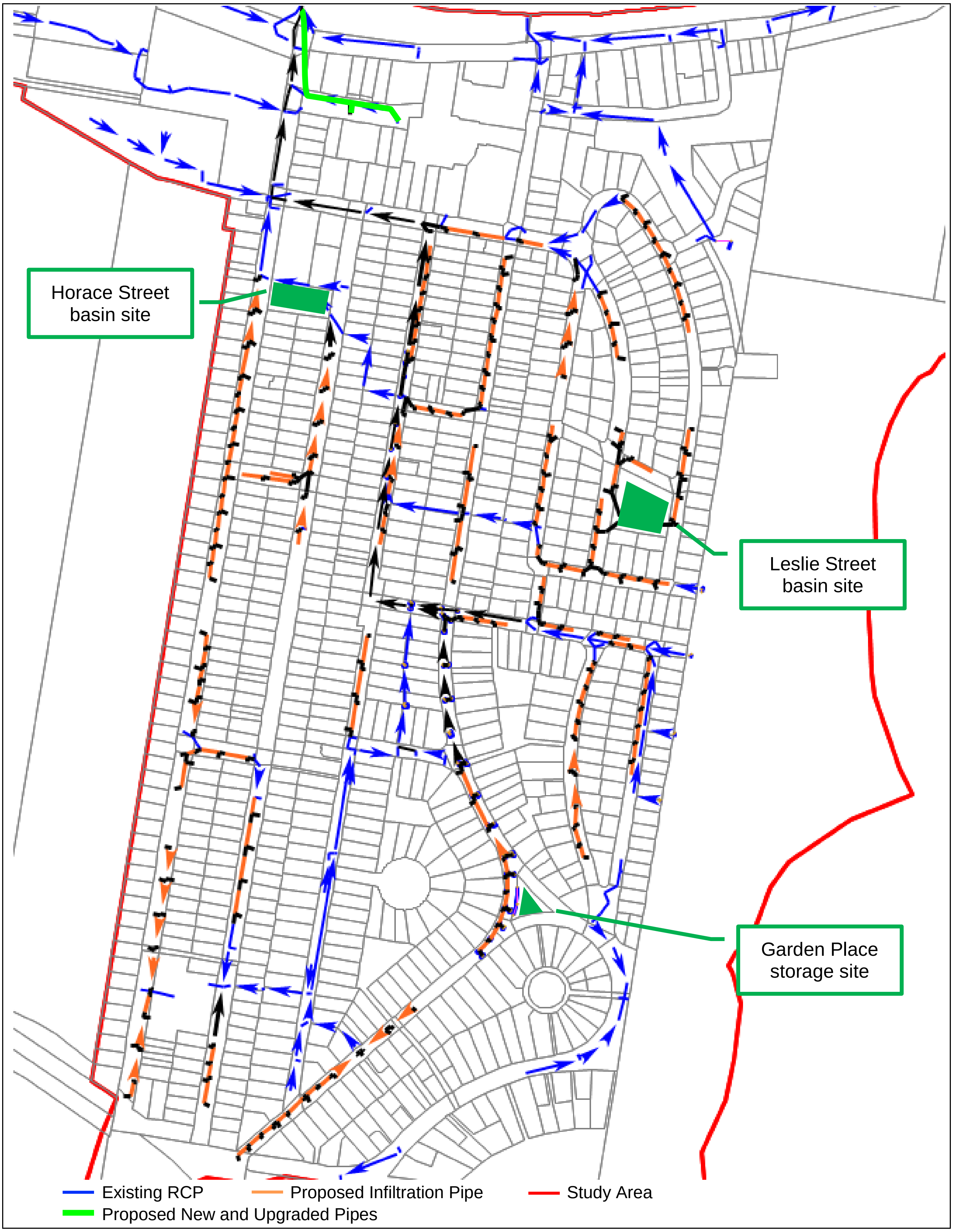


Figure 6-5 Proposed Option 1A Infiltration Devices and Stormwater Upgrades

This option includes the same proposed works from Option 1:

> Permeable pipe installations (refer **Figure 6-2**);

> A basin at Leslie Street open space (refer **Figure 6-3**);

> A storage and infiltration device at Garden Place, and;

> New kerb and gutter along roadways with new stormwater pipes (refer **Figure 6-4**).

The proposed basin at the north end of Horace Street involves the lowering of ground levels in the public space between Horace Street and Government Road, south of the tennis courts. The bottom of the proposed basin is 2.5 m AHD representing an average cut depth of approximately 1.5 m. This extra storage volume will reduce the maximum flood water levels in the immediate surrounds and allow for an additional quantity of water to infiltrate. However, similar to the existing Passchendaele Park, it is expected that infiltration rates will be diminished due to a high water table prior to significant storm events. **Figure 6-6** depicts the extent of the proposed basin at Horace Street.

Figure 6-6 Proposed Horace Street Basin

The proposed pipe upgrades and realignment of the discharge point for Bullecourst Street involves reconstruction of the stormwater pits to larger 2.4 m lintel inlet pits and increasing pipe sizes from 375 mm to 900 mm in diameter. Under existing conditions, during all modelled storm events this pipe network is subject to backwater flows from the main 2x900 mm diameter pipe under Government Road. By realigning the discharge point of the Bullecourt Street stormwater network to Shoal Bay Beach, the backwater pressure is eliminated. The only inflows from Government Road are overland flows along the roadway which the network will be able to capture and drain without excessive ponding. The new outlet to Shoal Bay Beach will be incorporated into the existing headwall and is not expected to have a significant impact in terms of erosion or visual amenity.

6.2.2.1 Advantages and Disadvantages

Option 1A has the following advantages:

> Reduces the volume of flooding in lower elevations of the study area;

> Existing flooding issues along Bullecourt Street are eliminated;

> Combines new kerb and gutter with permeable pipes to increase the total kerb and gutter in Shoal Bay;

> Can be implemented over time when budget becomes available;
> Minor change to infrastructure at Shoal Bay beach;
> Reduces flows and pollutant loads at beach outlets, and;
> Majority of existing drainage system remains untouched.

The following disadvantages are associated with Option 1:

> Will take significant time for complete implementation;
> More effective for short duration and/or intensity rainfall events where the total rainfall volume is relatively small;
> Infiltration potential of permeable pipes and basins will decrease over time as the system suffers from siltation;
> Requires significant maintenance regime for the system to be effective in the long term;
> Some risk for clashes with other underground services, and;
> Will not be able to reduce runoff enough to achieve the objective of having a major/minor drainage system, as described in AR&R.

6.2.2.2 Costs

The overall cost of this option takes into account both the capital costs and the maintenance cost associated with the proposed infrastructure over a period of 50 years.

A summary of assumptions and costs for Option 1A are as follows:

> Capital cost = $5.29 M
> Maintenance cost for first year = $30,000
> Assumed increased cost of maintenance per year = 3%
> Assumed interest rate for Net Present Value = 5%
> Life cycle of permeable pipes = 50 years
> Life cycle of reinforced concrete pipes (RCP) = 100 years
> Period of analysis = 50 years

The net present value for Option 1A is $6.27 M. This cost assumes that all capital cost works are completed in the first year.

Costing details of this option can be found in **Appendix D**.

6.2.2.3 Flood Results

Refer to **Appendix B**, Figures B25 to B33, for maps showing the impact Option 1A has on flooding in Shoal Bay.

It should be noted that contrary to **Table 4-1**, the critical storm duration modelled for this option is the same duration as the existing conditions scenario (i.e. 1% AEP 3 hours, 10% AEP 2 hours, and 20% AEP 2 hours). This duration is selected because there are no major changes to the capacity of the outlets at Shoal Bay beach and the critical storm duration is still dictated by the volume of flood waters in the significantly flooded locations within the study area (e.g. the north end of Horace Street, Passchendaele Park, etc).

The following are notes of significance on the results for areas of major flooding:

> Flooding at the low point at the north end of Horace Street was reduced by 200 mm in the 1% AEP, 300 mm in the 10% AEP, and 400 mm in the 20% AEP. In the 1% AEP event there is still significant flooding to private property, although is it a slightly lesser extent compared to the exiting scenario. In both the 10% and 20% AEP events, flooding to private property was not eliminated, but the depth and extent were significantly reduced compared to the existing scenario.

> Flooding was removed from Bullecourt Street in all AEP events; however, there was still flooding in the private properties to the south of Bullecourt Street as it is lower than the road.

> Flooding between Fingal Street and Verona Road was reduced between 100 mm and 200 mm for all AEP events. Maximum depths remain at 800 mm for the 1% AEP event, 600 mm for the 10% AEP event, and 500 mm for the 20% AEP.

> Flooding in Passchendaele Park and the surrounding properties (including up to Rigney Street) were reduced between 200 mm and 300 mm for all AEP events.

6.2.3 Option 2

This option primarily consists of upgrading the main stormwater trunk drainage line affecting significantly flooded locations. This objective of this is to increase pipe sizes enough to allow for the 1% AEP flows to be safely conveyed in a major stormwater network and not flow overland through private properties.

All of the mitigation measures included in this option are:

> Stormwater pipe capacity upgrade from the beach outlet up to Tomaree Road to convey the 1% AEP event and realignment along Rigney Street and Messines Street;

> Construction of a storage/infiltration basin at Leslie Street;

> Construction of a storage/infiltration basin at the north end of Horace Street, immediately south of the tennis courts;

> Stormwater capacity upgrades at Bullecourt Street, and;

> Construction of kerb and gutter along roadways directly affected by the above measures which currently do not have kerb and guttering.

The extent of trunk stormwater upgrades includes from the Shoal Bay beach headwall upstream to Verona Road, where the majority of significant flooding occurs in the study area. Further upstream, this stormwater line receives overflow from Passchendaele Park. It is proposed that part of this stormwater network will be realigned to Messines Street to avoid surcharging of pits in private property between Horace Street and Rigney Street. **Figure 6-7** outlines the extent of the proposed stormwater underground network upgrades.

Figure 6-7 Proposed Option 2 Stormwater Upgrades

Major details of the proposed stormwater upgrades include:

> Double 1.5 m high x 2.8 m wide RCBC from Shoal Bay beach to approximately Messines Street, then gradually decreasing to a double 1.5 m high x 2.4 m wide RCBC to Edward Street, then a single 1.2 m high x 1.5 m wide RCBC at Verona Road to convey 1% AEP flow from the upstream catchment;

> New outlet structure at Shoal Bay beach and energy dissipator, similar to an RMS ED7 energy dissipator (refer **Appendix E**);

> New kerb and gutter and minor system stormwater drainage along Tomaree Road from Government Road (south) to north of Rigney Street (south) to convey 20% AEP flow;

> New kerb and gutter and major system stormwater drainage along Tomaree Road from Verona Road to Lloyd Lane to convey the 1% AEP flow and reduce flooding through the Shoal Bay Public School;

> Introduce larger inlet pits in rear of lots between Fingal Street and Verona Road to reduce ponding, and;

> Increase the number of inlet pits and stormwater pipes sizes at Bullecourt Street from 375 mm diameter to 450 mm diameter.

Similar to Option 1, this option also involves the construction of a bund in the open space at Leslie Street producing a basin that will detain runoff and allow it to infiltrate (refer **Figure 6-3**) The height of the bund is that same as Option 1 (18.4 m AHD); however, instead of allowing for an outlet pit at the base of the basin, this option has a 5 m wide overflow weir located along the top of the bund at 17.9 m AHD to allow overflow to spill out onto Victor Parade. This is considered a more cost effective solution in this case as there is no underground stormwater infrastructure proposed nearby to convey the basin outflow downstream to the existing stormwater network.

As described in Option 1A, the proposed basin at the north end of Horace Street involves the lowering of ground levels in the public space between Horace Street and Government Road, south of the tennis courts. **Figure 6-6** depicts the extent of the proposed basin at Horace Street.

Upgrading stormwater pipes in roadways presents an opportunity to install kerb and guttering where it will assist efficient road drainage. **Figure 6-8** illustrates the proposed alignments for new kerb and gutter associated with Option 2.

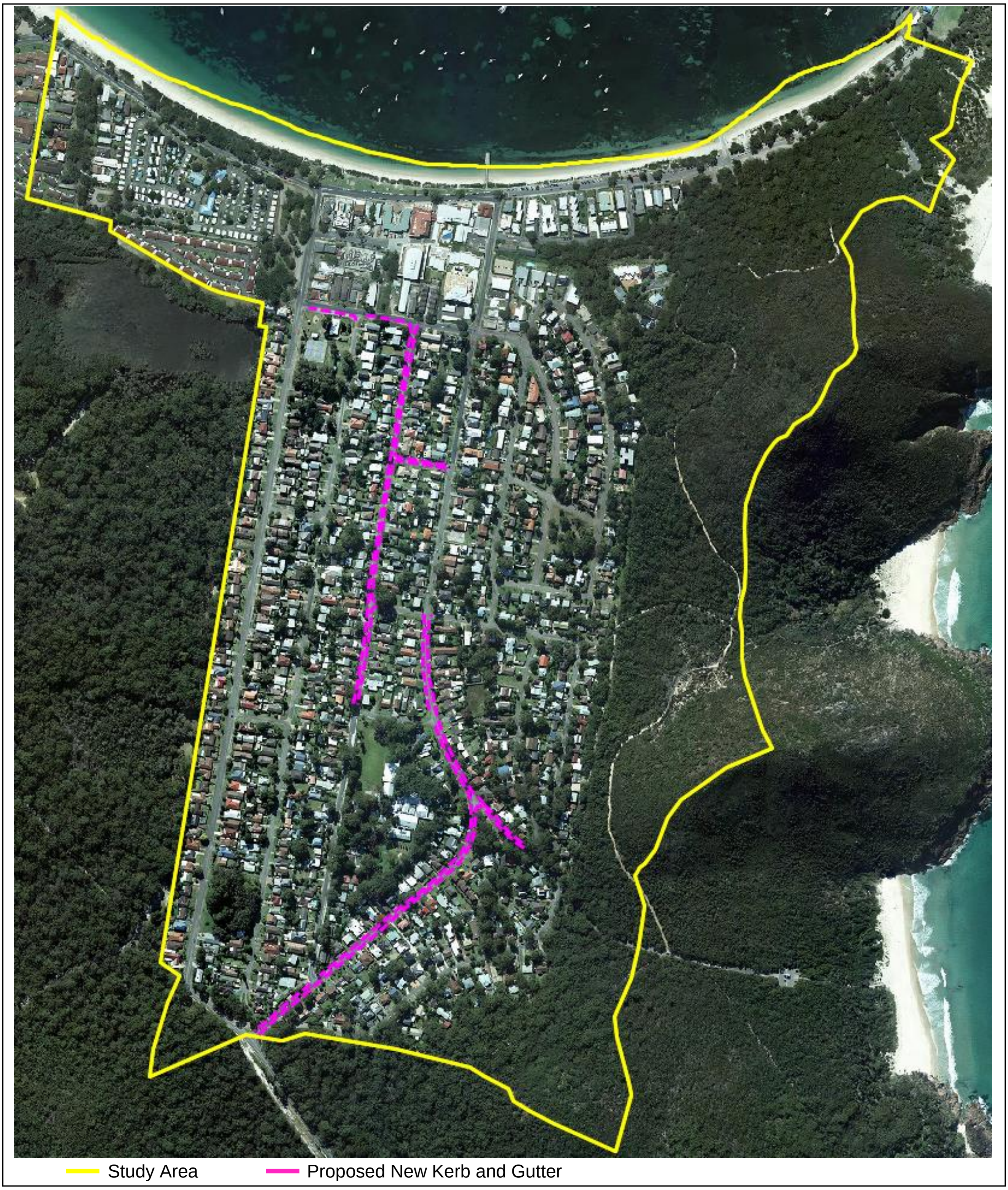


Figure 6-8 Proposed New Kerb and Gutter for Option 2

Kerb and gutter is proposed to be constructed at the following locations:

> Garden Place, between Tomaree Road and Essendene Road;

> Tomaree Road, from Marine Drive to Verona Road;

> Rigney Street, from Verona Road to Messines Street;

> Edward Street, and;

> Messines Street from Rigney Street to Government Road (south kerb).

These locations represent the best options for construction of new kerb and gutter for Option 2 taking into account the estimated capital costs and effectiveness of controlling stormwater runoff. In order to increase the capacity of flood storage on Rigney Street, and prevent overland flow from entering downstream properties, it is proposed to raise the western verge height by approximately 100 mm for a length of 190 m near Edwards Street. Additionally, raising the western verge height along Tomaree Road between Lloyd Lane and Garden Place to 100 mm above the back of kerb for approximately 150 m will prevent overland flows from entering downstream properties.

6.2.3.2 Advantages and Disadvantages

Option 2 has the following advantages:

> Significantly reduces the volume of flooding in lower elevations (and some higher elevations) of the study area for all storm events up to the 1% AEP;

> System is similar to a major/minor drainage network (as described in AR&R) along the upgraded stormwater lines;

> Less frequent maintenance required, and;

> Combines new kerb and gutter with new stormwater pipes to increase the total kerb and gutter in Shoal Bay.

The following disadvantages are associated with Option 2:

> Will take significant time for complete implementation;

> Relatively high capital cost (although lower than the other options modelled);

> Significant disruption from construction along roadways;

> High risk of significant cost increases to realign services affected by stormwater upgrades;

> Major works to Shoal Bay beach outlet and increased environmental impact on beach and receiving waters;

> Reconstruction or replacement of major GPT at the north end of Government Road;

> Minor works are required in private property, and;

> Does not reduce flooding in all areas of the catchment.

6.2.3.3 Costs

The overall cost of this option takes into account both the capital costs and the maintenance cost associated with the proposed infrastructure over a period of 50 years.

A summary of assumptions and costs for Option 2 are as follows:

> Capital cost = $5.86 M

> Maintenance cost for first year = $5,000

> Assumed increased cost of maintenance per year = 3%

> Assumed interest rate for Net Present Value = 5%

> Life cycle of RCP = 100 years

> Period of analysis = 50 years

The net present value for Option 2 is $6.02 M. This cost assumes that all capital cost works are completed in the first year.

Costing details of this option can be found in **Appendix D**.

6.2.3.4 Flood Results

Refer to **Appendix B**, Figure B34 to Figure B42, for maps showing the impact Option 2 has on flooding in Shoal Bay for the 1%, 5% and 10% AEP events.

The following are notes of significance on the results:

> The upgraded stormwater line from Shoal Bay beach to Verona Road conveys the 1% AEP flow from its upstream subcatchments. Rigney Street and Tomree Road effectively become a cut-off for minor event flows from the residential areas of Shoal Bay to be transferred via the underground stormwater conduits to Shoal Bay beach.

> At the northern end of Horace Street where major flooding occurs in the existing conditions scenario, flood depths are reduced significantly by flooding being diverted to the new Horace Street basin. Some flooding remains on the roadway and this has a maximum depth of 300 mm in the 1% AEP event. Roadway flooding is 200 mm in the 10% and 20% AEP events. Additional flooding does remain in private properties but these properties are at elevations lower than the roadway and flooding cannot be reduced any further from works on public land.

> Flooding was effectively eliminated along Bullecourt Street for the 10% and 20% AEP events – 200 mm deep flooding along the south eastern kerb. In the 1% AEP event does flooding occur along the southeast section of the road up to a depth of 300 mm. The private properties to the south of Bullecourt Street are still prone to flooding in all AEP events because their ground elevation is lower than the roadway.

> Flooding was significantly reduced in the private properties between Fingal Street and Verona Road down to a depth of 300 mm for the 10% and 20% AEP events, and 400 mm in the 1% AEP event. This flooding occurred at a localised low point in back yards of private properties which cannot be drained by the stormwater pipe aligned along the rear boundary of the properties.

> Flood waters in and around Passchendaele Park, up to Rigney Street, were reduced from 100 mm to 200 mm for all AEP events. The extent of flooding was reduced to eliminate significant on road flooding of Horace Street in the 10% and 20% AEP events.

> Increasing the proposed stormwater pipes beneath Tomaree Road to convey the 1% AEP event runoff reduces the flooding depth within Shoal Bay Public School by 100 mm to 200 mm and downstream properties along Rigney Street and Horace Street also by 100 mm to 200 mm for all AEP events.

6.2.4 Option 3

This option is similar to Option 2, as it involves the upgrading of stormwater pipes to convey the 1% AEP event flows down to Shoal Bay beach. However, this options introduces a new stormwater line running north along Tomaree Road to increase the flows to the outlet at the beach jetty.

The mitigation measures included in this option are:

> Stormwater pipe capacity upgrade from the beach outlet up to Verona Road to convey the 1% AEP event flow and realignment along Rigney Street and Messines Street;

> New stormwater line to convey the 1% AEP event flow along Tomaree Road from Verona Road down to the jetty outlet;

> Construction of a storage/infiltration basin at Leslie Street;

> Construction of a storage/infiltration basin at the north end of Horace Street, immediately south of the tennis courts;

> Stormwater capacity upgrades at Bullecourt Street, and;

> Construction of kerb and gutter along roadways directly affected by the above measures which currently do not have kerb and guttering.

The extent of trunk stormwater upgrades includes from the Shoal Bay beach headwall upstream to Verona Road, where the majority of significant flooding occurs in the study area. Further upstream, this stormwater line receives overflow from Passchendaele Park. It is proposed that part of this stormwater network will be realigned to Messines Street to avoid surcharging of pits in private property between Horace Street and Rigney Street. The new stormwater line along the length of Tomaree Road from Verona Road to Shoal Bay beach to conveys the upstream subcatchment's 1% AEP event flow and reduces the size of the aforementioned upgraded stormwater line along Government Road, Horace Street and Messines Street. **Figure 6-9** outlines the extent of the proposed stormwater underground network upgrades.

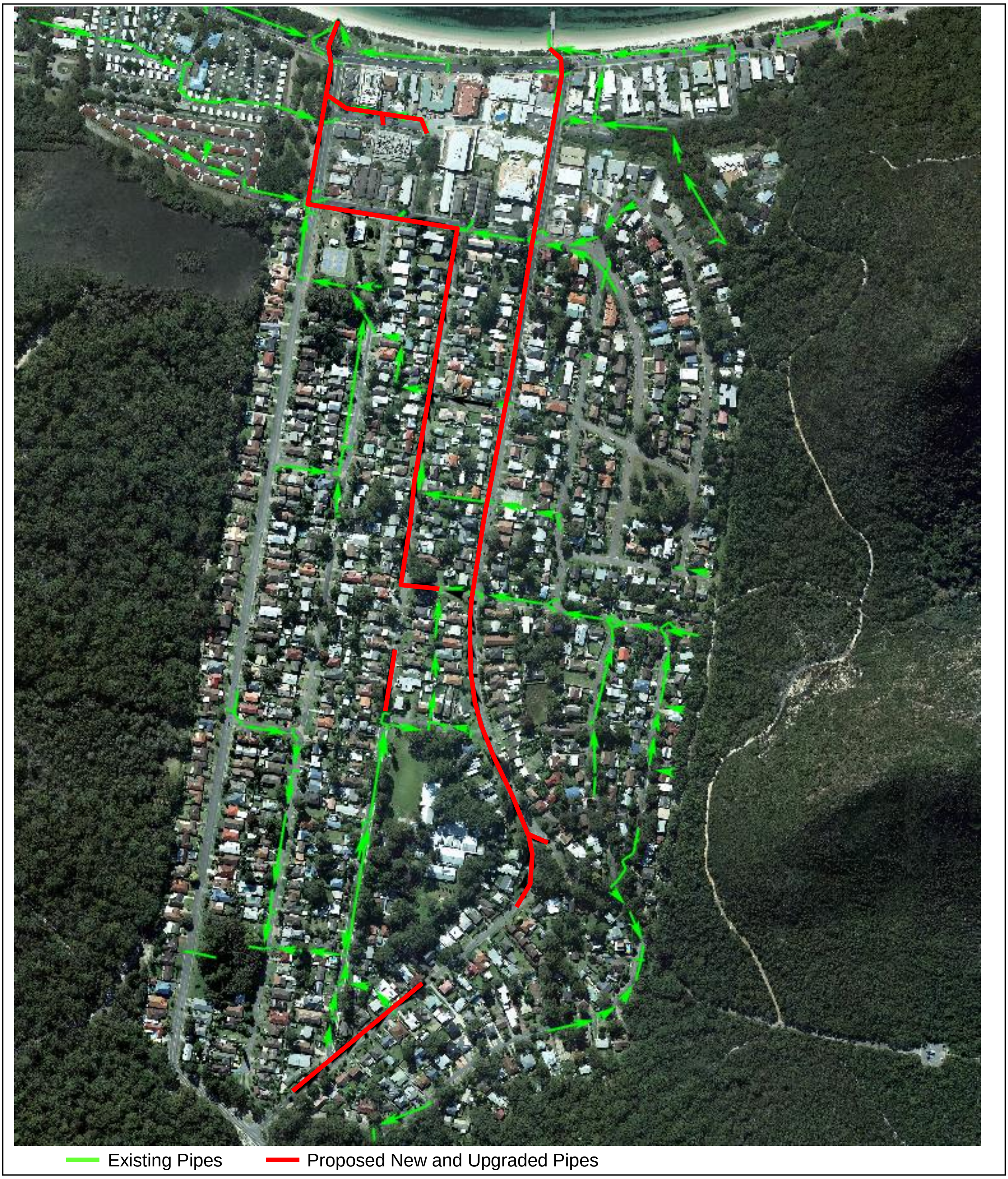


Figure 6-9 Proposed Option 3 Stormwater Upgrades

Major details of the proposed stormwater upgrades include:

> 1.2 m high x 2.1 m wide RCBC from Shoal Bay beach to just south of Edward Street, then decreasing to double 1.35 m diameter RCPs at Verona Road to convey 1% AEP flow from the upstream catchment;

> Double 1.35 m diameter RCPs from the jetty outlet up Tomaree Road to Verona Road to convey the 1% AEP flow;

> New outlet structure at Shoal Bay beach and energy dissipator, similar to an RMS ED7 energy dissipator (refer **Appendix E**);

> On Shoal bay beach, immediately upstream of the outlet jetty, a new surcharge structure is required to replace the existing junction pit to allow surcharging stormwater to flow through an RMS ED3 energy dissipator (refer **Appendix E**) then to the receiving waters;

> New kerb and gutter and minor system stormwater drainage along Tomaree Road from Government Road (south) to Verona Road to convey 20% AEP flow;

> Introduce larger inlet pits in rear of lots between Fingal Street and Verona Road to reduce ponding, and;

> Increase the number of inlet pits and stormwater pipes sizes at Bullecourt Street.

This option includes the same proposed works from Option 2:

> A basin at Leslie Street open space (refer **Figure 6-3**);

> A basin at the northern end of Horace Street (refer **Figure 6-6**), and;

> New kerb and gutter along roadways with new stormwater pipes (refer **Figure 6-8**).

The verge will also require raising along Rigney Street (near Edward Street, similar to Option 2) and Tomaree Road (from Messines Street to Edward Street) by 100 mm to increase gutter flow capacity and avoid overland flows passing into properties on the lower side of the road.

6.2.4.2 Advantages and Disadvantages

Option 3 has the following advantages:

> Significantly reduces the volume of flooding in lower elevations of the study area for all storm events up to the 1% AEP;

> System is similar to a major/minor drainage network (as described in AR&R) along the upgraded stormwater lines;

> Less frequent maintenance required;

> Combines new kerb and gutter with new stormwater pipes to increase the total kerb and gutter in Shoal Bay, and;

> Timeframe for completion can be staggered for the two major stormwater line upgrades.

The following disadvantages are associated with Option 3:

> Will take significant time for complete implementation;

> Relatively high capital cost;

> Significant disruption from construction along roadways;

> Very high risk of significant cost increases to realign services affected by stormwater upgrades;

> Major works to Shoal Bay beach outlets and increased environmental impact on beach and receiving waters;

> Reconstruction or replacement of major GPT at the north end of Government Road;

> Minor works are required in private property;

> Potential unfeasibility of installing stormwater pipe up to 10 m deep in rock along Tomaree Road from south of Messines Street to Lilian Street, and;

> Does not reduce flooding in all areas of the catchment.

6.2.4.3 Costs

The overall cost of this option takes into account both the capital costs and the maintenance cost associated with the proposed infrastructure over a period of 50 years.

A summary of assumptions and costs for Option 3 are as follows:

> Capital cost = $6.52 M
> Maintenance cost for first year = $6,500
> Assumed increased cost of maintenance per year = 3%
> Assumed interest rate for Net Present Value = 5%
> Life cycle of RCP = 100 years
> Period of analysis = 50 years

The net present value for Option 3 is $6.73 M. This cost assumes that all capital cost works are completed in the first year.

Costing details of this option can be found in **Appendix D**.

6.2.4.4 Flood Results

Refer to **Appendix B**, Figure B43 to Figure B51, for maps showing the impact Option 3 has on flooding in Shoal Bay for the 1%, 5% and 10% AEP. Results in existing significantly flooded areas were similar to Option 2 results.

The following are notes of significance on the results:

> The new and upgraded stormwater lines from Shoal Bay Beach to Verona Road conveys the 1% AEP flow from their respective upstream subcatchments. Rigney Street and Tomaree Road (south of Verona Road) effectively become a cut-off for minor event flows from the residential areas of Shoal Bay to be transferred via the underground stormwater conduits to Shoal Bay Beach.

> At the northern end of Horace Street where major flooding occurs in the existing conditions scenario, flood depths are reduced significantly by flooding being diverted to the new Horace Street basin. Some flooding remains on the roadway and this has a maximum depth of 300 mm in the 1% AEP event. Roadway flooding is insignificant in the 10% and 20% AEP events. Some flooding does remain in private properties but these properties are at elevations lower than the roadway and flooding cannot be reduced any further from works on public land.

> Flooding was effectively eliminated along Bullecourt Street for the 10% and 20% AEP events – 200 mm deep flooding along the south eastern kerb. Only in the 1% AEP event does flooding occur along the southeast section of the road up to a depth of 300 mm. The private properties to the south of Bullecourt Street are still prone to flooding in all AEP events because their ground elevation is lower than the roadway.

> Flooding was significantly reduced in the private properties between Fingal Street and Verona Road down to a depth of 300 mm in all AEP events. This flooding occurred at a localised low point in back yards of private properties which cannot be drained by the stormwater pipe aligned along the rear boundary of the properties.

> Flood waters in and around Passchendaele Park, up to Rigney Street, were reduced from 100 mm to 200 mm for all AEP events. The extent of flooding was reduced to eliminate significant on road flooding of Horace Street in the 10% and 20% AEP events.

6.2.5 <u>Option 4</u>

This option combines Option 1 and Option 2. That is, it includes the installation of infiltration pipes throughout the study area and an upgrade of the main stormwater line from the Shoal Bay beach outlet headwall up to Verona Road.

Specifically, this option includes:

> Installation of permeable pipes;
> Stormwater pipe capacity upgrade from the beach outlet up to Verona Road to convey the 1% AEP event and realignment along Rigney Street and Messines Street;

> Construction of a storage/infiltration basin at the north end of Horace Street, immediately south of the tennis courts;

> Construction of a storage/infiltration basin at Leslie Street;

> Construction of an underground storage/infiltration device at Garden Place, and;

> Construction of kerb and gutter along roadways directly affected by the above measures which currently do not have kerb and guttering.

Figure 6-10 shows the proposed works associated with Option 4.

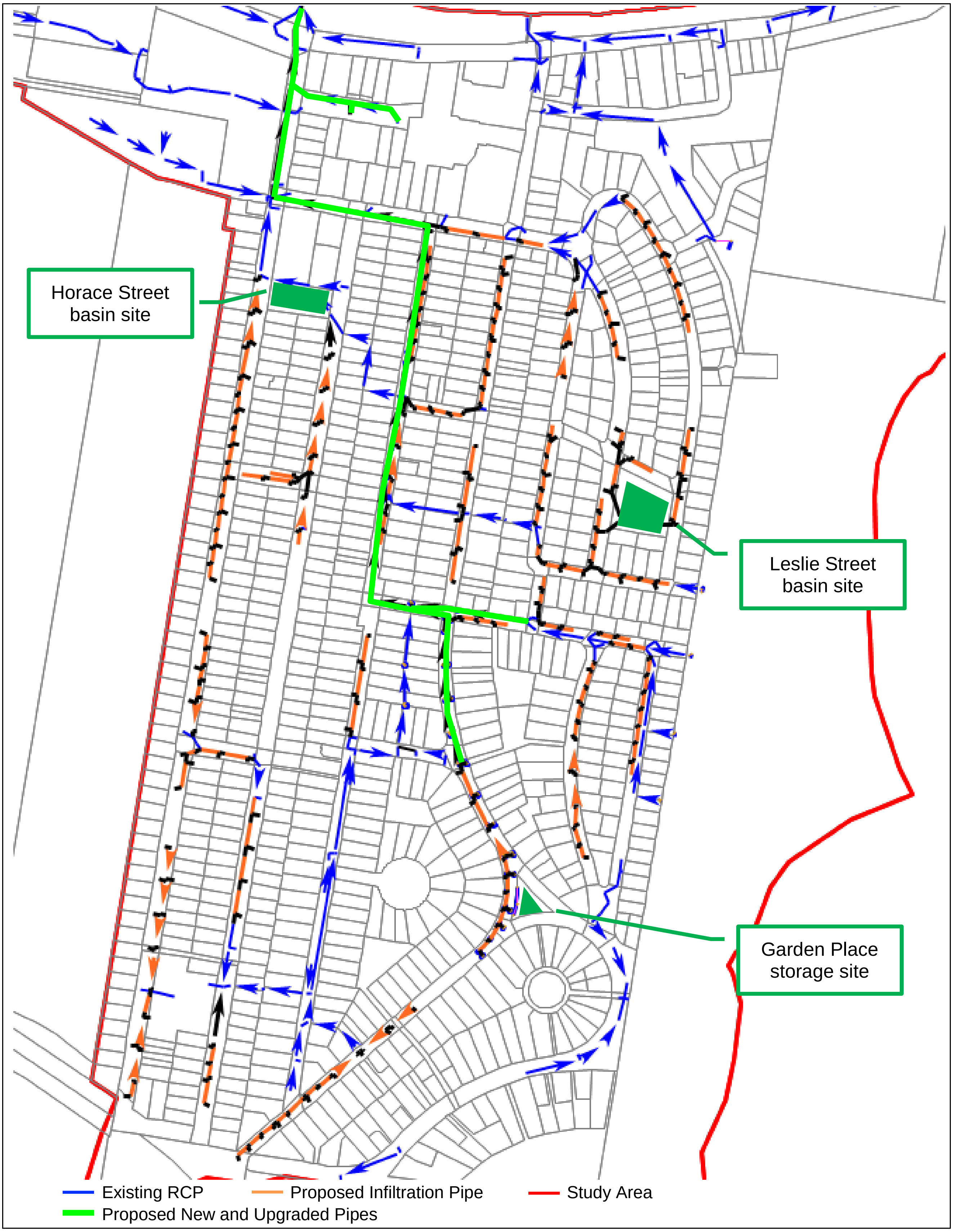


Figure 6-10 Proposed Option 4 Infiltration Devices and Stormwater Upgrades

Major details of the proposed stormwater pipe upgrades include:

> Double 1.2 m high x 1.5 m wide RCBC from Shoal Bay beach to just south of Edward Street, then decreasing to a double 1.35 m diameter RCP at Verona Road to convey 1% AEP flow from the upstream catchment;

> New outlet structure at Shoal Bay beach and energy dissipator, similar to an RMS ED7 energy dissipator (refer **Appendix E**);

> New kerb and gutter and minor system stormwater drainage along Tomaree Road from Government Road (south) to Verona Road to convey 20% AEP flow;

> Introduce larger inlet pits in rear of lots between Fingal Street and Verona Road to reduce ponding, and;

> Increase the number of inlet pits and stormwater pipes sizes at Bullecourt Street.

This option includes the same proposed works from Options 1 and 2:

> A basin at Leslie Street open space (refer **Figure 6-3**);

> A basin at the northern end of Horace Street (refer **Figure 6-6**);

> New kerb and gutter along roadways with new stormwater pipes (refer **Figure 6-4**), and;

> Western verge raising along Rigney Street (near Edward Street, similar to Option 2) by 100 mm to increase gutter flow capacity and avoid overland flows passing into properties on the lower side of the road.

6.2.5.2 Advantages and Disadvantages

Option 4 has the following advantages:

> Combines new kerb and gutter with new permeable pipes and stormwater pipes to increase the total kerb and gutter in Shoal Bay;

> Significantly reduces the volume of flooding in lower elevations of the study area for all storm events up to the 1% AEP;

> System is similar to a major/minor drainage network (as described in AR&R) along the upgraded stormwater lines, and;

> Infiltration pipes can be implemented over time when budget becomes available.

The following disadvantages are associated with Option 4:

> Will take significant time for complete implementation;

> High capital cost;

> Most effective for short duration and/or intensity rainfall events where the total rainfall volume is relatively small;

> Infiltration potential of permeable pipes and basins will decrease over time as the system suffers from siltation;

> Requires significant maintenance regime for the system to be effective in the long term;

> Will not be able to reduce runoff enough to achieve the objective of having a major/minor drainage system, as described in AR&R, for much of the existing network;

> Significant disruption from construction along roadways for the major stormwater line upgrade;

> High risk of significant cost increases to realign services affected by stormwater upgrades;

> Major works to Shoal Bay beach outlets and increased environmental impact on beach and receiving waters;

> Reconstruction or replacement of major GPT at the north end of Government Road, and;

> Minor works are required in private property.

6.2.5.3 Costs

The overall cost of this option takes into account both the capital costs and the maintenance cost associated with the proposed infrastructure over a period of 50 years.

A summary of assumptions and costs for Option 4 are as follows:

> Capital cost = $8.69 M
> Maintenance cost for first year = $30,000
> Assumed increased cost of maintenance per year = 3%
> Assumed interest rate for Net Present Value = 5%
> Life cycle of RCP = 100 years
> Life cycle of permeable pipes = 50 years
> Period of analysis = 50 years

The net present value for Option 4 is $9.67 M. This cost assumes that all capital cost works are completed in the first year.

Costing details of this option can be found in **Appendix D**.

6.2.5.4 Flood Results

Refer to **Appendix B**, Figure B52 to Figure B60, for maps showing the impact Option 4 has on flooding in Shoal Bay for the 1%, 5% and 10% AEP.

The following are notes on the results for areas currently experiencing significant flooding:

> The new and upgraded stormwater lines from Shoal Bay beach to Verona Road conveys the 1% AEP flow from their respective upstream subcatchments. Rigney Street effectively becomes a cut-off for flows from the residential areas of Shoal Bay to be transferred via the underground stormwater conduits to Shoal Bay beach.
> At the northern end of Horace Street where major flooding occurs in the existing conditions scenario, flood depths are reduced significantly by flooding being diverted to the new Horace Street basin. Some flooding remains on the roadway and this has a depth of 200 mm in all AEP events. Some flooding does remain in private properties but these properties are at elevations lower than the roadway and flooding cannot be reduced any further from works on public land.
> Flooding was effectively eliminated along Bullecourt Street for the 10% and 20% AEP events – 200 mm deep flooding along the south eastern kerb. Only in the 1% AEP event does flooding occur along the southeast section of the road up to a depth of 300 mm. The private properties to the south of Bullecourt Street are still prone to flooding in all AEP events because their ground elevation is lower than the roadway.
> Flooding was significantly reduced in the private properties between Fingal Street and Verona Road down to a depth of 300 mm in all AEP events. This flooding occurred at a localised low point in back yards of private properties which cannot be drained by the stormwater pipe aligned along the rear boundary of the properties.
> Flood waters in and around Passchendaele Park, up to Rigney Street, were reduced by 300 mm to 400 mm for all AEP events. The extent of flooding was reduced to eliminate significant on road flooding of Horace Street in the 10% and 20% AEP events. Minor flooding (less than 300 mm) was shown on Horace Street near Passchendaele Park in the 1% AEP event.

6.3 Multi-Criteria Assessment

To compare each option, and aid in the selection of a preferred option, a multi-criteria assessment has been undertaken.

Categories assessed include:

> Reduction of flood levels. This includes both the extent and magnitude of the reduction of flood levels within the study area. Qualitative assessment of this criteria are as follows:
  - Insignificant: flood levels are not impacted and the community is unlikely to notice any effect.
  - Minor: flood level reduction is less than desirable and the community may not notice the effects of proposed works.
  - Significant: flood level reduction is considerable with the stormwater network approaching the objectives of a major/minor system; the community notices a major positive impact on flooding experienced.

> Net Present Value. This is expressed as the cost in year one of the assessment and takes into account future maintenance and replacement costs over 50 years.

> Construction impact. This is expressed as the estimated timeframe for construction of the full option and the expected impact on traffic disruption and town aesthetics. It is assumed that each option will be fully funded at its outset. Under guidance from Council, it is assumed that completion time for each option is equal and approximately 5 years.

> Environmental impact. This includes the assumed effect of each option on the condition of the receiving waters' quality, air quality, groundwater regime, aquatic and terrestrial ecosystems, and aesthetics. A qualitative assessment of each option with the following criteria:
  - Negative: The option leaves the environment in a worse state than the existing condition.
  - Neutral: The option leaves the environment in a similar state to the existing condition.
  - Positive: The option leaves the environment in a better state than the existing condition.

Table 6-1 displays the results of the assessment and **Table 6-2** shows the rankings of the assessment.

Table 6-1 Multi Criteria Assessment Results

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact
1	Minor. Resulting flooding in study area is more desirable for smaller events (i.e. 10% and 20% AEP) but option has less of an impact on the 1% AEP event.	$6.10 M	~ 5 Years Less impact from staged infiltration devices and no large construction works to roads.	Receiving water quality – positive Air quality – neutral Groundwater– neutral Ecosystems – neutral Aesthetics – neutral Overall - positive
1A	Minor/Significant. Resulting flooding in study area is more desirable for smaller events (i.e. 10% and 20% AEP) but option has less of an impact on the 1% AEP event. Flooding along Bullecourt Street significantly reduced.	$6.47	~ 5 Years Less impact from staged infiltration devices and no large construction works to roads. More than Option 1 with other stormwater upgrades and earthworks.	Receiving water quality – positive Air quality – neutral Groundwater– neutral Ecosystems – neutral Aesthetics – neutral Overall - positive

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact
2	Significant. Areas with existing flooding issues experience a large reduction in flood depths. Not all areas of town are affected. Reduces flooding in area of Public School better than other options.	$6.02 M	~ 5 Years Significant and relatively widespread large roadworks resulting in road closures.	Receiving water quality – negative Air quality – neutral Groundwater– neutral Ecosystems – neutral Aesthetics – slightly negative Overall - negative
3	Significant. Areas with existing flooding issues experience a large reduction in flood depths. Not all areas of town are affected, but more than Option 2.	$6.73 M	~ 5 Years Significant and relatively widespread large roadworks resulting in road closures. More than all options.	Receiving water quality – negative Air quality – neutral Groundwater– neutral Ecosystems – neutral Aesthetics – negative Overall - negative
4	Significant. Areas with existing flooding issues experience a large reduction in flood depths. Greatest positive effect extent on flooding, apart from Public School.	$9.67 M	~ 5 Years Significant and relatively widespread large roadworks resulting in road closures, plus infiltration devices.	Receiving water quality – positive Air quality – neutral Groundwater– neutral Ecosystems – neutral Aesthetics – slightly negative Overall - negative

Table 6-2 Multi Criteria Assessment Ranking

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact	Sum of Rankings
1	5	2	1	1	11
1A	4	3	2	2	12
2	3	1	3	4	11
3	2	4	5	5	14
4	1	5	4	3	12

Assuming an equal weighting to each assessment criteria, results indicate that Option 2 or Option 1 would be the preferred option for implementation. However, this assumption is not always accurate and an assessment of different weighting for each of the criteria assists in determining a recommended option.

Cardno has identified alternative, value-based weighting for the criteria, as follows:

> Community desire outcome.
  Based on the two community workshops, this puts the most significant weighting on flood level reduction and less weighting on the overall cost of the option.

> Least cost outcome.
  This emphasises the option cost above other criteria while still achieving a good level of flood reduction.

> Best environmental outcome.
  This puts significant weighting on the environmental impacts of the option while still achieving a good level of flood reduction.

> Highest safety outcome.
  This emphasises overall level of safety to the community during and following completion of any proposed works.

Table 6-3 identifies initial criteria weighting scenarios.

Table 6-3 Multi Criteria Assessment Weighting Scenario

Scenario	Reduction of Flood Levels Multiplier	Net Present Value Multiplier	Construction Impact Multiplier	Environmental Impact Multiplier
Community Desire	1.8	0.5	0.8	0.8
Least Cost	1	1.5	0.5	0.5
Best Environmental	1	0.7	0.7	1.7
Highest safety	2	0.5	1	0.8

These weightings are multiplied by the rankings from **Table 6-2** and then summed to determine the recommended options for each scenario.

6.3.2 Community Desire Scenario

Using the weighting emphasising the community's desires, as interpreted during the two community workshops, the weighted rankings are shown below in **Table 6-4**.

Table 6-4 Multi Criteria Assessment Weighted Ranking – Community Desire

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact	Sum of Rankings
1	9	1	0.8	0.8	11.6
1A	7.2	1.5	1.6	1.6	11.9
2	5.4	0.5	2.4	3.2	11.5
3	3.6	2	4	4	13.6
4	1.8	2.5	3.2	2.4	9.9

Using this weighting, Option 4 would be the recommended option, followed by Options 2, 1 then 1A.

6.3.3 Least Cost Scenario

Using the weighting emphasising the community's desires, as interpreted during the two community workshops, the weighted rankings are shown below in **Table 6-5**.

Table 6-5 Multi Criteria Assessment Weighted Ranking – Least Cost

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact	Sum of Rankings
1	5	3	0.5	0.5	9
1A	4	4.5	1	1	10.5
2	3	1.5	1.5	2	8
3	2	6	2.5	2.5	13
4	1	7.5	2	1.5	12

Using this weighting, Option 2 would be the recommended option, followed closely by Option 1 then 1A.

6.3.4 Best Environmental Scenario

Using the weighting emphasising the community's desires, as interpreted during the two community workshops, the weighted rankings are shown below in **Table 6-6**.

Table 6-6 Multi Criteria Assessment Weighted Ranking – Best Environmental

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact	Sum of Rankings
1	5	1.4	0.7	1.7	8.8
1A	4	2.1	1.4	3.4	10.9
2	3	0.7	2.1	6.8	12.6
3	2	2.8	3.5	8.5	16.8
4	1	3.5	2.8	5.1	12.4

Using this weighting, Option 1 would be the recommended option followed by Option 1A.

6.3.5 Highest Safety Scenario

Using the weighting emphasising the community's desires, as interpreted during the two community workshops, the weighted rankings are shown below in **Table 6-7**.

Table 6-7 Multi Criteria Assessment Weighted Ranking – Highest Safety

Option	Reduction of Flood Levels	Net Present Value	Construction Impact	Environmental Impact	Sum of Rankings
1	10	1	1	0.8	12.8
1A	8	1.5	2	1.6	13.1
2	6	0.5	3	3.2	12.7
3	4	2	5	4	15
4	2	2.5	4	2.4	10.9

Using this weighting, Option 4 would be the recommended option, followed by Option 2 then Option 1.

7 Recommendation

Through hydrological and hydraulic analysis of the existing drainage network in Shoal Bay, five (5) separate options for mitigation of flooding issues were developed. These options were further analysed and assessed to quantify their impacts on flooding.

Option 2 represents a good level of flood depth reduction in most of the major problem flooding areas. It also provides this flood reduction at the least cost over a 50 year period. Other options do provide a further reduction of flood levels over a lager extent of the study area but do so at a significantly increased cost.

The environmental impacts of the recommended option are only slightly negative as it could potentially transport a greater pollutant load to the receiving waters in Port Stephens compared to the existing conditions. This will need to be further explored in future studies. Additionally, it may be less desirable and less aesthetically pleasing to the community to have a larger, more intrusive headwall and energy dissipation structure at Shoal Bay beach, than could be achieved through selection of another option.

Using four analysis criteria and four separate weighting scenarios, the multi-criteria assessment ranked Option 2 first in the least cost scenario, second in the community desire and highest safety scenarios, and fourth in the best environmental outcome scenario.

It is recommended that Option 2 be selected as the preferred option and proceed to concept and detailed design and eventually construction.

If Option 2 is progressed, it is further recommended that the following tasks and/or studies be carried out:

> Geotechnical investigation in key areas of the proposed works;

> Detailed survey in the area of proposed works, including underground utility survey to determine services which may be impacted by construction;

> Further study and hydrogeological assessment to quantify the effect of this option on groundwater levels and quality and potentially on the Hunter Water Corporation bores in the nearby Tomaree Sandbeds;

> Identification of any other Council works in the vicinity which may be incorporated into the recommended option to provide a greater overall cost savings;

> Further study to quantify the impacts of the recommended option on the receiving waters of Port Stephens, and;

> Further concept design to provide greater certainty on the expected cost of implementing the recommended option.