Appendix B

Water assessments

Appendix B1

Benedict Industries, Georges Cove Marina, Moorebank, October 2010

Appendix 1.1

Georges Cove Marina, Moorebank, Preliminary Marina Concept Design and Environmental Assessment – Worley Parsons October 2010

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BENEDICT INDUSTRIES PTY LTD

Georges Cove Marina, Moorebank

Preliminary Marina Concept Design and Environmental Assessment

301015-01167

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PRELIMINARY MARINA CONCEPT DESIGN AND ENVIRONMENTAL ASSESSMENT

SYNOPSIS

This report has been prepared to support the development application for the proposed Georges Cove Marina. It addresses the engineering, river and water related aspects of the proposed development.

The proposed marina, with appropriate design and mitigation measures as recommended in this report, would not have a significant adverse impact on the river processes, flooding or water quality and would provide a facility which is in short supply along the Georges River.

Disclaimer

PROJECT 301015-01167 - GEORGES COVE MARINA, MOOREBANK

REV	DESCRIPTION	ORIG	REVIEW	WORLEY-PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
B	per client comments	FC	MT	N/A	11 Mar 2009	N/A	
C	Revised for new marina concept	FC	MT	N/A	11 June 2010		N/A
D	Added copper discussion	FC	MT	MT	30 July 2010		N/A
E	Minor Edits	FC	MT	MT	7 Oct 2010		

CONTENTS

1. INTRODUCTION

Benedict Sand & Gravel Pty Ltd proposes to redevelop the Moorebank quarry site to incorporate a marina. This report has been prepared to support the development application for the proposed Georges Cove Marina. It addresses the engineering, river and water related aspects of the proposed development.

The site is located south of Newbridge Road at Moorebank and adjacent to the Georges River.

The marina basin would be formed in the location of the current quarry excavation.

The proposed marina site is adjacent to an area where a combination of residential and commercial development is proposed.

The proposed development site location is shown on **Diagram 1–1**.

Diagram 1–1– Site Location Plan

2. DESCRIPTION OF THE EXISTING ENVIRONMENT

2.1 River Processes

River processes relate to sediment transport, tidal flows, flood flows and water quality. Detailed analysis of tidal flows and water quality can be found in **Sections 5 and 6**.

2.1.1 Sediment Transport

The tidal velocities in the river are sufficient to transport sand as bed load along the river. This has been established in the Georges River Database Compilation and Assessment report prepared by PWD in 1991. It also identified that the Chipping Norton Lakes System formed in the early 1980s acts as a large trap for sand transported along the river.

2.1.2 Tidal Flows

Tidal flows in the river represent a significant volume of flow. The basin would create a small additional waterway area with a tidal prism forming a negligible fraction of the tidal volume.

2.1.3 Flooding

The site is located on a floodplain. Typically, a 2-year ARI event would overtop a river bank, in this case the bank has been artificially modified and it is uncertain if this would occur. The basin would provide a small increase in flood conveyance and storage area and would have a beneficial impact by marginally lowering the flood level in the local area.

2.1.4 Water Levels

The Georges River Database Compilation and Assessment prepared in February 1991 by the then Public Works Department provides a review of tidal measurements in the Georges River. The tidal planes determined at Milperra Bridge (closest location to the subject site) are presented in **Table 2–1**.

The predicted Georges River flood levels are presented in the Georges River Flood Study undertaken by Bewsher Consulting in 2000 which established a MIKE 11 model to reproduce the original levels predicted in the PWD physical flood model for Georges River. The predicted flood levels for the site are:

- 20 yr ARI RL 4.6 m AHD
- 100 yr ARI RL 5.6 m AHD
- PMF RL 10.2 m AHD

Table 2–1 – Tidal Water Levels

Plane	Datum (m)	
	ISLW	AHD
Higher High Water Springs	1.913	0.988
Mean High Water	1.466	0.541
Mean Water Level	1.002	0.077
Mean Low Water	0.538	-0.387
ISLW	0.193	-0.732

2.1.5 Water Quality

The major influences on water quality in the river are stormwater discharges, treated and untreated sewage discharges, tidal flushing and flood flows. The proposed development would not influence these major processes in the river.

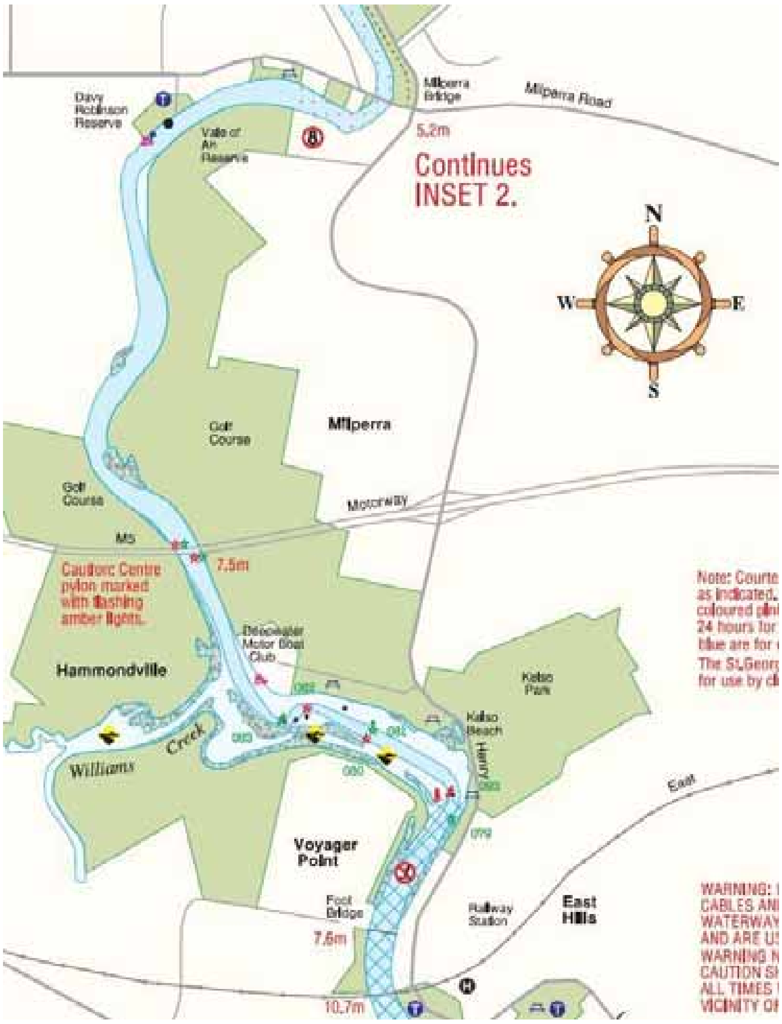
The water quality in the basin would therefore be primarily influenced by the tidal flushing from the river, i.e. the ability to maintain the same water quality in the basin as in the river. The basin has been designed to maximise circulation by maintaining an open connection to the river. The depth of the basin would be above the adjacent river bed level, avoiding poor circulation of water at the bed of the basin.

2.2 Sediment

2.2.1 Recreational Use of the River

There are numerous existing boat ramps along the Georges River supporting use of trailered recreational craft. **Diagram 2–1** shows the NSW Maritime Boating map for the area which indicates the location of existing facilities and any boating restrictions. The proposed marina is located within a length of river that is heavily used for speed boat racing and water skiing (between Davy Robinson boat ramp and the Deep Water Motor Club). This area has no speed restrictions (speed restrictions exist along most of the Georges River) and it is therefore an important amenity area for speed boat activities.

Diagram 2–1 – Local Boating Map (NSW Maritime)



Discussions with NSW Maritime indicate that there are no proposals to limit the speed along this reach of the river, so as to continue its availability for water skiing and other high speed uses.

The only aquatic licences in the area would be for events held by the Deepwater Motor Boat Club, which runs racing events.

3. DESCRIPTION OF THE PROJECT

3.1 Marina Basin

The proposal includes a marina basin of approximate dimensions of 150 m by 350 m. The marina would open to the Georges River with a short entrance channel 40-50 m wide. The layout of the proposed marina development and basin is shown in **Figure 1**.

The proposed approximately rectangular plan shape of the marina basin with dimensions of approximately 150 m by 380 m has been designed to alleviate the potential for the formation of poorly flushed corners and will assist to maintain good water quality and visual appearance.

The marina basin would be located on a relatively straight section of the river. These sections are generally more stable and less prone to the significant variations in flow velocity at bends in the river. These velocity variations typically result in sediment deposition on the inside of the bend and significant erosion on the outside of the bend. The location of the marina on the relatively straight river section will provide more stable bank and bed conditions and less potential for sedimentation in the marina basin or entrance.

The basin would be formed by filling the existing quarry to shape it into the final landform using a dredge and land based earth moving machinery. The dredge would operate as at present in a water filled basin. The excavated sand would be used for forming the Vessel Access Channel and associated land areas required for the marina. This work would commence at the landward end forming the basin and land base prior to breakthrough of the banks to the river. In this way, these works would not impact on the river water quality. The breakthrough to the river would be undertaken as the last activity after the water quality in the basin had stabilised and was suitable to discharge to the river once the banks were excavated. Water quality testing through the quarry pond and the river area adjacent to the site demonstrates that the water in the pond is of a similar quality to that of the Georges River (refer to **Table 6–2** for results of water quality testing).

3.2 Marina Basin Depth

The basin water depth is a function of the craft draught and the water depths available in the adjacent river.

The craft to be catered for in the marina would be power craft up to around 20 m in length. The Australian Standard AS 3962 – 2001, Guidelines for Design of Marinas recommends the draught of these craft of 1.5 m. The standard recommends that the basin water depth be determined by adding a freeboard of 0.3m to the largest vessel draught and half the design wave height as well as an allowance for sedimentation. The recommended basin bed level would be RL 2.2 m below the local Indian Spring Low Water (ISLW) which includes the 1.5 m draught, 0.3 m freeboard, 0.1 m half wave height and 0.3 m for sedimentation. ISLW is the chart datum or lowest expected tidal water level in the river. The proposed basin bed level would therefore be RL-2.932 m AHD.

The Georges River Flood Study employed a model from which the cross-sectional elevations were used to generate a terrain model of the river.

Survey of the depth of the river was undertaken adjacent to the proposed marina.

3.3 Berths Details

From consultation with NSW Maritime and boat user groups, the proposed marina is within a highly utilised stretch of the river. The existing layout would accommodate a total of 186 wet berths and 250 dry berths as indicated in the following tables. The number of berths is restricted by the site and space available. The marina layout is shown in **Figure 1**.

Table 3–1 – Marina

Length (m)	Beam (m)	Percentage	Draft (m)	Berth Widths (m)	Number Of Berths
Small Craft (<15m) ≥8m <10m	4	28%	1	9	52
Small Craft (<15m) ≥10m <12m	4.4	59%	1	9.8	110
Small Craft (<15m) ≥12m <15m	5	11%	1.2	11	20
Vessels ≥15m <18m	5.4	1%		11.8	2
Vessels <20	5.7	1%	1.5	12.4	2
Vessels >20m					0
Total					186

Table 3–2 – Dry Store

Length (m)	Beam (m)	Percentage	Draft (m)	Berth Widths (m)	Number Of Berths
Small Craft (<15m) ≥4m <6m	2.8	50%		4	125
Small Craft (<15m) ≥6m <8m	3.4	20%	0.9	4.6	125
Vessels ≥15m					0
Total					**250**

3.4 Marina Building

The proposed marina building would include parking on the ground floor, a conference facility, kiosks, function centre/ casual dining, dry storage for boats, marina office, amenities. The details of the dry berths are presented in **Table 3–2** and a cross section through the marina building is shown in **Figure 2**.

3.5 Marina Entrance

The proposed marina is recessed into the riverbank and entrance to the river would be via a short entrance channel (approximately 50 m long). The entrance would be 40 to 50 m wide.

A pedestrian bridge would be provided across the river entrance to the marina to allow pedestrian connectivity along the Georges River foreshore. This bridge would also act to limit the size of vessels entering the marina by precluding vessels above a set height.

3.6 Finished Levels

The foreshore levels for the marina area are influenced by the need for convenient access to craft, adjacent land and proposed development levels and flood levels.

The marina building would consist of parking, a conference facility, function centre, kiosks, waterside casual dining and dry boat sales, repair and storage. The minimum habitable floor level for the conference as designated by Liverpool City Council policy would be the flood planning level (0.5m above the 100yr ARI flood level) which would be RL 6.1 m AHD. The mean tide level in the marina basin would be around RL 0 m AHD and as such, provision of convenient access to craft is an issue which requires careful design consideration.

The northern carpark would be formed at the 20 year ARI flood level of RL 4.6 m AHD. Otherwise, the ground surrounding the marina building would generally be at a level of RL 2.5 m AHD. The entrance roadway, connecting to the residential area to the north, would need to rise to approximately RL 5.7 m AHD, at a relatively low grade (say 8%) to allow ease of transport of boats on trailers.

It will be necessary to incorporate a number of benches into the foreshore fronting the marina berths. The uses at the foreshore level have been selected to minimise flood damages. Given the flood warning time, which would be greater that 12 hours, important equipment or records could be relocated to higher ground to minimise flood damages even further. It would be necessary to work closely with Council to devise controls which were sufficiently practical to warrant a special “merits based” dispensation through Council’s flood policy for development at a level of RL 2.5 m AHD.

A timber boardwalk would be located at a level of RL 1.5 m AHD to provide a better amenity for access to craft and further opportunity for passive recreation close to the river/basin water level. A hinged ramp would provide access from the promenade level (RL 2.5 m AHD) to the floating pontoon berths. Access would be provided from the promenade level in order that the access ramp would not be damaged in a 100 yr ARI flood as it would be able to rotate up to the flood level of RL 5.6 m AHD.

The sections in **Figure 2** illustrate the proposed levels and their relationship with the surrounding area.

3.7 Bank Edge Treatments

The main function of the basin edge treatments is bank stability with a strong focus on creating environmental enhancement. Two potential edge treatments have been identified for use within the marina. The marina edge treatment locations have been suggested based on an appreciation of their integration with the proposed river bank protection, proposed development layout and the existing ecology.

Two edge treatments have been considered within the basin; **Type 1** is a combination of rock revetments and integrated vegetation zone; and **Type 2** is a concrete wall. Type 1 would be similar to those proposed to stabilise the river banks along the site.

A freshwater wetland is proposed along the inside of the river bank to treat runoff from the adjacent proposed residential area. The wetland would be vegetated with suitable macrophytes and would enhance the riparian zone with more diverse aquatic habitats and diverse vegetation.

Type 1 - Rock Revetment with Integrated Vegetation Zone

For this edge treatment the rock revetment would incorporate a berm of saltmarsh (or other suitable habitat) between the rock revetment and the edge treatment to improve the ecological and aesthetic value of the foreshore. The foreshore would then be backed by littoral vegetation and possibly an access path.

This option stabilises the slope through a rock revetment with the benefit of creating specific habitat environments within it. The revetment would consist of layers of rock armour to comply with the wave climate, over a geotextile fabric.

Further design of this option would be required in consultation with a specialist ecologist to establish the required height for the mid level vegetation and suitable native species.

Type 2 – Rock Revetment with Boardwalk

To enable access to berths, the use of a rock revetment with a timber boardwalk is suggested.

Where the boat moorings are proposed a timber boardwalk and a walkway down to floating pontoons would be constructed over the rock revetment. The boardwalk and pontoon would extend sufficiently over the revetment so that the required depth would be achieved below the moored boats. Alternatively, the walkway could extend directly from the shore out to floating pontoons.

3.8 Carparking

Carparking has been provided in accordance with the numbers agreed with Council. The carpark layout is shown on **Figure 2**.

3.9 River Bank and Riparian Zone

The existing riverbank over the 500 m of the site foreshore is generally devoid of any vegetation. The river banks are eroding thereby undermining what foreshore vegetation exists at present. Without this development, these processes will continue to remove the riparian vegetation and there will continue to be no public access to the foreshore.

This project proposes to rehabilitate the foreshore and re-establish the riparian corridor vegetation representing a significant improvement to the aquatic and riparian environment. The NSW government does not have the resources to undertake these regionally significant river works which is their responsibility. The residents of the region will benefit greatly from the improved river environment and foreshore access. The loss of a small section (40 to 50 m wide) of the river foreshore for the basin entrance is minor and readily compensated for with the proposed foreshore stabilisation and riparian rehabilitation over the 500m of the remainder of the site foreshore. The marina basin would provide additional river foreshore and riparian vegetation and habitats to compensate for the foreshore loss at the basin entrance.

It should be noted that the existing length of river bank, at the proposed basin entrance, is degraded. The bank is also continuing to erode. The proposed works would improve the river bank in a number of ways:

- reduce ongoing scour/erosion of the river bank;
- provide additional habitats (including wetlands) and improve local biodiversity;
- improve visual quality; and
- improve foreshore amenity.

The basin entrance has been located to avoid any significant existing vegetation. The impact of this entrance on the riparian zone is considered minimal given the overall positive benefits to be derived from the bank stabilisation and re-establishment of the riparian zone vegetation over the entire site.

3.10 Sedimentation of the Basin

Sedimentation of the basin could occur due to inflow of sand size sediments being transported along the bed of the river by tidal and flood processes and with the deposition of fine sediments from floodwaters.

The deposition of fine sediments in the basin during a large flood would occur but at a very slow rate. The velocities in the over bank flood area in which the basin would be located are sufficient to transport fine sediment and to prevent its deposition. The basin is not deep enough to provide sufficiently low velocities during a flood to cause significant deposition of fine sediments. At the conclusion of a flood, there will be the gradual deposition of an insignificant amount of fine sediments remaining in the water column.

The tidal velocities in the river are sufficient to transport sand as bed load along the river. This has been established in the Georges River Database Compilation and Assessment report prepared by PWD in 1991. It also identified that the Chipping Norton Lakes System formed in the early 1980s acts as a large trap for sand transported along the river. As such, the sand transport rate past the proposed marina basin will be low for many decades, until such time as the lakes are full of sediments or the landform is otherwise changed. Hence, the source of sediment from the Georges River would be low.

The reasons the sedimentation rate in the basin of sand size sediments will be low include:

- location of the basin entrance on a straight section of the river channel not exposed to the large depositional episodes on river bends;
- the bed of the basin has been located above the general bed level of the river and hence bed load sand transport will not enter the basin; and
- the bed load sand transport rate past the basin entrance will be low.

The depth of sediment collected in 100 years was estimated using the following parameters:

- an average suspended solids concentration of 30 mg/L,
- an average particle size of 2 micron (clay),
- a fluid density of 1000 kg/m^3,
- a sediment density of 2500 kg/m^3, and
- a fluid viscosity of 0.01 Pa s.

The depth of sediment was thus estimated to be approximately 120 mm.

A 300 mm allowance has been adopted to accommodate minor sedimentation in the basin.

3.11 Construction

It is proposed to construct the marina using conventional land based plant and equipment *(such as excavators and all terrain trucks*), as is currently used to quarry material on the site. This marina basin is substantially already formed by the quarrying activities.

Connection to the river would be delayed for as long as practicable and would be completed during favourable water level conditions (i.e. at or around slack water) to enable management of the breakthrough with the deployment of an appropriate turbidity curtain.

The excavation works would be managed to minimise the total area to be disturbed and the exposure time of any disturbed area.

4. FLOODING

The Georges River Flood Study was completed in 2000 by Bewsher Consulting. The flood study was based on a MIKE11 hydraulic model and has been adopted by the Bankstown, Fairfield and Liverpool Councils. Results from the study have been incorporated into the Georges River Floodplain Risk Management Study and Plan, (Bewsher Consulting, 2004).

The model extends over a distance of some 46km, from above Cambridge Avenue to Botany Bay. There are over 278 cross sections and a number of separate overland flow paths.

The mid section of the model used for the Georges River Flood Study is the section relevant to this project, as it extends from Liverpool to Picnic Point. Prior to the MIKE11 model, this reach was modelled by the Georges River physical model, which had been extensively calibrated to floods that occurred in 1956, 1978, 1986 and 1988. Consequently, there is a high degree of confidence in the results from the physical model. The MIKE-11 model was calibrated to the physical model in the reach between Liverpool and Picnic Point.

4.1 Flood Impact Assessment

As the Georges River Flood Study (2000) is considered to be representative of the state of the current catchment, the flood impact assessment was based on the results of the existing study. The available flood storage is presented in **Table 4–1**. The calculations account for a loss of flood storage caused by the residential development, there is an overall gain in flood storage, which is largely due to the marina basin.

Table 4–1 – Flood Storage

Location	**Plan Area (m^2)**	**Existing Level (m AHD)**	**Proposed Level (m AHD)**	**Change in Elevation (m)**	**Change in Flood Storage (m^3)**
Northern Carpark (raised above 20 year ARI level)	7,801	2.7	4.6	-1.9	-14,822
Dry Storage Building	21,807	2.5	4.6	-2.1	-45,795
Southern Carpark	15,120	2.5	2.8	-0.3	-4,536
Riparian Zone	18,973	2	1.8	0.2	3,795
Wetlands	4,648	2	0.541	1.459	6,781
Marina Basin	48,727	2.7	0.541	2.159	105,202
Entrance to Basin	3,656	2	0.541	1.459	5,334
Net change					**55,959**

4.2 Marina Flood Management

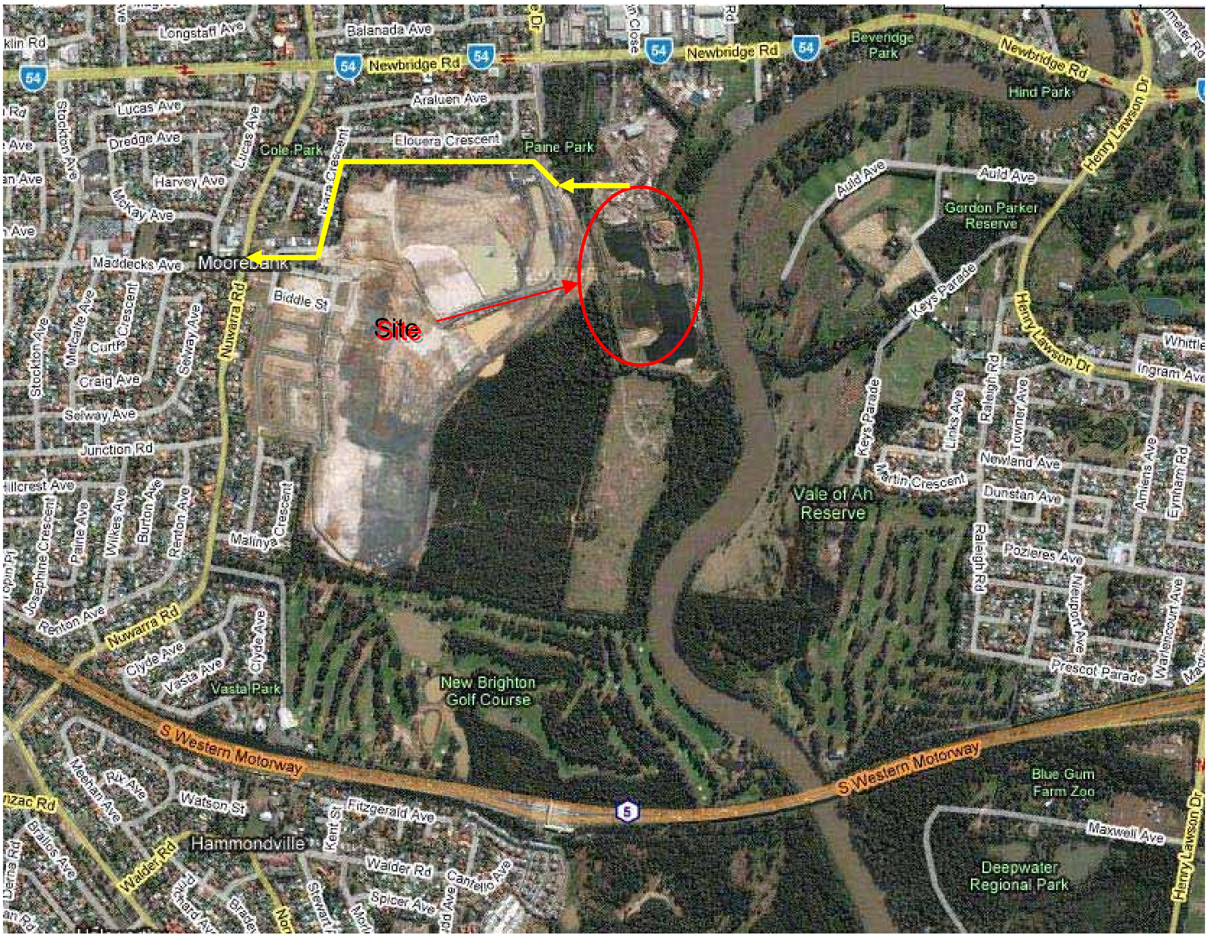
The floating marina pontoons would be designed to cater for the flood levels and flow and debris forces imposed by the 100yr ARI flood flows. This could be achieved with appropriate pile design in combination with consideration of the access connection between the floating pontoons and the foreshore areas. It is typical that the foreshore be gradually stepped down in a series of levels and boardwalks to provide a convenient amenity, access and visual aesthetics to this area.

4.3 Preliminary Flood Emergency Response Plan

The flood emergency response plan (FERP) is required for the marina and building to provide a strategy for dealing with floods more severe than the 100yr ARI event. There would be many hours warning for a severe flood. The key principle in the FERP is to provide an evacuation strategy from the site to a location above the water level in the Probable Maximum Flood (PMF).

The egress route for the marina is initially to the north, through the residential and commercial areas, to Newbridge Road, and then turning left to travel west along Newbridge Road. The route is indicated on **Diagram 4–1**.

Diagram 4–1 – Flood Evacuation Route



A full FERP would be prepared at a later date once the full details of the occupation of the building are known, including nomination of individual staff for emergency roles.

Flood warnings for the Georges River are available from the Bureau of Meteorology. For a fee, these can be sent directly to a site, to the Flood Warden.

Given the nature of the conference facility, the space would not be permanently occupied, reducing the risk of a flood occurring while people are on site. It would also be possible to prevent access to the site when a flood warning had been received.

A backup anchor pile and chain system would hold in place the marina pontoons should the piles fail or the pontoons float free of them.

All craft would also be readily tied to the chain system with quick lock fixtures when a severe flood warning was received.

5. TIDAL HYDRODYNAMICS

A hydraulic model was created using the RMA 2 software which provides results detailing the flow behaviour.

RMA-2 is a two dimensional fluid flow program which allows complicated and detailed flow systems to be designed and modelled. RMA-2 effectively simulates water flow through different networks; however it is essential that the network has been created carefully in order to efficiently model the land topography.

The model network consists of a mesh of variable sized quadrilaterals and triangles, similar in nature to a triangulated irregular network (*TIN*). Each quadrilateral or triangular element represents a portion of the ground surface defined by elevations at the corner and midside nodes. The elements also represent roughness of the surface defined by Manning's n or Chezy C values. The flexible nature of the network permits complex changes in topography, built environment and hydraulic conditions to be modelled with appropriate degrees of accuracy or representation. Elements represent areas of land, therefore the smaller the elements the more effectively the area is modelled, however the associated processing time increases. Due to these factors a compromise is sought to effectively model the network with sufficient detail, yet while not complicating the model unnecessarily.

5.1 Model Extents

In order to capture the full tidal prism, it was necessary to create a model of the Georges River from downstream of the subject site extending upstream to the Chipping Norton lakes system.

The cross-sectional model extents were limited to the area wet under average conditions, i.e. overbank areas were not included. This created a more stable model without losing any necessary information for the water quality modelling.

5.2 Boundary Conditions

The location and model boundary conditions were selected based on a range of criteria, so that the model results were stable and produced an accurate replication at the subject site.

The upstream and downstream boundary condition needed to be located a sufficient distance away from the subject site to ensure that the model stabilises and produces accurate results. The upstream and downstream boundary conditions were adopted from historical data supplied by the Manly Hydraulics Laboratory. The upstream site of Lansvale was adopted as it lies at the upstream boundary of the Chipping Norton lakes system. For the downstream boundary, the next suitable location was at Picnic Point. Despite the closer Kelso Creek station being located closer to the subject site, yet sufficiently downstream, it was not suitable as it did not capture the full tidal range thereby producing inaccurate results.

5.3 Model Terrain

The model terrain used to construct the RMA model, was adopted using a combination of data. Specifically, the information used included:

- Aerial photography supplied by the NSW Department of Lands;
- A digital terrain model generated from cross-sectional survey data from the Georges River Flood Study; and
- Visual inspections and measurements (which were used to accurately define the Manning's "n" values along the channel and the dimensions of hydraulic features).

5.4 Marina Basin

The marina basin layout was adopted based on the current layout drawing as shown in **Figure 2**. An entrance 40 m wide and 50 m long was adopted. This is represents a worse case for flushing than a 50 m wide entrance.

5.5 Model Calibration

Once the 2D RMA model was built it was checked to ensure that it behaved correctly and represented the terrain. The model cross sections (specifically bridge structures which can have significant localised affects) were compared against the survey data (obtained from the Georges River Flood Study) and site inspection data and adjusted accordingly so that it correctly represented the terrain.

The Manning's "n" values were also checked and adjusted against site inspection and aerial photos to correctly determine and model the river's behaviour.

5.6 Tidal Exchange

The model results indicated that a high degree of exchange would occur between the river and the marina, due to the relatively wide marina entrance. The river is approximately 80 m wide in this location, while the entrance is at least 40 m wide. The water levels in the marina mimic those in the river, indicating that the entrance does not control flows entering and leaving the marina basin.

5.7 Flow Velocities

The model indicated that flow velocities due to tidal flows were low. Velocities in the marina basin were below 0.05 m/s and velocities in the river adjacent to the proposed marina were generally less than 0.3 m/s.

5.8 Bank Stability

Although the tidal flow velocities are low, other factors also affect bank stability. The 1.5 year ARI flow is considered to be the "bank forming" flow, during which velocities would likely be higher than those during tidal flow conditions. Waves generated by boats on the river also affect bank stability. The bank protection would be designed to withstand these forces.

6. WATER QUALITY

6.1 Introduction

The Georges Cove Marina would be connected to the Georges River. The river encompasses a large part of the Sydney urban area catchment and as such, receives significant pollutant loads from urban runoff and sewage overflows/discharges from the sewerage system and sewage treatment works. The proposed marina water quality will reflect the river water quality because the river flows are enormous in comparison to the volume of water in the marina basin. Nonetheless, the aim is for the marina to provide water quality which is better than the river quality so that it will not preclude the longer term improvement in river water quality. Also, it will be important that the marina does not contribute to make one element of water quality worse in the river. This will be achieved by targeting ANZECC water quality guidelines in the marina suitable for estuarine ecology.

The potential pollutant sources in the marina development are runoff from the land, copper from antifouling paints on the craft hulls and indiscriminate discharge of sewage from craft while at the marina berths. The development would incorporate water sensitive urban design measures to treat runoff prior to entering the marina basin and a sewage pumpout facility would be provided to alleviate unlawful discharges of sewage from craft holding tanks.

The proposed marina on the Georges River was assessed for the following purposes:

- To determine the potential impacts on the water quality in the river; and
- To determine the water quality within the waterbody of the marina.

The key consideration is the introduction of copper, used in antifouling paints, to the aquatic system. The introduction of copper would be limited through the management measures listed in **Section 6.1.1**. The other important parameters are suspended solids, nitrogen and phosphorus. These parameters are to be assessed on two bases:

- Comparison with existing water quality in the river; and
- Comparison with water quality guidelines.

The relevant water quality guidelines are the Australian and New Zealand Environment and Conservation Council (*ANZECC*), *Australian and New Zealand Guidelines for Fresh and Marine Water Quality,* 2000.

6.1.1 Environmental Measures

Specific facilities within the development will reduce the environmental impact of the marina. A brief description of the facilities is provided below:

- The dry storage facility reduces the exposure of anti-fouling paint on craft hulls in the marina;

- The sewage and bilge pumpout facility within the marina will alleviate nutrient loads entering the marina through indiscriminate discharge of sewage from craft while at the marina berths; and
- Water sensitive design elements throughout the development will provide treatment of stormwater runoff and a reduction of pollutant load prior to entry into the marina/Georges River.

Management has decided not to sell anti-fouling paint that contains copper through the marina. This measure will decrease the copper load through the berthing facility.

6.1.2 Guidelines

The ANZECC/ARMCANZ 2000 guidelines aim to maintain or improve the environmental value of natural or semi-natural water resources. The guidelines recommend monitoring a range of indicators, collecting biological effect data for the existing ecosystem and the use of reference sites where no site-specific data is available. The guidelines are recommend tailoring for local conditions. Currently the site of the proposed marina is occupied by a sand extraction operation, including water bodies created through the dredging process. The construction of the marina would result from the sand extraction operation. This investigation compares predicted water quality indicators with default guideline values to assess the water quality performance of the proposed design.

6.1.3 Trigger values

'Trigger values' are defined as numerical values of an indicator (*such as concentration of total copper*) at which some management action is triggered.

As the marina does not exist, indicators were used predict the water quality behaviour. The aim of investigations is to ensure, to the extent possible, that the water quality indicators will be below the trigger values for a sufficient proportion of the time.

In line with current guidelines, the predicted stormwater water quality will need to be modelled and assessed in terms of the concentration of total nitrogen (TN), total phosphorus (TP) and total suspended solids (TSS).

Copper loading from stormwater and copper leaching from antifouling has been calculated in terms of total copper (TCu). The TCu concentrations do not always meet the trigger values so, in accordance with the guidelines, copper speciation in the marina has been modelled and the labile (*bioavailable*) concentration (LCu) compared with the trigger values.

Nutrients

The guidelines give a set of default trigger values for physical and chemical stressors, such as nutrients, below which there is considered to be a low risk to the aquatic ecosystem from the physical and chemical stressors (Table 3.3.2 in ANZECC/ARMCANZ, 2000). The Guidelines are given in terms of the 80th percentile of measurements.

	Total Nitrogen (μg/L)	Total Phosphorus (μg/L)
Estuarine waters	300	30

The estuarine values have been adopted for the marina and the Georges River.

The primary mechanism by which excessive nutrients threaten the aquatic ecosystem is the same as for recreational and aesthetic values - algal blooms. A low risk to the aquatic ecosystem means a low risk of algal blooms and hence low risk also to the recreational and aesthetic values of the water body.

Suspended Solids

Estuaries typically experience a wide range of TSS concentrations. No absolute trigger values for TSS are given in the guidelines in relation to secondary contact recreational use, aesthetic value or risk to aquatic ecosystems. Effects of TSS loading on the aquatic ecosystem and aesthetic value can be discussed in terms of change from pre-development conditions. TSS loads to the marina are also relevant to the fate of copper in the marina.

Copper

The initial indicator for assessing the toxic effect of copper is concentration of total copper. However not all forms of copper are equally toxic. The decision tree for metal speciation given in the guidelines (Figure 3.4.2 in ANZECC/ARMCANZ, 2000) describes how if the total copper concentration fails to meet the trigger value then copper speciation and bonding with sediment can be considered to establish a bioavailable or 'labile' concentration that can be compared to the trigger value.

For protection of aquatic ecosystems the trigger values for concentration of labile copper are given in the guidelines (Table 3.4.1 in ANZECC/ARMCANZ, 2000) and presented in Table 6–1. The values for marine water are applicable to the site. The values for freshwater are shown for comparison purposes.

Table 6–1 – ANZECC Trigger Values for Copper

	Level of Protection (Percent of Species)			
	99%	95%	90%	80%
Trigger values for marine water (μg/L)	**0.3**	**1.3**	**3.0**	**8.0**
Trigger values for freshwater (μg/L)	1.0	1.4	1.8	2.5

For recreational use, the guidelines give a trigger value for copper concentration of 1000µg/L (Table 5.2.3 in ANZECC/ARMCANZ, 2000).

The guidelines do not give specific advice on copper concentrations for protection of human consumers of aquatic food. For the protection of aquaculture species a default trigger value of 5µg/L is given (Table 4.4.3 in ANZECC/ARMCANZ, 2000).

The marina's primary purpose is to provide a facility for recreational boats and the marina is not intended to extend the existing aquatic habitats available in the Georges River. Hence, the 90% or even 80% species protection value (**3.0** *or* **8.0 µg/L**) may be appropriate trigger values for the in-marina water quality. This should allow colonisation of the marina by copper tolerant species.

6.1.4 Water Quality Monitoring

Water quality monitoring of the existing waterbodies has been undertaken, primarily by Marine Pollution Research in 2006. **Table 6–2** lists the results of monitoring by Marine Pollution Research as reported in 2010.

Table 6–2 – Water Quality Test Results by Marine Pollution Research

Analyte**	Units	DetLim	Site	N	Min	Max	Mean	SE
pH	pH units		Bank	48	3.2	6.3	5.0	0.14
pH			Pond	12	6.8	8.7	7.9	0.16
pH			River	24	6.5	7.6	7.3	0.06
Cond	µS/cm		Bank	16	5400	14600	10038	695
Cond			Pond	4	7200	14800	11250	1571
Cond			River	8	8700	21000	16488	1771
TDS	mg/L		Bank	16	4700	10000	7438	441.10
TDS			Pond	4	7300	9600	8450	608.96
TDS			River	8	9600	14000	11950	562.84
Alkalinity	mg/L	< 0.1	Bank	16	0.05	150	32.6	11.86
Alkalinity			Pond	4	110	140	122.5	7.50
Alkalinity			River	8	65	95	75.1	3.41
TOC	mg/L		Bank	16	4	29	13.1	1.42
TOC			Pond	4	16	30	24.3	2.95
TOC			River	8	2	7	3.8	0.56
NH4-N	mg/L	<0.1	Bank	16	0.64	3.8	1.5	0.23
NH4-N			Pond	4	0.042	0.2	0.1	0.04
NH4-N			River	8	0.044	0.05	0.0	0.00

Analyte**	Units	DetLim	Site	N	Min	Max	Mean	SE
NOx	mg/L	<0.005	Bank	22	0.0025	0.22	0.1	0.01
NOx			Pond	4	0.0025	0.43	0.1	0.10
NOx			River	8	0.031	0.62	0.2	0.07
Al	mg/L	< 0.1	Bank	8	0.05	65	17.7	9.84
Al			Pond	2	0.05	0.2	0.1	0.08
Al			River	4	0.05	0.05	0.1	0.00
Cu	mg/L	<0.01	Bank	12	0.005	0.1	0.023	0.01
Cu			Pond	3	0.005	0.005	0.005	0.00
Cu			River	6	0.005	0.005	0.005	0.00
Fe	mg/L	<0.02	Bank	8	22	230	87.4	26.22
Fe			Pond	2	0.05	0.22	0.1	0.09
Fe			River	4	0.01	0.36	0.1	0.08
Mn	mg/L		Bank	16	0.7	4.4	1.8	0.25
Mn			Pond	4	0.04	0.15	0.1	0.02
Mn			River	8	0.02	0.06	0.0	0.01
Pb	mg/L	<0.01	Bank	12	0.005	0.07	0.0	0.00
Pb			Pond	3	0.005	0.015	0.0	0.00
Pb			River	6	0.005	0.015	0.0	0.00
Zn	mg/L	<0.02	Bank	10	0.03	0.3	0.1	0.03
Zn			Pond	0	0	0	0.0	0.00
Zn			River	2	0.01	0.01	0.0	0.00

* Bank = 4 bore sites on riverbank, pond is single dredge pond site and river = two river edge sites

**All other analytes (As, Ba, Cd, Cr, Hg, Se, OC pesticides, PAH & Phenols) were below detection or non-significant.

Additional monitoring was undertaken by quarry staff in 2008 for testing of the various copper components and associated suspended sediment levels in the river adjacent to the site. The samples were tested by the CSIRO laboratory at Lucas Heights. Sampling was undertaken on both spring and neap tides and at both high and low tides to gather the widest possible range of information on water quality at the site. Wet weather sampling was also undertaken for a rain event during which 27.2 mm of rain fell at Liverpool and 32 mm of rain fell at Bankstown. The results of this monitoring are shown in **Table 6–3**. It can be seen from the test results that the water has a dry weather labile copper concentration of 1.5 ± 0.40 µg/L, while combining the dry and wet weather results gives a concentration of 1.4 ± 0.41 µg/L. Thus, the water in the Georges River in the vicinity of the site generally only complies with the copper trigger value for protection of 90% of species.

Table 6–3 – Water Quality Test Results by Benedict / CSIRO

Laboratory I.D.	Sample I.D.	Date Sampled	Time Sampled	Conditions	Total Copper (µg/L)	Dissolved Copper (µg/L)	Labile Copper (µg/L)	Total Suspended Solids (mg/L)
CE5-1	MBK001	16/01/2008	10.10	Low neap tide	3.0	2.4	1.3	8
CE5-2	MBK002	16/01/2008	16.15	High neap tide	2.6	2.0	1.3	8
CE5-3	MBK003	22/01/2008	10.05	High spring tide	3.3	2.2	2.1	15
CE5-4	MBK004	22/01/2008	16.50	Low spring tide	3.0	2.2	1.3	15
CE5-5	MBK005	1/02/2008	12.00	Wet weather	3.0	2.3	1.0	21

6.2 Pollutant Inputs

6.2.1 Stormwater

Runoff from urban areas in the marina's catchment is not the primary source of pollutants entering the marina, but this source has been taken into account in the modelling for nutrients (*nitrogen and phosphorous*), suspended solids and copper.

A MUSIC water quality model was created to determine the pollutant inputs from stormwater in the catchment.

Rainfall data was collected from the *Bureau of Meteorology* (*BoM*) to be used in the MUSIC model. Pluvial rainfall data from Liverpool rainfall station (*station number 67035*) was used as it is the closest rainfall station to the marina and provided all years of data (*excluding 1994 where a complete year was not recorded*) between 1963 and 2000. The average annual rainfall depth for this period was 866 mm.

For this study the pluviograph record from 1st January 1972 through until 31st December 1975 was utilised for the MUSIC modelling because this relatively recent period contained both a dry, average and wet year, and has an annual average rainfall of 885 mm which is close to the long term average for the region.

The default soil parameter values in the MUSIC program were calibrated to reflect the runoff characteristics of the site. The soil parameters used for marina model were adopted from a previous study, where the soil parameters were calibrated to reflect the flood plain characteristics of a nearby site also located on the Georges River. The soil parameters adopted are shown in **Table 6–4**.

Table 6–4 – Adopted Soil Parameters

Parameters	Value
Soil Storage Capacity	120 mm
Initial Storage	30 %
Field Capacity	30 mm
Daily Deep Seepage Rate	30 %

The total rainfall catchment flowing into the marina is enclosed by the site boundaries with a total area of 16.5 ha. The site does not receive any water from any external catchments. The site comprises of 7.5 ha of residential development, 1.53 ha of commercial development and 9.94 ha of open space for the marina.

The development was modelled with general urban pollutant characteristics within the MUSIC model. Based on descriptions of the site, geographical maps and architectural drawings, the proposed development was modelled with impervious percentage of 70%.

The volumetric runoff coefficient (Cv) was assessed to check whether the adopted impervious percentage was acceptable. Calculations showed that an impervious percentage of 70% and the adopted soil parameters yielded a Cv value of 0.65 and a runoff volume of 94 ML/year. This Cv value typically reflects the characteristics of an urbanised site, and concludes that the adopted values in the MUSIC model will produce reliable indicative results. MUSIC results of pollutant discharge are shown in **Table 6–5**. Site runoff will be treated to attain best practice standards of 80% reduction in suspended solids and 45% reduction in nitrogen and phosphorus. It is assumed that copper concentrations would also be reduced through treatment. A reduction rate of 45% was conservatively adopted.

Table 6–5 – Music Results

Pollutant	Load Before Treatment (kg/year)	Minimum Reductions	Maximum Resultant Load (kg/year)
TSS	9,320	80 %	1,864
TP	22.5	45 %	12.4
TN	187	45 %	103
TCu	7.44	45 %	4.1

6.2.2 Discharges from Moored craft

General

The marina is being designed to accommodate 182 small craft and 4 larger vessels as per **Table 3–1**. Of these it is predicted that 176 will be private recreational craft and 10 will be commercial craft. Pumpout facilities as well as associated management practices will be incorporated within the marina to cater for the disposal of bilge water and sewage from craft while at berths. Notwithstanding these precautions the modelling has incorporated allowance for some illegal discharges of sewage and bilge water from craft.

All vessels berthed within the marina are expected to be painted with antifouling paint to slow the growth of marine organisms on the hull. Antifouling paint typically incorporates a biocide that is slowly leached from the paint into the surrounding water. By far the most common biocide currently used on recreational craft and small commercial vessels is copper. Copper leaching from antifouling paint represents a pollution load into the marina, as referred to in Section 6.1.1, anti-fouling paint which contains copper will not be sold at the marina.

Illegal or Accidental Discharge of Sewage and Bilge Water

With the pump-out facilities and management controls, accidental or illegal discharges of bilge water and sewage are expected to be minor. The pollution loadings for these discharges from vessels have been included in the marina water quality modelling.

Vessel usage will vary over the year, with a predicted maximum usage of 40% of craft moored at berths in peak summer periods. If it is assumed that the peak usage rate is extended over the entire year, the estimated weekly pollutant load from accidental sewage would conservatively be:

- Number of craft in the development = 186
- Usage rate of craft in any one week = 40%
- Average number of people on craft = 4
- Number of craft which cause discharges of sewage to the waterway = 20%

- All discharges occur within the marina waterway and not in the river
- Previous studies indicate the following loading factors for the use of toilets:
  - Suspended Solids 50 g/person/day
  - Total Nitrogen 8 g/person/day
  - Total Phosphorus 2 g/person/day

The total number of people who are assumed to contribute to unauthorised sewage discharge during a week can be calculated as follows:

186 craft x 40% occupancy x 4 people x 20% causing discharge = 60 persons

Hence, the pollution loading from accidental sewage discharge weekly by 60 persons would be approximately:

- Suspended Solids 21.0 kg/wk
- Total Nitrogen 3.36 kg/wk
- Total Phosphorous 0.84 kg/wk

The estimated potential quantity of accidental bilge water which may be discharged to the marina has been based on the craft usage noted above and the following conservative assumptions:

- Average craft bilge water pumpout volume per annum is 400 L (i.e. about 8L per week);
- Usage rate of craft in any one week = 40%;
- Number of craft which cause accidental discharge of bilge water = 20%;
- All discharge occurs within the marina waterway;
- Number of craft causing discharge per week = 15;
- Volume of bilge water discharged per week = 120 L.

Typical concentrations in bilge water, based on data collected in an earlier study by Dames and Moore, are as follows (rates for steel-hulled vessels have been conservatively adopted):

- Suspended Solids 355 mg/L
- Total Nitrogen 14 mg/L
- Total Phosphorous 1380 mg/L
- Copper 0.8 mg/L

The conservatively estimated pollutant loads entering the marina waterway in a week due to accidental bilge water discharge are presented below:

- Suspended Solids 42.6 g/wk
- Total Nitrogen 1.68 g/wk
- Total Phosphorous 166 g/wk
- Copper 0.096 g/wk

6.2.3 Copper Leaching from Antifouling Paint

The copper loading from antifouling is a function of the number and size of craft, the usage patterns of the craft, and the copper leaching rate from the antifouling paint.

Vessel Type and Size Distributions

The marina is being designed to accommodate 182 small craft and 4 larger vessels as per **Table 3–1**, with an assumed average wetted surface area of 24.6 m^2.

The distribution of berths in the marina is based on the predicted craft size distribution, taking into account the river location. The adopted craft size distribution is given in **Table 3–1**.

A craft-type distribution was estimated considering that typically most private marinas are dominated by power boats and, due to its location on the Georges River, this marina would have a particular bias towards power boats.

Curves are shown in **Figure W** relating hull length to wetted surface area for traditional long-keeled sailing boats, modern fin-keeled sailing boats, and a high and low curve for power boats which can vary greatly in shape and underwater appendages.

Vessel Usage Patterns

The usage patterns of vessels can have a significant effect on the copper leaching rate from the antifouling paint. Prior studies indicate that:

- most vessels are antifouled once per year;
- in water hull cleaning is either banned or discouraged on environmental grounds;
- most private marinas are dominated by power cruisers;
- on a busy weekend typically 10%-40% of boats may depart the marina;

- marinas surrounded by good cruising grounds, sheltered waterways with plenty of destinations within easy reach tend to have a higher rate of departures;
- up to 60% of vessels leave their berth very rarely or never.

Commercial vessels are predicted to be limited at the marina, due to its location on a river, distant from more popular coastal destinations.

Occupancy rates of 95% and 80% were adopted for private vessels and commercial vessels, respectively. These values are considered conservative.

Copper Leaching Rates

Copper leaching rates from antifouling paint may vary with many factors including the amount of copper in the paint, type of paint, water flow past the paint surface, temperature, salinity and pH of the water, presence of a bio-film on the paint surface, time since application and mechanical cleaning.

An emerging alternative to copper based antifouling systems are silicone based coatings which aim to make the surface so slippery that fouling organisms are washed off as the vessel moves through the water. These systems are considered to be superior from an environmental point of view as the do not include any biocide that is released into the water. However, at the current time and for the foreseeable future, copper based paints are considerably cheaper and more effective and are much more commonly used on both recreational and commercial vessels.

Valkirs et al 2003 conducted an extensive set of experiments to quantify leaching rates under a range of exposure conditions in San Diego Bay. Seven different self-polishing paints and one ablative antifouling paint were tested. Based on this study and other available information, a leaching rate of 7 $\mu g/cm^2/day$ was adopted.

Copper Loading from Antifouling

The copper loading from antifouling is calculated from the number of vessels, vessel size distribution, the wetted surface area curves, the proportion of time the vessels are expected to be in the marina and the adopted leaching rates. The predicted copper loading is 123 kg/yr.

6.2.4 Summary of Pollutant Inputs

A summary of the pollutant loadings adopted for modelling of the marina water quality is presented in **Table 6–6**.

Table 6–6 Summary of Pollutant Inputs to the Marina, ultimate developed scenario (kg/yr)

Pollutant	Load

From Stormwater	
Total Nitrogen	103
Total Phosphorus	12.4
Total Suspended Solids	1,864
Total Copper	4.1
Discharges from Craft	
Total Nitrogen	1094
Total Phosphorus	175
Suspended Solids	52.3
Copper	0.005
Copper from Antifouling	123
Total Copper from Stormwater, Craft Discharges and Antifouling	127

6.3 Flushing

6.3.1 General

Flushing is the exchange of water between the marina and the river, and it is an important mechanism for removal of pollutants from the marina. Natural flushing is driven by tidal flows, wind, density currents, stormwater inflows and diffusion. Tidal and stormwater flows would be the most important processes for the proposed marina.

6.3.2 Tidal Flushing

The proposed marina is situated on the Georges River, connected by a 40 m wide, 50 m long entrance channel. The tidal flushing of the marina is predominant because water will move all the way out of the channel and well beyond in one tidal cycle. There is the potential for up to 40 to 60% of the marina water to be exchanged between each peak and trough of a tidal cycle, varying with the tidal range.

6.3.3 Stormwater Flushing

Stormwater flushing is not considered to be the dominant flushing mechanism for the marina, as tidal flushing would occur daily and thus be more significant to the typical water quality in the marina.

Due to the relatively small catchment size, only 94 ML/year of runoff passes through the marina. This equates to approximately half the volume of the marina. Hence the tidal flushing of 40 to 60% per tidal cycle is of greater significance than 50% per year due to stormwater flushing.

6.4 Water Quality Modelling

Using the results from the hydraulic RMA-2 model, a water quality model describing the transportation and dispersion of pollutants could then be developed using the RMA-11 software.

RMA-11 is a two dimensional water quality and environmental transport software package, which can be used to model the advection-diffusion process in the aquatic environment along with basic algal, nitrogen, phosphorus and oxygen processes. RMA-11 operates on a physical hydrodynamic environment provided by the results of the RMA-2 modelling, that is, it is designed to accept velocity and depth data generated from the two-dimensional hydrodynamic model (*i.e., RMA-2*).

The RMA-11 model indicated that, under the tidal conditions modelled, approximately 53% of a conservative pollutant would be washed out of the marina in a day. This indicates that the marina basin layout and entrance configuration are not constraining tidal flushing.

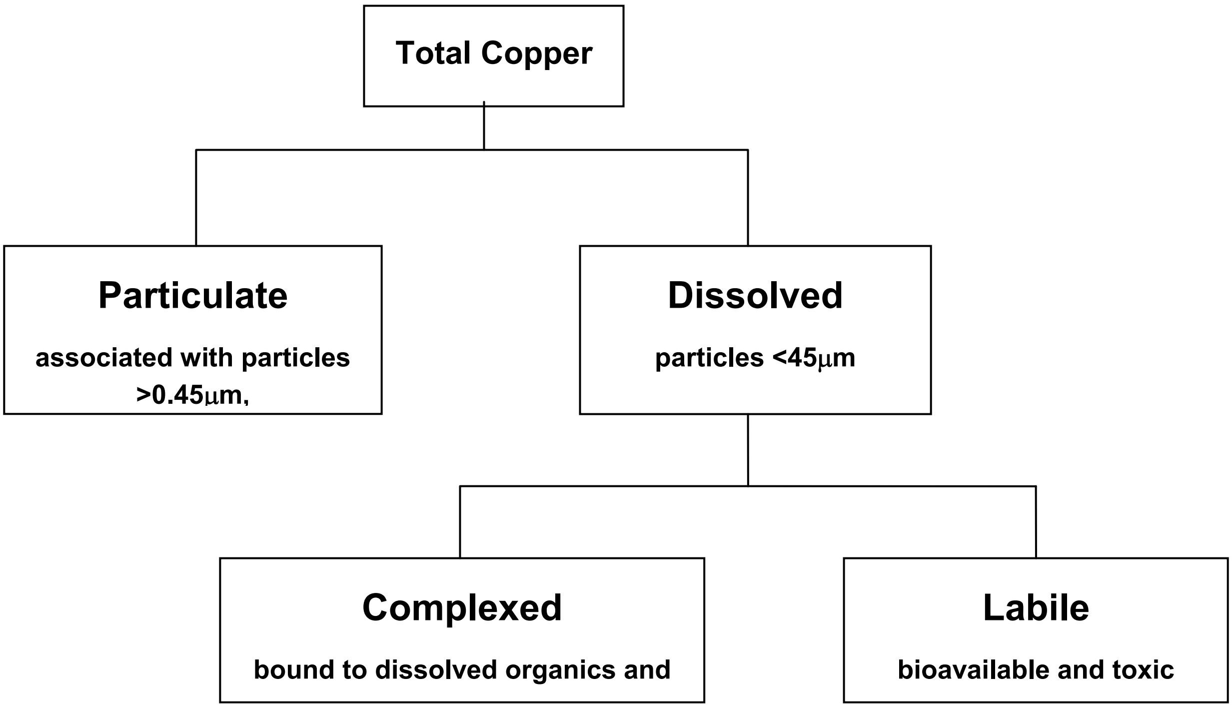
6.5 Copper

6.5.1 Copper Loading

Inputs to the model are average copper loadings from stormwater and vessel antifouling paint as described in **Section 6.1.4**.

6.5.2 Copper Partitioning

The terminology used to describe the copper partitioning is illustrated below.



Following the method of Chadwick et al (2004), the total copper within the water of the marina is divided into dissolved and particulate phases so that:

$$c_t = c_d + c_p$$

where: c_t = concentration of total copper

c_d = concentration of dissolved copper

c_p = concentration of particulate copper

'Particulate copper' is copper associated with particles greater than 0.45 µm diameter and is less toxic. 'Dissolved copper' is everything less than 0.45 µm diameter. A suspended solid dependant equilibrium between the particulate and dissolved copper is assumed such that:

$$c_p = K_d \, c_d \, TSS$$

where: K_d = partitioning coefficient (L/mg)

TSS = total suspended solids (mg/L)

The dissolved phase is further divided into complexed and labile phases:

$$c_d = c_c + c_l$$

where: c_c = concentration of complexed copper

c_l = concentration of labile copper

'Labile copper' is the bioavailable and therefore more toxic phase. 'Complexed copper' is bound to dissolved organics or colloids and is not bioavailable. The partitioning coefficient for the complexed and labile phases varies with the amount of available complexing material in the water. Based on advice from Dr Graeme Batley of CSIRO for coastal/estuarine waters, the complexed copper concentration has been determined as (refer **Appendix**):

$$c_c = 0.65\, c_d \text{, up to a maximum of } 4.0\ \mu g/L$$

6.5.3 Flushing

Flushing exchanges a portion of the water of the marina with water from the river every day, as described in **Section 6.3.2**, so the flux of copper out of the marina due to flushing is:

$$F = f\,V\,(c_t - c_{t,\ background}) \quad \text{(g/day)}$$

where: f = flushing rate (% per day)

V = volume of marina

$c_{t,\ background}$ = Georges River background total copper concentration

<u>Flux to Sediments</u>

Chadwick et al (2004) investigated a uniform first order loss to sediments as well as a simple partial settling model. They found that the particle settling model better predicted the copper concentration in San Diego Bay and this model has been adopted for the marina, so the flux to sediments over the marina is:

$$L = w\,V\,c_p / h \qquad \text{(g/day)}$$

where: w = effective fall velocity (m/day)

h = depth of marina

Chadwick et al acknowledge that this particle settling model is a simplification of the actual sediment-water exchange of copper which results from a number of processes including particle settling and re-suspension, as well as pore water advection and diffusion. Even in existing systems where data can be collected it is difficult to characterise all of these processes so an 'effective' fall velocity which integrates all of the processes is used.

6.5.4 Mass Balance Model of Copper in the Marina Basin

Copper from antifouling paint enters the water as copper ions, a large portion of which quickly binds to suspended solids or forms complexes with dissolved organics and colloids. This leaves only a fraction of the total copper in a bioavailable or 'labile' form. A mass balance model of copper in the marina has been constructed based on the work of Chadwick et al (2004) who modelled the mass balance and fate of copper in San Diego Bay and earlier studies.

The mass balance model considers the marina as a single, well mixed element in steady-state. The model considers the attachment of copper to sediments and settling to the bed, the complexing of dissolved copper and the removal of copper from the marina by tidal flushing.

Evaluation of Parameters in Copper Mass Balance Model

The particulate-dissolved partitioning coefficient, K_d, can be calculated from physical measurements in an existing system of interest but the effective fall velocity, w, is to a large extent a calibration parameter. Calibration of the model was based on the results of calibration in other projects.

The particulate-dissolved partitioning coefficient, K_d for the Georges River averaged 0.033 for dry weather events.

San Diego Bay

Chadwick et al (2004) calibrated their model of San Diego Bay to 4 sets of measured data. Their best-fit parameters are given below in **Table 6–7**:

Table 6–7 Best-fit parameters for San Diego Bay (from Chadwick et al 2004)

	Combined	Aug 2000	Jan 2001	May 2001	Sept 2001
K_d (L/mg)	0.1	0.22	0.06	0.05	0.08
w (m/day)	1.4	0.85	4.8	3.3	1.3

It was noted that the two parameters were not independent, with high partitioning coefficients occurring with low effective fall velocities and vice versa. It is thought that the sediment in the bay in May 2001 was coarser and less reactive than the sediment in August 2000. The difference may be explained by the variation in river and stormwater flows entering the bay. Coarse sediments have proportionally less surface area by weight and therefore less affinity for copper (*smaller* K_d) and higher fall velocities than fine sediment. Organic sediment tends to have a higher affinity for copper than inorganic sediment.

Other Systems

Chadwick et al. (2004) also give some values for dissolved-particulate copper partitioning coefficients determined from other studies:

Location	K_d (L/mg)
Melbourne	0.076
Southern coast of England	0.0003-0.12
San Francisco Bay	0.07
Galverston Bay, Texas	0.03

Proposed Marina

The sources of sediment to the marina will be the river, treated local catchment stormwater, resuspension of sediments on the bed stirred up by craft movement, wave action and discharges from vessels. Much of the stormwater will have passed through a treatment train consisting of devices such as wetlands, swales and gross pollutant traps. Furthermore, the stormwater flushing is low relative to tidal flushing. It is therefore expected that the river will be the primary source of the suspended solids entering the marina.

The concentration of suspended solids (TSS) in the marina is an important parameter for prediction of copper partitioning. TSS in the marina has been established by measurements taken in the Georges River and taken as the average of dry weather conditions. The concentration as well as the characteristics of the suspended solids varies with the antecedent rainfall conditions. From the available data on TSS in the Georges River a value of 12 mg/L was chosen to represent average conditions. This value is conservative for modelling purposes, as a higher TSS concentration would lead to a greater loss of copper from the water column through sedimentation.

Table 6–8 Copper Mass Balance Parameters

Conditions	K_d (L/mg)	w (m/d)	TSS (mg/L)
average	0.033	1.4	12

The Georges River background total copper concentration as averaged from **Table 6–3** was 3 μg/L.

PREDICTED COPPER CONCENTRATIONS IN THE MARINA

The predicted copper concentrations are presented in **Table 6–9**. The labile copper concentration with TSS varies significantly and is most sensitive to the concentration of suspended solids during dry weather conditions. For average conditions, the labile copper concentration is predicted to meet the target for 90% of species (3 μg/L), just failing to meet the 95% target of 1.3 μg/L.

Table 6–9 Predicted Copper Concentrations

Case	Total (μg/L)	Particulate (μg/L)	Dissolved (μg/L)	Complexed (μg/L)	Labile (μg/L)
Average	5.82	1.65	4.17	2.71	1.46

6.5.5 Sensitivity Analysis

Sensitivity analyses were carried out as presented below.

Tidal Flux

The hydrodynamic model predicted a tidal exchange rate between the marina and the river of 53% of the marina water per day. The following table indicates that an extreme variation in the tidal flux value would be required in order to have a significant impact on the predicted labile copper concentration, in consideration of the monitored range of labile copper in the river (1.4 ± 0.41 µg/L).

Table 6–10 – Tidal Flux Sensitivity Analysis

Tidal Flux (%)	Predicted Labile Copper Concentration (µg/L)
30	1.79
40	1.59
50 (adopted value from hydrodynamic model)	1.46
60	1.36
70	1.29

Marina Occupancy

Occupancy rates of 95% for recreational vessels and 80% for commercial vessels were adopted for modelling. If a boat leaves the marina for two of the seven days of the week, it would occupy the marina 71% of the time. If a boat leaves the marina for a long day out (say 12 hours), it would occupy the marina 93% of the time. Model runs were carried out as below to represent both higher and lower occupancy rates than those adopted in the modelling.

Table 6–11 – Marina Occupancy Sensitivity Analysis

	Low Occupancy	Adopted Value	High Occupancy
Recreational Vessel Occupancy %	71	95	100
Commercial Vessel Occupancy %	50	80	93

Total Copper from Antifouling (kg/year)	89.7	123	131
Total Copper from Stormwater (kg/year)	4.1	4.1	4.1
Total Copper Discharges from Craft (kg/year)	0.005	0.005	0.005
Total Copper from Stormwater, Craft Discharges and Antifouling (kg/year)	93.8	127	135
Predicted Labile Copper Concentration in Marina (μg/L)	1.23	1.46	1.52

The occupancy sensitivity analysis indicates that modelling increased values for occupancy would not have a significant effect on the percentage of species protected as the predicted labile copper concentration lies within the existing monitored range in the river.

Boat Numbers

A sensitivity analysis for the number of boats was undertaken as presented in the following table

Table 6–12 Boat Number Sensitivity Analysis

	Low	Adopted	High
Number of Recreational Craft	164	176	200
Number of Commercial Craft	9	10	11
Total Copper from Antifouling (kg/year)	114	123	139
Total Copper from Stormwater (kg/year)	4.1	4.1	4.1

Total Copper Discharges from Craft (kg/year)	0.005	0.005	0.005
Total Copper from Stormwater, Craft Discharges and Antifouling (kg/year)	118	127	143
Predicted Labile Copper Concentration in Marina (µg/L)	1.40	1.46	1.57

The results indicate that the predicted labile copper concentration is not highly sensitive to the number of boats in the marina, as the predicted high and low concentrations still lie within the current monitored range of copper in the river (1.4 ± 0.41 µg/L).

6.5.6 Summary of Copper in the Marina

The toxicity of copper varies as copper is present in different chemical states. Of most concern to the aquatic environment is labile copper, which is a bioavailable form of copper, The ANZECC guidelines state trigger values for the concentration of labile copper in an aquatic environment, which correlate to a percentage of aquatic species that will be protected at this value.

As the marina's primary function is to provide a facility for recreational boats and not to extend the existing aquatic habitats which are available in the Georges River, 90-95% of species protection (which equates to a labile copper concentration of 1.3 – 3 µg/L) would be appropriate for in-marina water quality. Labile copper in the Georges River has been measured at 1.0 - 2.1 µg/L.

To calculate the quantity of copper entering the marina for comparison to the ANZECC trigger levels the following information was utilised and incorporated into a mass balance assessment:

- Sources of copper into the marina are stormwater runoff (this is treated by water sensitive urban design measures), illegal bilge discharge from moored craft and leaching from copper-based anti-fouling paint on craft hulls;
- The leaching from anti-fouling paint varies as a function of the number of boats in the marina and their size, usage patterns of craft and the leaching rate of paint;
- Tidal flushing provides an exchange of 40-60% of the water in the marina;
- The river is expected to be the main source of TSS in the proposed marina. TSS in the marina is important for the prediction of copper partitioning. The value of 12 mg/L was adopted as an average river value, based on water quality measurements. On average, the background level of total copper in the Georges River is 2.98 ±0.25 µg/L.

Using the above information in Chadwick's model, the quantity of labile copper in the marina is predicted to be 1.46 µg/L. Based on the ANZECC guidelines, a concentration of 1.46 µg/L of labile copper correlates to the protection of 90-95% of aquatic species, and also indicates no change from the monitored range of copper concentrations in the river (1.4 ± 0.41 µg/L).

6.5.7 Conclusion

This analysis demonstrates that development of the marina would not have a significant impact on the existing copper concentrations in the Georges River and the predicted concentrations would allow 90 to 95% of species to inhabit the marina in accordance with the ANZECC/ARMCANZ (2000) guidelines for the protection of aquatic ecosystems.

6.6 Acid Sulfate Soils

Management of Acid Sulfate Soil is an important aspect of the construction and operation processes. A geotechnical investigation would be required to identify the presence of Acid Sulphate Soils on the site, and within the area where works would occur. If Acid Sulfate Soil is present on-site, an Acid Sulfate Soil Management Plan would be required.

Acid Sulfate Soil is the common name given to sediment and soil containing iron sulfide. The exposure of iron sulfides to air will result in oxidation and the generation of sulfuric acid. Acid leachate can strip metals such as aluminium and iron from the soil matrix and release them into water bodies. Toxic concentrations of these metals will affect water quality and adversely affect aquatic organisms (disease or death) that inhabit the water body.

When saturated mud, gravel or sand containing iron sulfides is disturbed by excavation, dredging or dewatering and exposed to air, the generated acid leaches from the soil (Acid Sulfate Soils Planning Guidelines, 1998). Acid leachate can cause severe environmental degradation and/or contamination. In discussion of acid sulfate soils the following definitions are important (ASSMAC, 1998):

Acid Sulfate Soils (ASS) include actual acid sulfate soils and potential acid sulfate soils. Actual and potential acid sulfate soils are often found in the same soil profile, with actual acid sulfate soils generally overlying potential acid sulfate soil horizons.

Actual Acid Sulfate Soils are soils containing highly acidic (pH ≤ 4) soil horizons or layers resulting from the aeration of soil materials that are rich in sulfides, primarily iron sulfide. This oxidation produces hydrogen ions in excess of the sediments capacity to neutralise the acidity of the soil. These soils can usually be identified by the presence of pale yellow mottles and coatings of jarosite.

Potential Acid Sulfate Soils are soils which contain iron sulfide material which have not been exposed to air and oxidised. However they pose a considerable environmental risk when disturbed, as they will become severely acidic when exposed to air and oxidised. Exposure of acid sulfate soils to the atmosphere (lowering of the watertable or disturbance through dredging/excavation) has the potential to produce acid generating conditions that may adversely affect the local environment.

The Acid Sulfate Soils Planning Guidelines (1998) have a hierarchy of management techniques for dealing with ASS. These are listed as follows:

1. **Avoid land where acid sulfate soils occur**

   If the preliminary soil survey indicates that the site contains high levels of acid sulfate soils, the most environmentally responsible action may be to investigate alternative feasible sites that meet the operational needs of the applicant.

   This principle applies equally when selecting routes for drains, roads or pipelines or for individual sites for residential developments, infrastructure projects, agricultural enterprises or quarries. In the case of quarries, dredging or other operations which have the potential to result in moving acid sulfate soils problems on to another site, the onsite mitigation measures prior to transport plus the cost of quality assurances programs will need to be factored into the project along with the costs associated with the liability for damages if acid is generated at the other site.

2. **Avoid disturbing ASS if present on land**

   To develop effective avoidance strategies, a more detailed investigation is required to understand the soils, surface and sub-surface water characteristics on the site and the sensitivity of the surrounding environment. In many cases, the site should be mapped indicating the depth to sulfide material and groundwater and the variation in the soil characteristics including the concentration of the sulfidic material. The advantage of an "avoidance" approach is that there is no ongoing mitigation required. Possible avoidance mitigation options include the following options.

   a. Undertake shallow soil disturbance so as not to disturb acid sulfate soils;

   b. Redesign existing drains so they are shallow and do not disturb acid sulfate soils

   c. Avoid activities which result in the fluctuation of groundwater, in particular the lowering of groundwater

   d. Cover acid sulfate soils with clean fill material so as not to disturb them

   e. Set aside acid sulfate soil areas and not disturb them

   f. Set aside highest sulfide areas and disturb only the lowest

3. **Prevent the oxidation of sulfide**

   Mitigation strategies to prevent oxidation depend on maintaining the sulfidic material in an anaerobic environment. However, soils or soil layers with existing acidity from previous oxidation of sulfide (indicated by field pH_F < 4.5) are more difficult to prevent further oxidation by denial of oxygen alone, as oxidation may proceed by electron transfer between compounds at different oxidation states. Usually some addition of a neutralising agent will also be necessary when acidity has already been produced.

   a. Stage projects to prevent oxidation
   b. Place any excavated sulfidic material immediately under water
   c. Raise the watertable to maintain potential acid sulfate soils in a saturated state
   d. Cap the acid sulfate soil material

4. **Oxidation of sulfide and neutralising acid as it is produced**

   The most common acid sulfate soils mitigation methods relies on providing sufficient neutralising agent to neutralise acid as it is produced over time due to the gradual oxidation of acid sulfate soils. Most mitigation strategies will result in a certain amount of oxidation of acid sulfate soils either deliberately or inadvertently. In most cases, the natural buffering capacity of the system will initially contribute to the neutralisation of acid produced. However, depending on the sulfide content, substantially more neutralising material usually needs to be added.

   a. Oxidation of sulfide and neutralising using lime or similar agents
   b. Neutralisation using the buffering capacity of estuarine water
   c. Vertical mixing and neutralisation using the buffering capacity of soil

5. **Separate out and treat the sulfidic component**

   With some types of sediments extracted by dredging, it may be possible to partially or fully separate the acid sulfate fines from the sand resource by mechanical methods such as sluicing or hydrocycloning techniques. The method is a particularly attractive mitigation option when full separation can be easily achieved, as the resource can be considered to be “clean” and require the addition of little or no neutralising agent prior to use.

7. NAVIGATION

7.1 Marina Entrance

The proposed marina is recessed into the riverbank and entrance to the river would be via a short entrance channel (approximately 50 m long). Navigation through the entrance would be available at all states of tide as sufficient water depth would be provided at low water for the types and size of vessels likely to use this facility.

Due to the location of the marina entrance, within the speed boat area, measures are proposed to maintain safety and minimise the impact on existing river users. Navigation markers could be positioned near the marina entrance to denote a specific entry channel and keep passing boats away from the entrance side of the river. The design and location of the navigation markers would be undertaken in close consultation with NSW Maritime.

Signs would also be used to alert those leaving the marina to proceed with caution at a maximum of 4 knots onto the river. River users could also be alerted to the marina entrance by appropriate signage.

7.2 Impact on Bank Stability

The marina would result in additional craft using the adjacent stretch of water in the Georges River compared to current conditions, however this would not necessarily impact the river banks, as the reach is already currently heavily used at high speeds.

The marina design incorporates bank stabilisation works, which would be an improvement on the existing scenario, where the river bank is eroding. The bank stabilisation works would be undertaken in consideration of the boat generated waves in the river, hence resulting in a more stable stretch of foreshore.

8. SUMMARY AND CONCLUSION

The proposed design of the Georges Cove Marina has incorporated a number of measures to address flooding, tidal hydrodynamics, water quality and navigation issues. These are summarised below:

- Floor levels of the building and marina facilities have been determined utilising the 100-year ARI flood levels produced in the Georges River Flood Study (adopted by Liverpool Council) and incorporate the appropriate amount of freeboard;
- Tidal hydrodynamics have been assessed through hydraulic modelling. There is a high degree of tidal exchange between the marina and the river, this is not significantly affected by varying the entrance channel between 40 and 50 metres in width.
- Based on water quality testing, the water quality in the quarry is of a similar quality to that in the Georges River. Breakthrough of the bank from the marina to the Georges River would not significantly affect the water quality of either bodies;
- Pollutants sources for the marina are stormwater runoff, illegal discharges of sewage and bilge water, copper leaching from anti-fouling paint on boat hulls and sediment from the Georges River.
  - The incorporation of water sensitive urban design elements provides stormwater treatment to reduce nitrogen, phosphorus, TSS and heavy metals;
  - Bilge and sewage pump-out facilities would be provided in the proposed marina. It is expected that there will still be a small amount of illegal discharge. This pollutant load was modelled and due to the high flushing rate of the marina through the tidal cycle, nutrients will not accumulate in the basin; and
  - Modelling of copper concentrations indicates that the development of the marina would not have a significant impact on the existing copper concentrations in the Georges River and the predicted concentrations would allow 90 to 95% of species to inhabit the marina per the ANZECC/ARMCANZ (2000) guidelines for the protection of aquatic ecosystems.
- Navigation through the channel to enter the marina would be available through all states of the tide. Measures are proposed to maintain safety in the area as the entrance of the marina is within a speed boat area;

The marina design incorporates bank stabilisation works; currently there is no bank protection and erosion is occurring.

The proposed marina basin, with appropriate design and mitigation measures as recommended in this report would provide a facility which would meet the community environmental standards and would provide a recreational asset, which is in short supply along the Georges River.

9. REFERENCES

1. Australian Standard AS 3962 – 2001, Guidelines for Design of Marinas
2. ANZECC/ARMCANZ (2000) *Australian and New Zealand Guidelines for Fresh and Marine Water Quality*
3. Public Works Department (June 1990) *The East Hills Bank Erosion Study, Georges River*
4. Public Works Department (February 1991) *The Georges River Database Compilation and Assessment*
5. Bewsher Consulting (2000) *Georges River Flood Study*
6. Chadwick D, Zirino A, Rivera-Duarte I, Katz C and Blake A (2004) *Modelling the mass balance and fate of copper in San Diego Bay*, Limnology and Oceanography, vol.49(2), 355-366
7. Marine Pollution Research (2010) *Georges River Marina, Moorebank Aquatic Ecology Aspects & Environmental Assessment Of Marina Concept Design Report Prepared For Benedict Industries Pty Ltd*
8. Valkirs A, Seligman P, Haslbeck E and Caso J (2003) *Measurements of copper release rates from antifouling paint under laboratory and in situ conditions: implications for loading estimation to marine water bodies* Marine Pollution Bulletin 46

Figures

FIGURE 1

LEGEND

EXISTING CONTOURS

Numbers

Carparking		Number
Carpark A		70 on-grade car spaces
Carpark B		133 on-grade car spaces
Carpark C		70 on-grade car spaces
Carpark - Dry Store		216 under-cover car spaces
Total Car Spaces		**489**

Marina	Length	Number
Small Craft (<15m)	≥ 8m < 10m	52 Marina Berths
Small Craft (<15m)	≥ 10m < 12m	110 Marina Berths
Small Craft (<15m)	≥ 12m < 15m	20 Marina Berths
Vessels	≥ 15m < 18m	2 Marina Berths
Vessels less than	20m	2 Marina Berths
Vessels	20m or more	0 Marina Berths
Total Marina Berths		**186**

Dry Store	Length	Number
Small Craft (<15m)	≥ 4m < 6m	125 Dry Berths
Small Craft (<15m)	≥ 6m < 8m	125 Dry Berths
Vessels	≥ 15m	0 Dry Berths
Total Dry Berths		**250**

Note Berths & fairways have been designed in accordance with AS. 3962-2001

WorleyParsons

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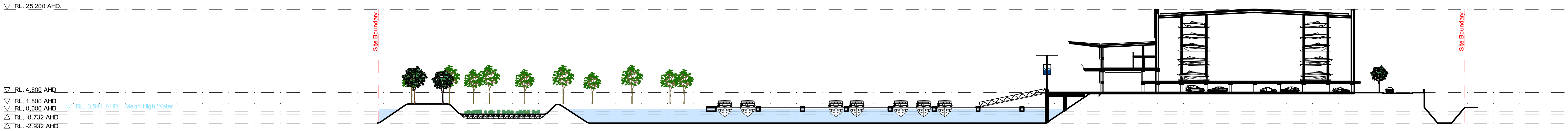
EcoNomics

A		
ISSUE	DATE	ISSUE DESCRIPTION

MARINA BASIN LAYOUT

LOCATION: W:_Infrastructure\Projects\301015\01167 - BS&G Marina\Drawings\SKETCHES\01167 FIGURE 1.dwg DATE & TIME: 30-Sep-10, 1:58:00 PM

FIGURE 2



01 Section AA
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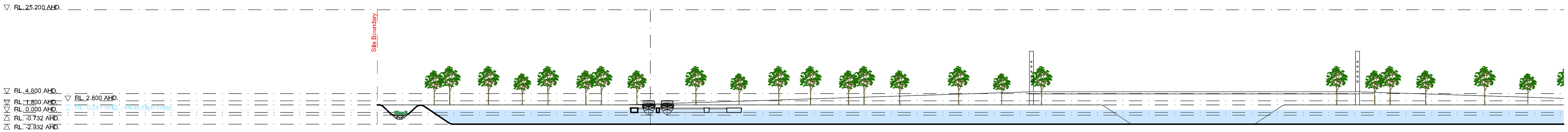
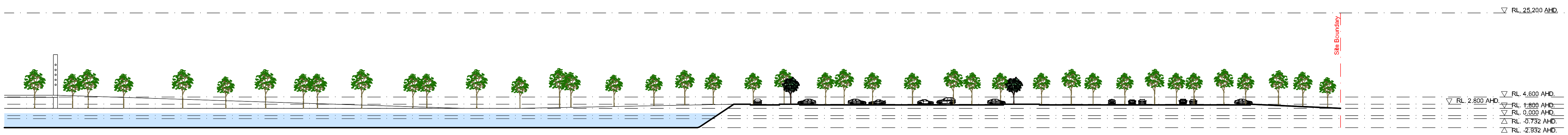


02 Section BB - North
1 : 500



03 Section BB - South
1 : 500

0 10 20 30 40 50m

1:500(A1) 1:1000(A3)

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EcoNomics

A		
ISSUE	DATE	ISSUE DESCRIPTION

SITE SECTIONS

LOCATION: W:_Infrastructure\Projects\301015\01167 - BS&G Marina\Drawings\SKETCHES\01167 FIGURE 1.dwg

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Appendix B2

National Project Consultants, Re Georges Cove Marina – New development application, March 2015

management consultants &
project managers

npc

27 March 2015

Liverpool City Council
Locked Bag 7064
Liverpool BC, NSW 1871

Attention: Rajendra Autar

Dear Sir,

Re Georges Cove Marina – New development application

Benedict Industries proposes to submit a new development application for the Georges Cove Marina. The proposal is exactly identical, in flooding terms, to the one considered and approved by the JRPP in August 2014.

The flood modelling work undertaken for the original approved marina proposal therefore remains contemporary and applicable to this new application for the marina.

The attached flood reports to support the new application for the marina therefore comprise the following:-

- Attachment 1: Assessment of the Flood Impact of the Proposed Bridge on Flooding in the Vicinity of the Georges Cove Marina (Cardno, May 2014)
- Attachment 2: Flood Risk Management Report (npc, November 2013)
- Attachment 3: Flood Impact Assessment for the Proposed Georges Cove Marina, Moorebank (Cardno, January 2013).

The flood impacts are unchanged from the approved marina and will remain negligible.

Yours faithfully,

M Tooker

Mark Tooker

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po box 1060 crows nest nsw 1585 australia
www.npc.com.au
national project consultants pty ltd abn 40 004 004 160

ATTACHMENT 1

Assessment of the Impact of the Proposed Bridge on Flooding in the Vicinity of the Georges Cove Marina, Moorebank (Cardno, May 2014)

Our Ref: NA49913037:BCP/bcp
Contact: Dr Brett C. Phillips

23rd May 2014

The Manager
npc
PO Box 1060
CROWS NEST NSW 1585

Attention: Mr Mark Tooker

Cardno (NSW/ACT) Pty Ltd
ABN 95 001 145 035

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203 Pacific Highway
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Dear Mark,

ASSESSMENT OF THE IMPACT OF THE PROPOSED BRIDGE ON FLOODING IN THE VICINITY OF THE GEORGES COVE MARINA, MOOREBANK

In 2012 Cardno was commissioned by npc to prepare a flood model to undertake a flood impact assessment of the proposed Georges Cove Marina development in Moorebank, within the Liverpool City Council Local Government Area (LGA).

Cardno has been requested to present the flood impact detailed for the proposed road bridge crossing located to the north of the Georges Cove Marina site. The bridge is proposed to connect Brickmakers Drive to the Benedict site and crossing over the access road to the Moorebank Recyclers site.

A previous concept design of the bridge with four 18m spans (total span of 72m) was approved by Council. The updated design modelled within this assessment is a clear 32m span covering the access road reserve and adjacent open channel.

1. ASSESSMENT SCENARIOS

The proposed location and design layout of the road bridge is shown in **Figure 1**.

The bridge crossing has been modelled in accordance with the concept design (Drawing no. 101015-00561-ci-fig3, Worley Parsons). The following details were adopted for the purpose of the hydraulic modelling assessment:

- The bridge has a clear span with the underside of the bridge higher than the 100 yr ARI flood level (5.52m AHD) so that the waterway opening is not pressurised during a 100 yr ARI flood and the bridge deck can be omitted from the hydraulic model. Consequently the hydraulic roughness of the bridge waterway remained unchanged from existing conditions; and
- The footprint of the proposed embankment for the bridge extends as in the above design plans and as shown schematically in **Figure 1**.

The following scenarios were assessed under developed conditions without and with the bridge crossing:

- A 100 yr ARI flood in the Georges River;
- The 100yr ARI local storm event in combination with two Georges River flooding scenarios:
  - a 20 yr ARI Georges River flood; and,
  - a Georges River base flow of 200 m^3/s.

No blockage has been adopted within the hydraulic modelling for the bridge as the clear span is 32m, which is significantly greater than the 18m spans within the previous bridge design. This approach is in accordance with blockage assessment techniques set-out in AR&R Project 11 Stage 2 Report (Engineers Australia, 2013).

2. FLOOD IMPACT ASSESSMENT

The peak flows under each scenario are summarised in Table 1 while Table 2 summarises the flood level and peak flow velocity at two reference locations. These are located just upstream of the proposed bridge crossing (Location 1) and downstream of the stormwater outfall south of Newbridge Road (Location 2).

Table 1 Peak Flows (m^3/s)

Scenario	Georges River	Local Catchment
1	2,081	0*
2	1,567	35.8
3	200	35.8

* In a 36 hour 100 yr ARI storm the estimated peak runoff from the local catchment was only around 5 m^3/s consequently this local inflow was not included in Scenario 1

Under Scenario 2 the runoff from the local catchment in a 100 yr ARI storm equates to 2.3% of the 20 yr ARI peak flow in the Georges River while under Scenario 3 the runoff from the local catchment in a 100 yr ARI storm equates to 18% of the 200 m^3/s base flow in the Georges River.

Table 2 Peak Water Level (m AHD)

Scenario	Location 1			Location 2		
	Without Bridge	With Bridge	Difference	Without Bridge	With Bridge	Difference
Peak Water Level (m AHD)						
1	5.52	5.52	0.00	5.52	5.52	0.00
2	4.60	4.60	0.00	4.60	4.60	0.00
3	3.33	3.39	0.06	3.56	3.59	0.03

It is noted that under Scenario 3 the peak water level is 1.9 m to 2.2 m lower than the 100 yr ARI flood level.

2.1 Flood Impacts during a 100 yr ARI Georges River Flood

The peak water level differences during a 100 yr ARI flood in the Georges River (Scenario 1) resulting from the proposed bridge are shown in **Figure 2**. It can be seen from Figure 2 and Table 2 that the impact of the proposed bridge crossing on the 100 yr ARI flood level is negligible (ie. <0.01 m). This is because the 100 yr ARI flood level is governed by backwater flooding from the Georges River.

The peak flow velocity differences during a 100 yr ARI flood in the Georges River resulting from the proposed bridge are shown in **Figure 3**. There are minor peak velocity increases (<0.04 m/s) in the vicinity of the bridge span, with minor decreases in velocity (0.04 m/s) for an area south of the bridge.

2.2 Flood Impacts during a Local Catchment 100 yr ARI Storm

In response to preliminary comments received from Liverpool City Council dated 13 May 2014, local catchment flooding has been assessed in accordance with the methodology outlined in the Moorebank Recyclers Flood Impact Assessment (WMA Water, 2013).

A hydrological model of the local catchment was prepared using **xprafts** based on the details given in Section 4.2 of the Moorebank Recyclers Flood Impact Assessment report (WMA Water, 2013). The hydrological model was run and the 100yr ARI flows from the catchment were found to match the peak flow of 35.8 m^3/s reported in the 2013 study.

The flow hydrograph for the local catchment was input in the hydraulic model at the stormwater outfall location identified in **Figure 4** (immediately south of Newbridge Road).

The 100yr ARI local storm event was assessed in combination with two Georges River flooding scenarios, namely in combination with:

- a 20yr ARI Georges River flood (where the peak flows from the local catchment was delayed so that it coincided with the peak flow in the Georges River) (Scenario 2); and,
- a constant Georges River base flow of 200 m^3/s (Scenario 3)

Under Scenario 2 the peak water level differences are shown in **Figure 4** while the peak velocity differences are shown in **Figure 5**.

These results show negligible water level impacts (<0.01m) resulting from the bridge structure as the runoff in a local 100 yr ARI storm has negligible impact on the floodplain compared to the flooding caused by the 20 year ARI flood in the Georges River. It was also found that there are minor peak velocity increases (<0.04 m/s) in the vicinity of the bridge span, with significant decreases in velocity (0.15 m/s) occurring in an area south of the bridge embankment.

Under Scenario 3 the peak water level differences are shown in **Figure 6** while the peak velocity differences are shown in **Figure 7**.

It is noted that the water level increases locally upstream of the bridge by up to 0.1 m in the floodplain, with minor increases (<0.02 metres) further upstream within the Newbridge Road reserve to the north. There are minor peak velocity increases (<0.05 m/s) in the vicinity of the bridge crossing with significant decreases in velocity (0.8 m/s) for an area south of the bridge embankment.

It is noted that under Scenario 3 the peak water level is 1.9 m to 2.2 m lower than the 100 yr ARI flood level.

The water level increases of 0.1 metres for local catchment flooding are considered minor as:

- They do not represent a significant alteration of flow regime when considering the negligible velocity impacts for the area; and,

- They do not have any impact on compliance with Council's development controls because the 100 yr ARI is governed by flooding from the Georges River and the bridge crossing has negligible impacts in this event.

2.3 Flood Storage

The estimated loss of flood storage in a 100 yr ARI Georges River flood due to the construction of the road embankment is estimated to be around 4,780 m3 (in comparison with an indicative flood volume of 12.8 million m3) ie. the loss of flood storage equates to 0.0037% of the 100 yr ARI flood volume.

Note that the previously approved bridge design resulted in a flood storage loss of approximately 1,960m3, therefore the updated bridge design results in an increase in flood storage loss of approximately 2,820m3 over the previous approved design. i.e. the impact of the new design is 0022% of the 100 yr ARI flood volume.

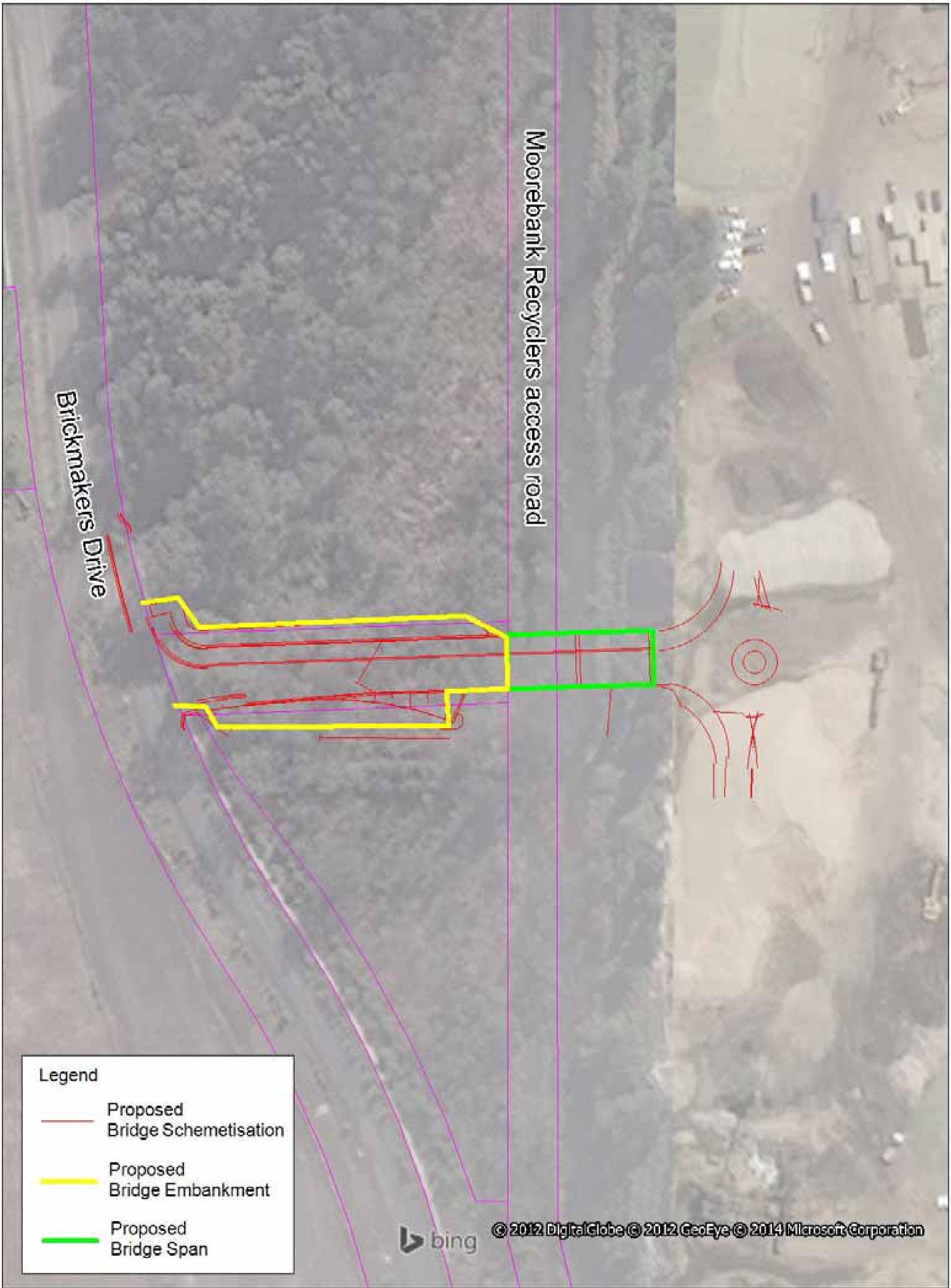
We note from previous correspondence that Council's default position in relation to applications which have some impact on flood storage but do not carry out a detailed Flood Impact Assessment is to require compensatory earthworks or the like to compensate for any loss of storage. The point of undertaking a Flood Impact Assessment is to determine if any loss in floodplain storage has any impact. In this instance, the Flood Impact assessment has clearly shown that the impact of the loss of such a small amount of flood storage has no impact and in our view there is no justification to require works to balance these storage losses.

Yours faithfully

Brett C. Phillips

………………………………

Dr Brett C. Phillips
Director, Water Engineering
for **Cardno**

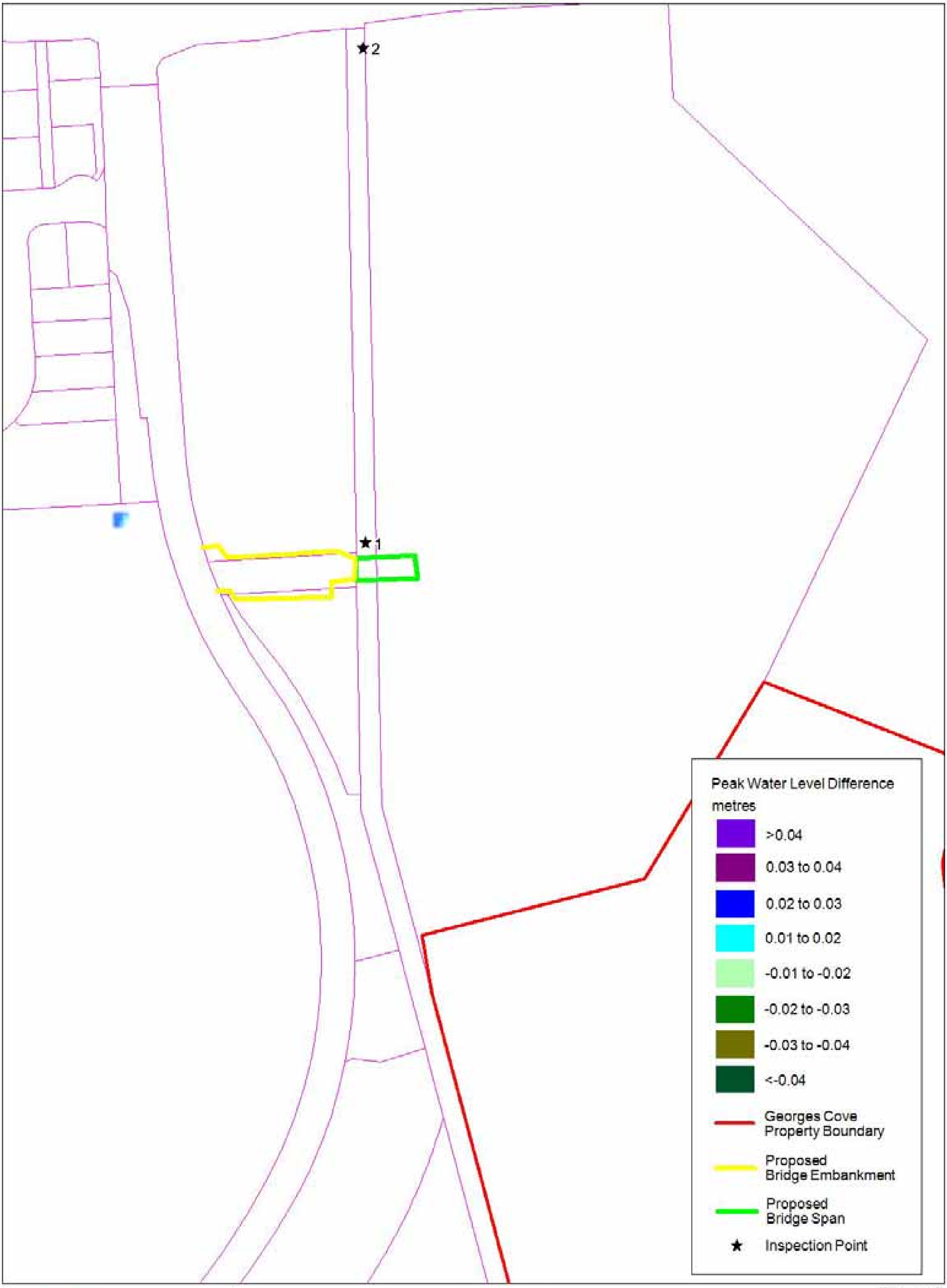


Cardno

Georges Cove
Flood Impact Assessment

Figure 1
Proposed Road Bridge

NA49913037
May 2014



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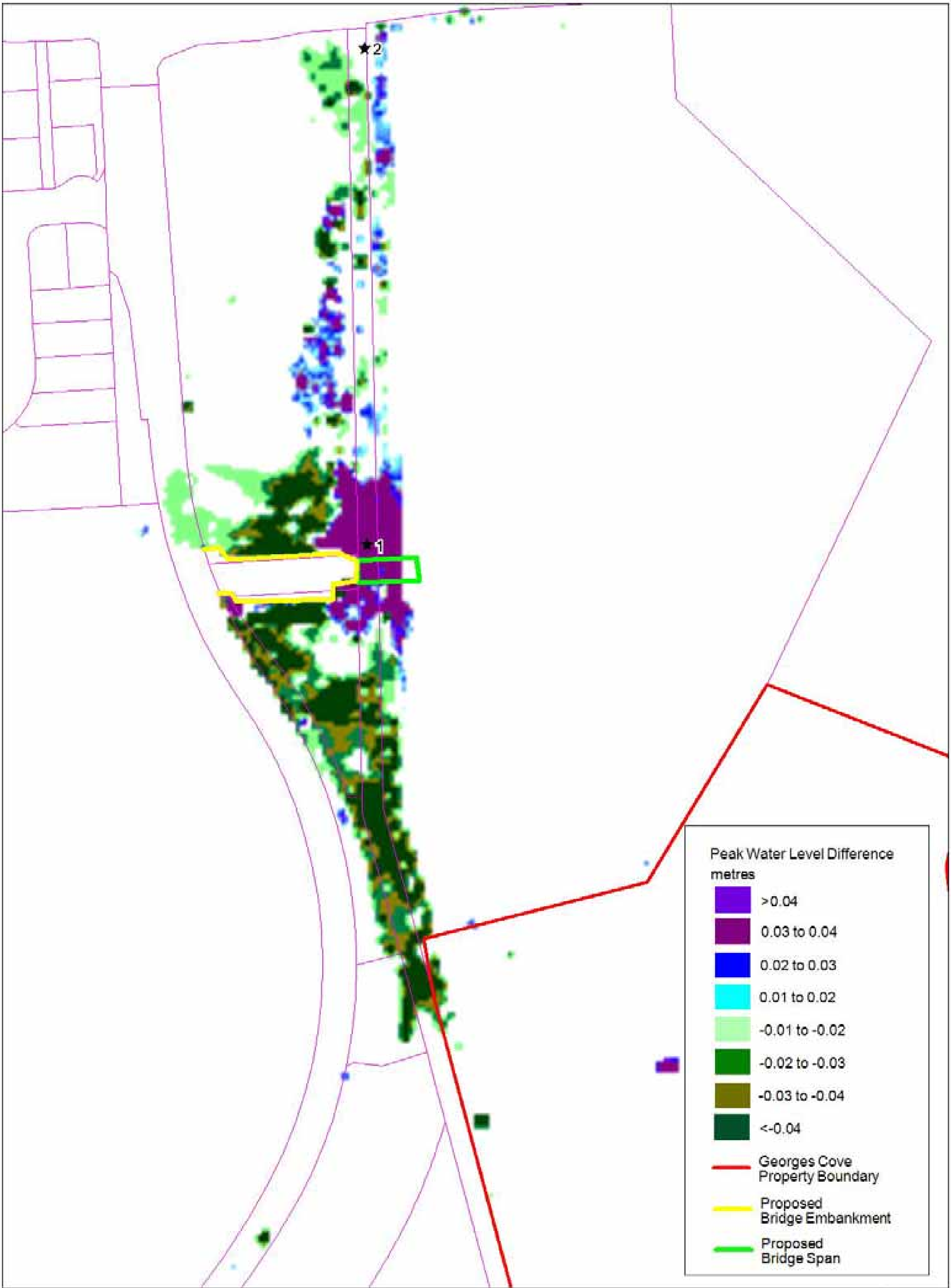
NA49913037
May 2014

Georges Cove
Flood Impact Assessment

Figure 2
100 year ARI Georges River Flooding
Peak Water Level Differences
Post-Development - Road Bridge



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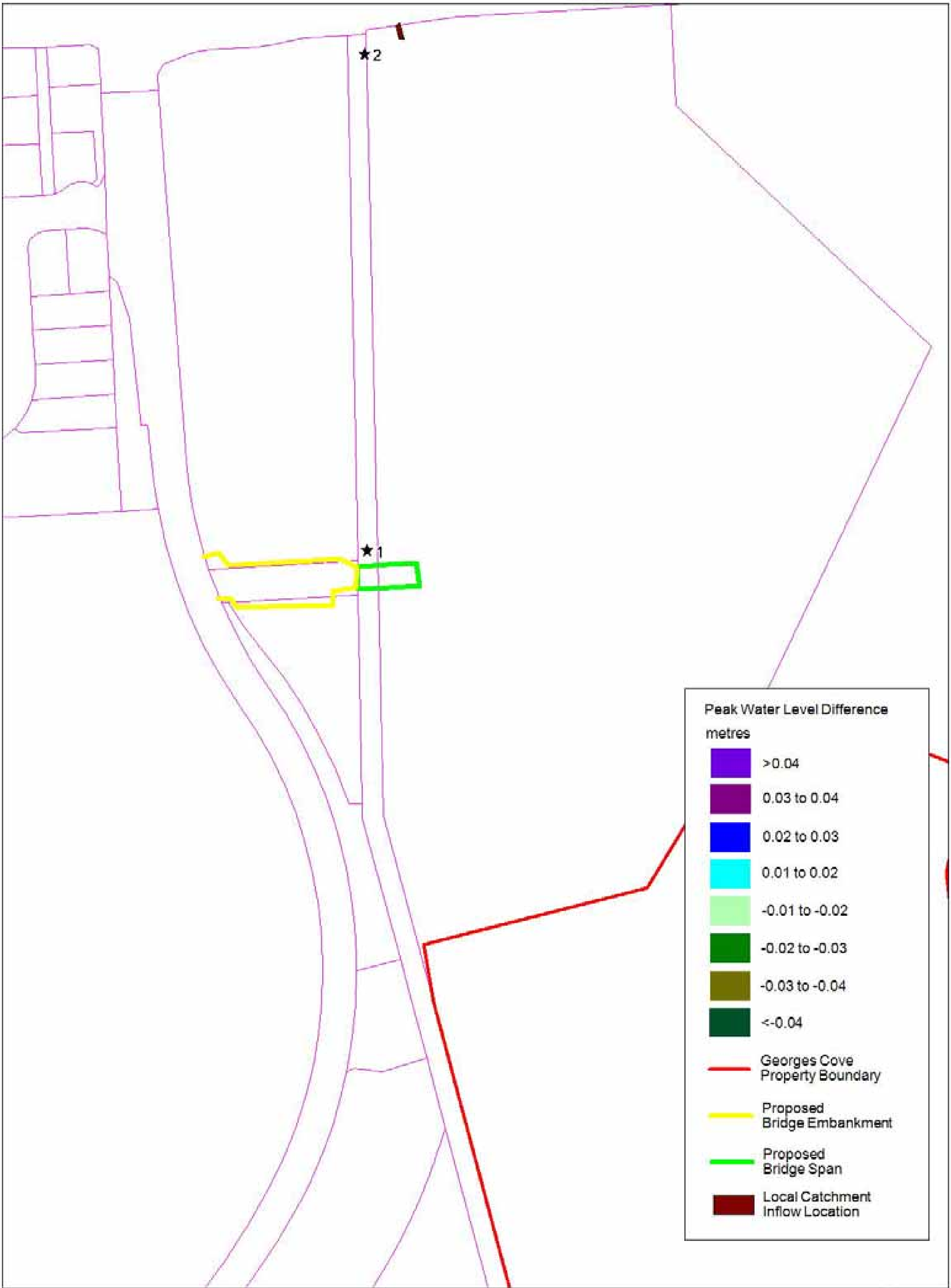
Georges Cove
Flood Impact Assessment

Figure 3
100 year ARI
Peak Velocity Differences
Post-Development - Road Bridge



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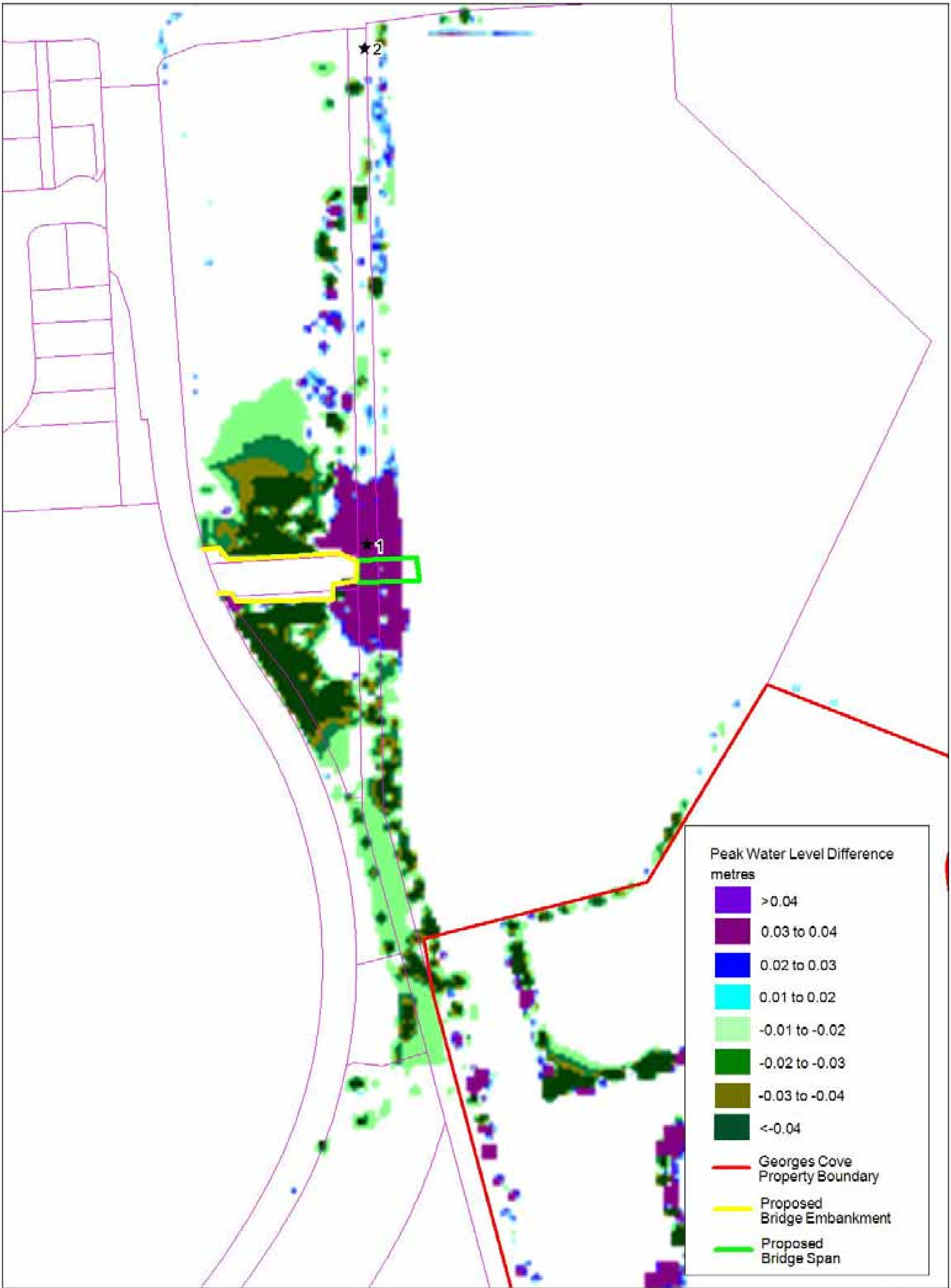
NA49913037
May 2014

Georges Cove
Flood Impact Assessment

Figure 4
20 year ARI Georges River Flooding
100 year ARI Local Flooding
Peak Water Level Differences
Post-Development - Road Bridge

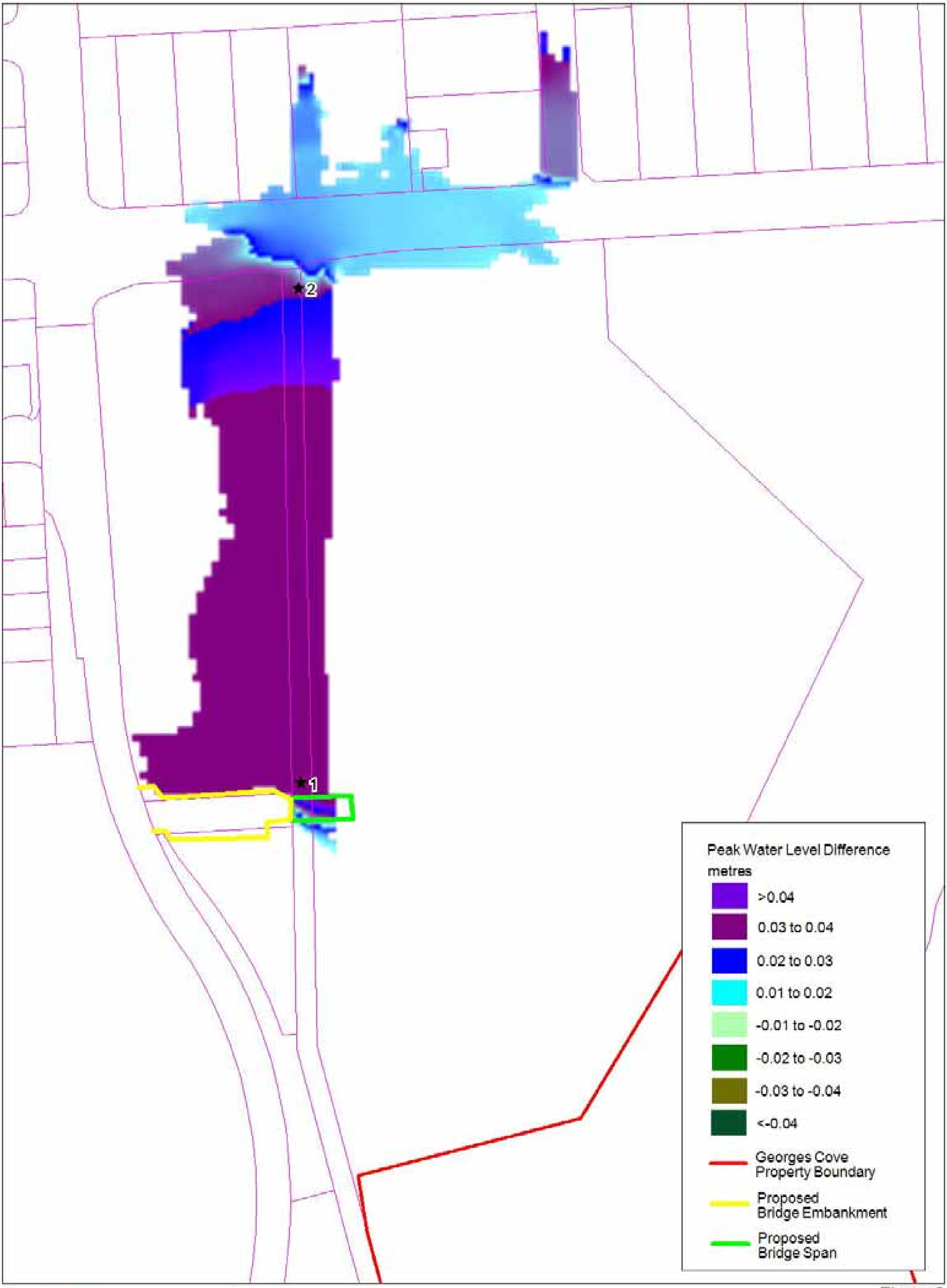


Cardno

NA49913037
May 2014

Georges Cove
Flood Impact Assessment

Figure 5
20 year ARI Georges River Flooding
100 year ARI Local Flooding
Peak Velocity Differences
Post-Development - Road Bridge

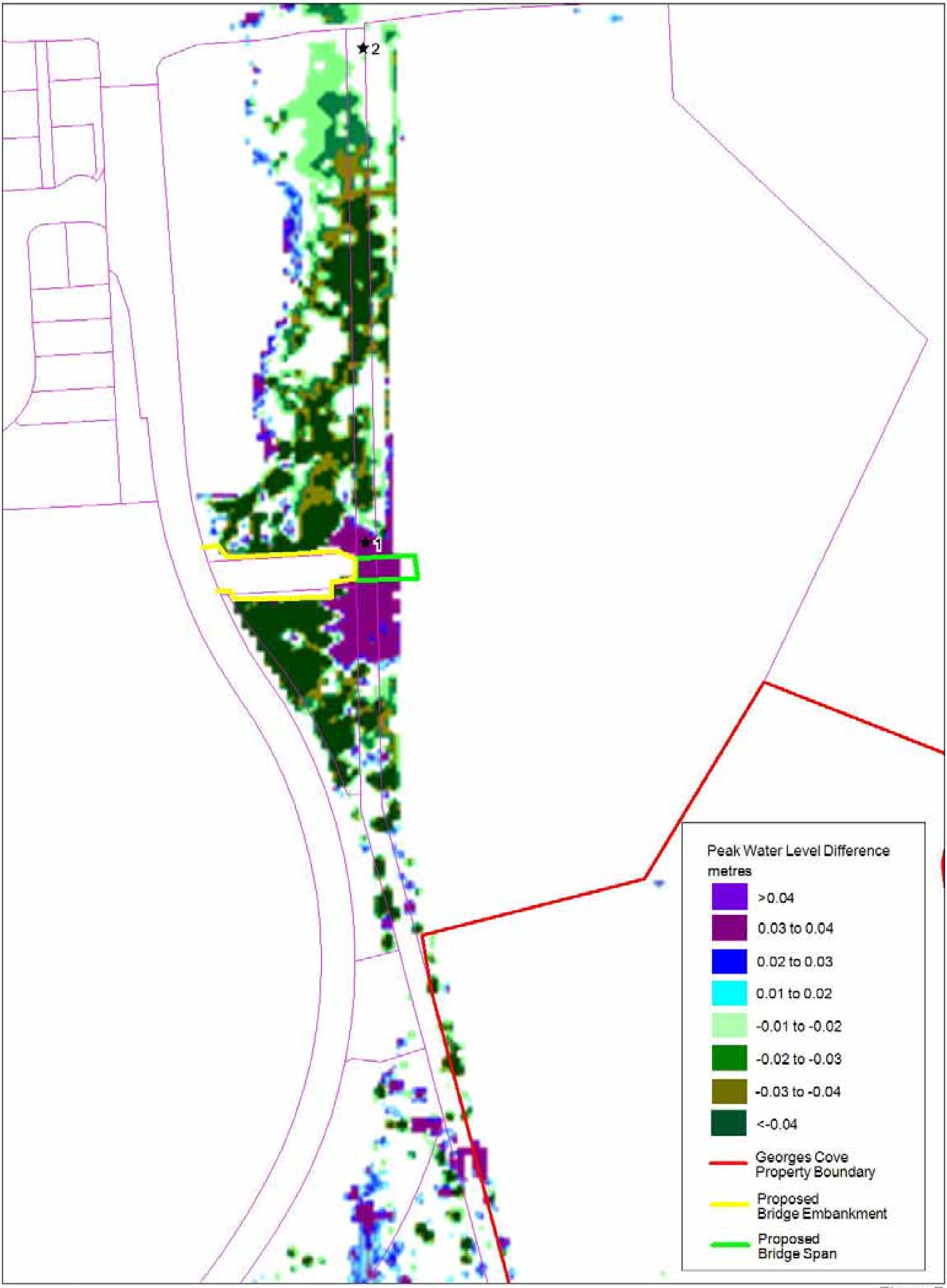


Cardno

NA49913037
May 2014

Georges Cove
Flood Impact Assessment

Figure 6
100 year ARI Local Flooding
Peak Water Level Differences
Post-Development - Road Bridge



Cardno

NA49913037
May 2014

Georges Cove
Flood Impact Assessment

Figure 7
100 year ARI Local Flooding
Peak Velocity Differences
Post-Development - Road Bridge

ATTACHMENT 2

Flood Risk Management Report (npc, November 2013)

Georges Cove Marina
Flood Risk Management

November 2013

1. Flood Levels

The predicted flood levels for the marina site are:-

20 yr ARI	– RL 4.6 to 4.7m AHD
100 yr ARI	– RL 5.6m AHD
Probable Maximum Flood (PMF)	– RL 10.2m AHD

The flood planning level (FPL) is RL 6.1m AHD (100 yr ARI + 0.5m freeboard).

2. Proposed Marina Development

The marina buildings have been designed to minimize their impact on flood behavior and the impact of flooding on the buildings. The main buildings housing the boat storage and amenities has the amenity facilities located above the PMF flood level at a minimum level of RL 10.525m AHD. The upper floor of the amenity facilities will be at RL 14.71m AHD. The first level of boat storage in this building will be at RL 7.3m AHD at least 1.7m above the 100 yr ARI flood level. The building below RL 7.3m AHD will be an open structure to allow flow through during floods. Parking under this main building will be at RL 4.6m AHD which is the 20 yr ARI flood level.

The small building will be the Private Marina Club house with a minimum floor level at the flood planning level of RL 6.1m AHD.

The southern carpark will be at a level of RL 1.65m AHD. This is equivalent to a 5 yr ARI flood level.

The marina development has been designed to ensure there is no loss of flood storage to minimize the impact on flood behavior.

3. Flood Behaviour

The proposed marina is located westwards of the main river flood flow paths and protected from these flows by the high lands immediately north and south of the site. The detailed 2D flood modeling of the proposed development by Cardno reaffirmed this behavior with low peak flow velocities in the 100 yr ARI flood of mainly 0 to 0.3 m/s with some isolated areas of higher velocity up to 0.5m/s. The floodway was located in the main river with velocities around 0.5 to 2 m/s (refer to Figure 3-18 in Cardno report).

The proposed marina and the area to the south west is a flood storage area which also plays an important part in the flood behavior. The proposed marina was designed to minimize any loss of flood storage.

The Cardno report demonstrated that the proposed marina would not have a significant adverse impact on flood levels and velocities. This is shown in the plot of water level and velocity differences between the post and before development scenarios presented in Figures 4-1 to 4-8 in the Cardno Report.

The Cardno flood study established that the preliminary hydraulic hazard in the marina would be rated as high in the 100 yr ARI flood.

4. Government Flood Risk Management Policy

The NSW Government's Flood Prone Land Policy and Floodplain Development Manual support the wise and rational development of flood prone land, the area inundated by the probable maximum flood (PMF). The policy acknowledges that flood prone land is a valuable resource that should not be sterilised by unnecessarily precluding its development and that development should be treated on its merits rather than through the application of rigid and prescriptive criteria.

The Manual specifies a process for appropriate risk management which requires Councils to under a flood study followed by a flood management study which should lead to the formulation of a floodplain management plan.

Liverpool City Council has prepared such a plan and integrated it into its Local Environmental Plan 2008 (Section 7.8) and the Liverpool Development Control Plan 2008 (Section 9 Flooding Risk).

The DCP specifies an industry best practice approach based on a matrix system which provides guidance on allowable development depending on flood risk category, land use risk category and planning controls. Table 4 for Georges River from Section 9 of the DCP applies to the proposed marina development. A copy of Table 4 is contained at Attachment A.

The flood risk category is high and the land use risk category is recreation and non-urban. Development of the marina is permissible subject to the controls listed in Table 4.

In terms of floor level, Table 4 requires non-habitable floor levels to be no lower than the 5 yr ARI level and habitable floors at the FPL. The proposed marina complies with these requirements (refer to Section 2).

For building components, Table 4 requires that all structures have flood compatible building components below the FPL. This can be complied with and is a proposed consent condition (Condition 27).

With structural soundness, Table 4 requires that an Engineer's report be provided to confirm the structure can withstand forces of floodwater, debris and buoyancy up to the FPL. The buildings will be open structures up to the FPL thereby limiting the force on the structures in a flood. The low velocities within the marina will also assist to alleviate the forces on the structures.

The building design would incorporate piles and columns capable of resisting the flood forces. The height of the building to the 100yr ARI flood level from the RL 4.5m and RL 1.65m ground levels would be 1m and 3.95m respectively. This is equivalent to just over a one store building. Also, the buildings will be open structures up to a minimum level of 0.5m above the 100yr ARI flood level.

For the PMF flood, the height to this flood level would be 5.6m and 8.55m respectively. This would represent a 2 to 3 storey building. A well designed building would be able to resist the hydraulic loads from a flood in the proposed conditions. A report from a certified engineer would be obtained to provide this evidence at the construction certificate stage. There is a proposed consent condition requiring this design and report from a certified engineer (Condition 28).

For flood effects, Table 4 requires conformance to three requirements. The first is to provide a report demonstrating no significant impacts on flood behaviour. The Cardno report demonstrates conformance to this requirement.

The second requirement deals with a floodway or major flood conveyance area and the likely adverse impact of structures in this area on flood behaviour. As noted in Section 3, the building structures are not located in a floodway and as such, this is why the Cardno flood study can demonstrate no significant adverse impact on flood behavior.

The third requirement deals with the need for a balanced cut and fill to avoid adverse impacts on flood behavior. This was an important consideration in the formulation of the marina proposal and involved considerable discussions with Council. The proposed marina complies with this requirement. There is a consent condition requiring the balanced cut and fill (Condition 29).

For car parking and driveway access, Table 4 requires conformance to four conditions. The first condition is that open car parking shall be as high as possible. The northern carpark is at the 20 yr ARI flood level at RL 4.6m AHD. The southern carpark is at the lowest level required in floor level conditions in order to balance the cut and fill. As such, this part of the parking conforms to the minimum requirement while being at the maximum level to conform to the balanced cut and fill requirement.

The second condition requires the driveway to be as high as practical and be generally rising. The driveway connects to the two parking areas which are as high as possible. It also generally rises from the southern carpark at RL 1.65m AHD to the northern carpark at RL 4.6m AHD and then to the proposed residential area to the north at RL 6.1m AHD. The driveway therefore conforms to this condition.

The third condition requires parking areas below the 20 yr ARI flood level shall have warning systems, signage and exits. The southern carpark would be fitted with these requirements as part of the Site Emergency Response Flood Plan to be formulated during the detailed design phase. The Site Emergency Response Flood Plan is discussed in Section 7. There is a condition proposed by Council to require a Flood Emergency Response Plan (Condition 115).

The fourth condition requires barriers to be provided in under building carparks to prevent floating vehicles from leaving the site. This condition will be complied with and this condition is proposed by Council (Condition 31).

For evacuation, Table 4 requires conformance with two conditions. The first condition requires that the development is consistent with any relevant flood evacuation strategy or similar plan. The existing strategy is to proceed to higher ground above the PMF level. The proposed evacuation strategy is outlined in Section 7.

The second condition requires that evacuation requirements be considered and an engineer's report would be required where evacuation might not be achieved within the effective warning time. The evacuation requirements have been considered in the provision of a rising route to land above the PMF level. This is discussed more fully in Section 7 along with the 12 hour warning time available which would be readily adequate to evacuate the proposed marina development.

For management and design, Table 4 requires conformance to three conditions. The first condition requires a safe Emergency Response Flood Plan where floor levels are below the design floor level. The Response Plan is outlined in Section 7 and Council has included a Condition requiring such a plan (Condition 115).

The second condition requires that there are areas above the PMF level to store goods. There are areas available in the main building which are above the PMF level which could be used to store goods during a severe flood.

The third condition requires that no materials are to be stored below the FPL which may cause pollution or be potentially hazardous during a flood. This can be readily achieved in the boat storage area which has a minimum storage level of RL 7.3m AHD. This is 1.2m above the FPL. There is a consent condition addressing this issue (Condition 144).

For fencing, Table 4 requires three conditions. These conditions relate to the fences not having an adverse impact on flood behavior by being permeable, allowed to collapse if necessary or not being unsafe during floods. This will be complied with and Council has incorporated a consent condition to address this issue (Condition 25).

The proposed marina development therefore complies with all the flood related requirements in the Liverpool DCP 2008 such that the flood risk management is appropriate and meets the requirements of NSW Government policy and legislation. Confirmation of this is required by the consent conditions throughout the project approval and development phases.

5. Liverpool Local Environmental Plan 2008

Section 7.8 (3) in the Liverpool LEP 2008 specifies the Flood Planning requirements for the proposed marina development. These requirements are addressed below.

5.1 Flood Behaviour and Adjacent Property

a) *will not adversely affect flood behaviour and increase the potential for flooding to detrimentally affect other development or properties*

The detailed flood impact assessment undertaken by Cardno established that the proposed development would not adversely impact flood behavior and would not adversely impact on the flood behaviour on adjacent properties (refer to Figures 4-1 to 4-8 in Cardno report).

5.2 Flow Distributions and Velocities

b) *will not significantly alter flow distributions and velocities to the detriment of other properties or the environment*

The proposed marina structures are located outside the main flood flow areas and is located in a flood storage area with low velocities. As such, there is no adverse impact on flood flow distributions and velocities (refer to Figures 3-8 and 3-18 in the Cardno report and in the responses in Section 5.1).

5.3 Safe Occupation and Evacuation

c) *will enable the safe occupation and evacuation of the land*

The proposed marina will have an approved safe emergency response flood plan as described in Section 7. It meets all the Government requirements for floor and car parking levels, rising evacuation routes, more than adequate warning times, dedicated and trained staff to manage the evacuation plan and a fallback option of vertical evacuation in the main building to levels significantly above the PMF level. The proposed development meets all the requirements of the NSW Government's Floodplain Development Manual and the Council's LEP and DCP for flooding. As such it is considered to enable safe occupation and evacuation.

5.4 Adverse Environmental Impacts

d) *Will not have a significant detrimental effect on the environment or cause avoidable erosion, siltation, destruction of riparian vegetation or a reduction in the stability of any riverbank or watercourse*

The proposed marina development will incorporate rock walls around the marina basin perimeter and on the outer walls along the river. This will stabilise the banks and prevent erosion. As the flood velocities are low, any erosion potential would be low.

As the development does not cause any significant change to the flow distribution and velocities as discussed in Sections 5.1 and 5.2, the development would not induce any new instability in the riverbank.

There will be a low rate of siltation in the marina basin due to sediment ladened flood flow. This is addressed in Section 3.10 of the Worley Parsons report supporting the application. The estimated rate of siltation in the marina basin is approximately 120mm over 100yrs. This will not cause any significant problems as a siltation allowance of 300mm has been incorporated into the selection of the design depth of the basin.

The existing riparian vegetation along the river foreshore at the marina site is limited and will be maintained.

5.5 Sustainable Flood Related Social and Economic Costs

e) will not be likely to result in unsustainable social and economic costs to the flood affected community or general community as a consequence of flooding

The proposed marina has been designed to minimize the potential flood related damages in terms of the building form, materials selection and adopted floor levels. Also, flood safety has been an important design principle. The proposed development is in accord with the NSW Government Floodplain Development Manual and thus, along with the above design approach, ensures that the development offers a sustainable approach to the social and economic costs of the local and general community. Importantly, it does not require significant additional flood related infrastructure or resources to support the proposed development.

5.6 Compatible with Flood Flow and Hazard

f) if located in the floodway, will be compatible with the flow of flood waters and with any flood hazard on that floodway

The development is not located within a floodway however it still is compatible with the flood flow and hazard. The buildings have been specifically located west of the main flood flows and designed to comply with its flood hazard and the associated requirements of Council's LEP and DCP as discussed in Section 4.

6. NSW Government Flood Related Legislation

The NSW Government's Floodplain Development Manual 2005 sets out the Government's Flood Prone Land Policy. Section 117 of the Environmental Planning and Assessment Act 1979 (EPA Act) allows the Minister for Planning to give directions to Councils regarding principles, aims, objectives or policies to be achieved in the preparation of draft local environmental plans (LEP).

The Minister released new directions on 1 July 2009 under Section 117(2) of the EPA Act. For directions related to Flood Prone Land, they were the same as Direction 15 issued on 31 January 2007.

The Directions for Flood Prone Land relate to the preparation of a Planning Proposal or draft LEP and as such are not relevant to the Georges Cove Marina DA.

Also, these Directions permit inconsistency with the Directions if the inconsistency was in accordance with the principles and guidelines of the Floodplain Development Manual 2005. Liverpool City Council has conformed with the Floodplain Development Manual 2005 in the undertaking and preparation of the Georges River flood study, floodplain management study and plan, the Liverpool LEP and DCP. In this way, the Section 117(2) Directions relating the Flood Prone Land are not applicable to the subject development as the development complies with the Liverpool LEP and DCP.

7. Site Emergency Response Flood Plan

The site emergency response flood plan would be formulated in detail as required in Council's consent conditions (Conditions 115-119). The approach and structure of this plan is discussed in the following sections.

The plan would be managed on site by the manager of the marina development. The leases for the onsite activities would identify the manager of the plan and provide the manager with the authority to order various activities under the plan such as training drills and evacuations.

Flooding in the Georges River has a 12 hour warning time issued by the Bureau of Meteorology for severe flooding. This warning can be issued electronically direct to the marina manager and other dedicated staff in the marina facility. In addition to this warning, there would be water level readers located at the waters edge which issue an electronic warning and sound an audible alarm when the river level reaches RL 1.3m AHD. The marina manager would then assess the flood risk and decide on the appropriate actions.

In considering the appropriate actions, the manager would review whether advice had been received from SES.

The first action would be to clear any cars parked in the southern carpark to areas offsite above the PMF level.

If the flooding was considered to be severe then the manager would instigate an orderly evacuation of the site. The evacuation would involve:-

- locking down the moored boats;
- storing any hazardous materials into designated areas above the FPL;
- requiring all persons to evacuate by the designated route and remove cars from the northern carpark.

The marina pontoons and pile supports would be designed to cater for flood levels, flood flows and debris imposed by the 100 yr ARI flood. A back up anchor pile and chain system would hold in place the marina pontoons. All craft could be readily tied to the chain system with quick lock fixtures when a severe flood warning was received.

The marina manager would act as the flood warden and he would have a number of designated assistant flood wardens. It would be the responsibility of the assistant flood wardens to ensure all people and cars in the facility have been evacuated.

The designated evacuation route would be east along the rising marina access road to the proposed bridge to Governor Macquarie Drive and up to Nuwarra Road. Nuwarra Road is above the PMF flood level and provides opportunities for refuge.

Flood warning signs would be provided in the carparks indicating that evacuation may be required and providing directions as to the evacuation route.

Each lease provided in the marina would include a flood management package alerting lessees of the potential flood risk, the evacuation plan and the need to follow the directions of the flood warden.

The flood warden would be responsible for providing flood training at the beginning of each new lease and organizing flood evacuation training for all employees on site at least once a year.

The flood risk management onsite is relatively straight forward as the people on site will be either employees or visitors to the site all under the control and management of the marina manager. There is also considerable flood warning time allowing for an orderly evacuation. Importantly, there is a fail safe back up evacuation plan which should not need to be used but if for some reason, a person does not evacuate the site in time, there is refuge available in the upper floors of the main building in areas above the PMF flood level.

8. JRPP Flood Related Issues

Liverpool City Council has conformed to the requirements of the NSW Government Flood Prone Land Policy and Floodplain Development Manual for the Georges River floodplain by undertaking a flood study, floodplain management study and floodplain management plan. Council has devised an appropriate means of achieving an acceptable flood risk management in the development of the floodplain in the formulation of a flood risk management matrix in its Liverpool DCP 2008 and the flood planning requirements in its LEP 2008.

The proposed marina conforms with the flood risk management guidelines thereby balancing the issues of risk management with the social and economic benefits of development. The specific aspects raised by the JRPP and the compliance of the marina development is summarised in the following sections.

8.1 Compliance with LEP

The compliance with Section 7.8 (3) of the Liverpool LEP 2008 is discussed in Section 5. This compliance is supported by the detailed flood impact assessment by Cardno and the discussion in Section 4 of the marina compliance with the flood risk management matrix in the Liverpool DCP 2008.

8.2 Compliance with NSW Legislation and Floodplain Development Manual

NSW Government policy requires a merit based approach to flood risk management based on a specified process of defining the flood behavior (flood study) and formulating a strategy for how best to deal with the flood risks (flood management plan, LEP and DCP requirements). This process has been followed by Council and the discussion in Sections 4, 5, 6 and 7 demonstrates how the proposed marina conforms to the requirements.

8.3 Building Adequacy

The proposed buildings are not located within the floodway. The Cardno detailed flood study demonstrates that the flow velocities are low in the area of the proposed buildings and the buildings are located in a flood storage area. Nonertheless, the building has been designed as an open structure below the FPL to minimize the flood loads. The low flow velocities will assist to minimize the flood forces on the building.

The structural adequacy issue is discussed in Section 4 and would be verified by a report from a certified engineer.

8.4 Site Evacuation

A site emergency response flood plan is discussed in Section 7 and outlines how this plan would provide a “trigger point” for critical flood events and an evacuation strategy for practical and safe passage of vehicles and patrons from the site.

ATTACHMENT A

Table 4

Section 9 Liverpool DCP 2008

Flooding Risk

Table 4 Georges River Floodplain (Includes Harris Ck and Williams Ck, lower parts of Anzac Ck, but not Cabramatta Creek)

Flood Risk Category	Land Use Risk Category	Planning Controls							
		Floor Level	Building Components	Structural Soundness	Flood Effects	Car Parking & Driveway Access	Evacuation	Management & Design	Fencing
Low Flood Risk	Critical Uses & Facilities								
	Sensitive Uses & Facilities	13	4	4	2, 4, 5	2, 3, 6, 7, 8	6, 8, 9	2, 4	
	Subdivision				2, 4, 5			1	
	Residential (++)	2, 6	2	3	2, 4, 5	2, 3, 6, 7, 8	6, 9		
	Commercial & Industrial	4, 8, 15	2	3	2, 4, 5	2, 3, 6, 7, 8	(4 or 9), 6	2, 3, 5	
	Tourist Related Development	2, 6, 15	2	3	2, 4, 5	2, 3, 6, 7, 8	6, 9	2, 3, 5	
	Recreation & Non-Urban	2, 7	2	3	2, 4, 5	1, 5, 7, 8	6, 8	2, 3, 5	
	Concessional Development	14, 15	2	3	2, 4, 5	1, 7, 8, 9	6, 9	2, 3, 5	
Medium Flood Risk	Critical Uses & Facilities								
	Sensitive Uses & Facilities								
	Subdivision				1, 4, 5			1	1, 2, 3
	Residential	2, 6, 15	2	2	2, 4, 5	2, 3, 6, 7, 8	6, 9		1, 2, 3
	Commercial & Industrial	8, 4, 15	2	2	2, 4, 5	2, 3, 6, 7, 8	4, 6	2, 3, 5	1, 2, 3
	Tourist Related Development	2, 6, 15	2	2	2, 4, 5	2, 3, 6, 7, 8	6, 9	2, 3, 5	1, 2, 3
	Recreation & Non-Urban	2, 7	2	2	2, 4, 5	1, 5, 7, 8	6, 8	2, 3, 5	1, 2, 3
	Concessional Development	14, 15	2	2	2, 4, 5	1, 7, 8, 9	8, 9	2, 3, 5	1, 2, 3
High Flood Risk	Critical Uses & Facilities								
	Sensitive Uses & Facilities								
	Subdivision								
	Residential								
	Commercial & Industrial								
	Tourist Related Development								
	Recreation & Non-Urban	2, 7	2	2	1, 4, 5	1, 5, 7, 8	6, 8	2, 3, 5	1, 2, 3
	Concessional Development	14, 15	2	2	1, 4, 5	1, 7, 8, 9	6, 9	2, 3, 5	1, 2, 3

Key:

(teal)	Not Relevant
(pink)	Unsuitable Land Use
1, 2, 3	Control reference number relevant to the particular planning consideration. (see Table 6)
(++)	Attached dwellings, Dwelling houses, dual occupancies, multi unit dwelling housing, residential flat buildings (not including development for the purpose of group homes or seniors housing), Secondary dwellings and Semi-detached dwellings are exempt from these controls.

Table 5 Local Overland Flooding

Flood Risk Category	Land Use Risk Category	Planning Controls							
		Floor Level	Building Components	Structural Soundness	Flood Effects	Car Parking & Driveway Access	Evacuation	Management & Design	Fencing
Local Overland Flood Risk	Critical Uses & Facilities	13	4	5	3	4, 7, 8	7	3, 5	2, 4
	Sensitive Uses & Facilities	13	4	5	3	4, 7, 8	7	3, 5	2, 4
	Subdivision				3		5	1	2, 4
	Residential	3, 5	1	6	3	4, 7, 8	5		2, 4
	Commercial & Industrial	10	1	6	3	4, 7, 8	5	3, 5	2, 4
	Tourist Related Development	3, 5	1	6	3	4, 7, 8	5	3, 5	2, 4
	Recreation & Non-Urban	3, 5	1	6	3	4, 7, 8	5	3, 5	2, 4
	Concessional Development	14	1	6	3	4, 7, 8	5	3, 5	2, 4

Key:

(shaded)	Not Relevant
1, 2, 3	Control reference number relevant to the particular planning consideration.

Table 6 Explanation of Development Controls

Ref No	Controls
Floor level	
1	All floor levels to be as high as practical but not less that the 20% AEP flood level.
2	Non habitable floor levels to be as high as practical but no less than the 5% AEP flood level.
3	Non-habitable floor levels to be not less than the 1% AEP flood.
4	The level of Non-habitable and general Industrial floor areas to be as high as practical but not less that the 2% AEP flood. Where this is impractical for single lot developments within an existing developed area, the floor shall be as high as practical but no less than the 5% AEP flood.
5	Habitable floor levels to be equal to or greater than the 1% AEP flood level plus 300mm freeboard.
6	Habitable floor levels to be equal to or greater than the 1% AEP flood level plus 500mm freeboard.
7	Habitable floor levels to be no lower than the 1% AEP flood plus 500mm freeboard unless justified by site specific assessment.
8	Habitable and general commercial floor levels to be as high as practical but no lower than the 1% AEP flood plus 500mm freeboard unless justified by site specific assessment.
9	The level of habitable floor areas to be equal to or greater than the 1% AEP flood level plus 500mm freeboard. If this level is impractical a lower floor level may be considered provided the floor level is as high as possible but no less than the 5% AEP flood level.
10	All floor levels to be equal to or greater than the 1% AEP flood level plus 300mm freeboard. Freeboard may be reduced if justified by site specific assessment.
11	All floor levels to be no lower than the 1% AEP flood plus 500mm freeboard. Freeboard may be reduced if justified by site specific assessment.
12	All floor levels to be equal to or greater than the PMF level. If this level is impractical a lower floor level may be considered provided the floor level is as high as possible but no less than the 1% AEP flood level plus 500mm freeboard.

Ref No	Controls
13	Floor levels to be no lower than the PMF level unless justified by a site specific assessment.
14	Floor levels to be equal to or greater than the minimum requirements normally applicable to this type of development. Where this is not practical due to compatibility with the height of adjacent buildings, or compatibility with the floor level of existing buildings, or the need for access for persons with disabilities, a lower floor level may be considered. In these circumstances, the floor level is to be as high as practical, and, when undertaking alterations or additions no lower than the existing floor level.
15	A restriction is to be placed on the title of the land, pursuant to S.88B of the *Conveyancing Act*, where the lowest habitable floor area is elevated more than 1.5m above finished ground level, confirming that the undercroft area is not to be enclosed.
Building Components & Method	
1	All structures to have flood compatible building components below the 1% AEP flood level plus 300mm freeboard.
2	All structures to have flood compatible building components below the 1% AEP flood level plus 500mm freeboard.
3	All structures to have flood compatible building components below the 1% AEP flood level plus 500mm freeboard or a PMF if required to satisfy evacuation criteria (see below).
4	All structures to have flood compatible building components below the PMF level.
Structural Soundness	
1	Applicant to demonstrate that the structure can withstand the forces of floodwater, debris and buoyancy up to and including a 1% AEP flood plus 500mm freeboard or a PMF if required to satisfy evacuation criteria (see below). An engineers report may be required.
2	Engineer's report to certify that the structure can withstand the forces of floodwater, debris and buoyancy up to and including a 1% AEP flood plus 500mm freeboard.
3	Applicant to demonstrate that the structure can withstand the forces of floodwater, debris and buoyancy up to and including a 1% AEP flood plus 500mm freeboard.
4	Applicant to demonstrate that any structure can withstand the forces of floodwater, debris and buoyancy up to and including a PMF. An engineers report may be required.
5	Applicant to demonstrate that any structure can withstand the forces of floodwater, debris and buoyancy up to and including a PMF.
6	Applicant to demonstrate that the structure can withstand the forces of floodwater, debris and buoyancy up to and including a 1% AEP flood plus 300mm freeboard.
Flood Effects	
1	Engineers report required to certify that the development will not increase flood effects elsewhere, having regard to: (I) loss of flood storage; (ii) changes in flood levels, flows and velocities caused by alterations to flood flows; and (iii) the cumulative impact of multiple similar developments in the floodplain.
2	The flood impact of the development to be considered to ensure that the development will not increase flood effects elsewhere, having regard to: (i) loss of flood storage; (ii) changes in flood levels and velocities caused by alterations to the flood conveyance; and (iii) the cumulative impact of multiple potential developments in the floodplain. An engineer's report may be required.
3	The flood impact of the development to be considered to ensure that the development will not increase flood affectation elsewhere having regard to changes in flood levels and velocities caused by

Ref No	Controls
	alteration of conveyance of flood waters. An engineers report may be required if Council considers a significant affectation is likely. The unmitigated obstruction, concentration or diversion of overland flow paths to adjacent property shall not be permitted.
4	A floodway or boundary of significant flow may have been identified in this catchment. This area is the major conveyance area for floodwaters through the floodplain and any structures placed within it are likely to have a significant impact on flood behaviour. Within this area no structures other than concessional development, open type structures or small non habitable structures (not more than 30sqm) to support agricultural uses will normally be permitted. Development outside the Boundary of Significant flow may still increase flood effects elsewhere and therefore be unacceptable
5	Any filling within the 1% AEP flood will normally be considered unacceptable unless compensatory excavation is provided to ensure that there is no net loss of floodplain storage volume below the 1% AEP flood.
Car Parking and Driveway Access	
1	The minimum surface level of open car parking spaces, carports or garages, shall be as high as practical.
2	The minimum surface level of a car parking space, which is not enclosed (e.g. open car parking space or carport) shall be as high as practical, but no lower than the 5% AEP flood level or the level of the crest of the road at the highest point were the site can be accessed. In the case of garages, the minimum surface level shall be as high as practical, but no lower than the 5% AEP flood.
3	Garages capable of accommodating more than 3 vehicles on land zoned for urban purposes, or basement car parking, must be protected from inundation by floods equal to or greater than the 1% AEP flood plus 0.1m freeboard.
4	Basement car parking shall be protected from inundation by the 1% AEP flood.
5	The driveway providing access between the road and car parking space shall be as high as practical and generally rising in the egress direction.
6	The level of the driveway providing access between the road and car parking space shall be no lower than 0.3mbelow the 1% AEP flood or such that depth of inundation during a 1% AEP flood is not greater than either the depth at the road or the depth at the car parking space. A lesser standard may be accepted for single detached dwelling houses where it can be demonstrated that risk to human life would not be compromised.
7	Basement car parking or car parking areas accommodating more than 3 vehicles (other than on Rural zoned land) with a floor level below the 5% AEP flood or more than 0.8m below the 1% AEP flood level; shall have adequate warning systems, signage and exits.
8	Barriers to be provided to prevent floating vehicles leaving a site during a 1% AEP flood.
9	Driveway and car parking space levels shall be no lower than the minimum requirements normally applicable to this type of development. Where this is not practical, a lower level may be considered. In these circumstances, the level is to be as high as practical and, when undertaking alterations or additions no lower than the existing level.
Evacuation	
1	Reliable access for pedestrians required during a 1% AEP flood.
2	Reliable access for pedestrians or vehicles is required from the building, commencing at a minimum level equal to the lowest habitable floor level to an area of refuge above the PMF level, or a minimum of 20% of the habitable floor area is above the PMF.
3	Reliable access for pedestrians or vehicles is required from the building to an area of refuge above the PMF level, or a minimum of 20% of the habitable floor area is above the PMF

Ref No	Controls
4	Reliable access for pedestrians or vehicles required during a 1% AEP flood to a publicly accessible location above the PMF.
5	The evacuation requirements of the development during flooding shall be considered.
6	The development is to be consistent with any relevant flood evacuation strategy or similar plan.
7	The evacuation requirements of the development are to be considered up to the PMF level.
8	The evacuation requirements of the development are to be considered. An engineers report will be required if circumstances are possible where the evacuation of persons might not be achieved within the effective warning time.
9	Adequate flood warning is available to allow safe and orderly evacuation without increased reliance upon the SES or other authorised emergency services personnel.
Management and Design	
1	Applicant to demonstrate that potential development as a consequence of a subdivision proposal can be undertaken in accordance with this DCP.
2	Site Emergency Response Flood Plan required where floor levels are below the design floor level, (except for single dwelling-houses).
3	Applicant to demonstrate that area is available to store goods above the 1% AEP flood level plus 500mmfreeboard.
4	Applicant to demonstrate that area is available to store goods above the PMF level.
5	No storage of materials below the design floor level which may cause pollution or be potentially hazardous during any flood.
6	Finished land levels in new release areas shall be not less than the 1% AEP flood unless justified by site specific assessment. A surveyor's certificate will be required upon completion certifying that the final levels are not less that the required level.
Fencing	
1	Fencing within a High Flood Risk area, Boundary of Significant Flow or floodway will not be permitted except for permeable open type fences.
2	Fencing is to be constructed in a manner that does not obstruct the flow of floodwaters so as to have an adverse impact on flooding.
3	Fencing shall be constructed to withstand the forces of floodwaters or collapse in a controlled manner so as not to obstruct the flow of water, become unsafe during times of flood or become moving debris.
4	Fencing shall be constructed to withstand the forces of floodwaters.

ATTACHMENT 3

Flood Impact Assessment for the Proposed Georges Cove Marina, Moorebank (Cardno, January 2013)

Our Ref: NA49913037-L02:BCP/bcp
Contact: Dr Brett C. Phillips

29th January 2013

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The Manager
npc
PO Box 1060
CROWS NEST NSW 1585

Attention: Mr Mark Tooker

Dear Mark,

FLOOD IMPACT ASSESSMENT FOR THE PROPOSED GEORGES COVE MARINA, MOOREBANK

Cardno was commissioned by npc to conduct a Flood Impact Assessment for the proposed Georges Cove Marina development in Moorebank, within the Liverpool City Council Local Government Area (LGA).

The objective of the study was to determine the flooding behaviour for the proposed Georges Cove Marina site during major flood events and to assess the impact if any of proposed development on flooding of adjacent properties.

At the request of Liverpool City Council flooding in the 20 year and 100 year Average Recurrence Interval (ARI) events has been assessed. Council was particularly concerned about the proposed marina impacting on flows during the 20 yr ARI flood into the flood storage area immediately southwest of the site and the potential increase in flood levels on adjacent properties.

1. BACKGROUND

1.1 Study Area

The Georges Cove Marina site is located on the western bank of the Georges River between Newbridge Road and the M5 motorway in the suburb of Moorebank, in south-west Sydney. The site location is shown in **Figure 1-1**.

The site covers an area of approximately 12.3 ha and is part of the Benedict Sand & Gravel (BS&G) landholding. Sand extraction activities are undertaken on the site. There are large expanses of undeveloped vegetated floodplain south of the site.

1.2 Flooding in the Georges River at Moorebank

The 2004 Georges River Floodplain Risk Management Study by Bewsher Consulting modelled the flooding behaviour for the Georges River from the upper Georges River upstream of Liverpool to Botany Bay downstream. The Study used the 1D hydraulic modelling program MIKE-11 as the basis for its flood analysis.

The following overview has been extracted from the Georges River Floodplain Risk Management Study (Bewsher Consulting, 2004) detailing various flood studies that had occurred prior to 2004:

> *Design flood levels on the Georges River are available from the Georges River Flood Study [PWD, 1991]. This study used a physical scale model of the Georges River to simulate flood conditions between Picnic Point and Liverpool.*
>
> *A number of other studies have also been undertaken to define flood conditions upstream of Liverpool and for the main tributary creeks of the Georges River. These studies include:*
>
> - *Upper Georges River Flood Study [DLWC, 1999];*
> - *Draft Cabramatta Creek Floodplain Management Study [Bewsher Consulting,1999];*
> - *Lower Prospect Creek Floodplain Management Study [Willing & Partners, 1990];*
> - *Milperra Industrial Area Hydraulic Study [Willing & Partners, 1990];*
> - *Little Salt Pan Creek Flood Study [Manly Hydraulics Laboratory, 1995];*
> - *Salt Pan Creek Flood Study [PWD, 1991];*
> - *Deadmans Creek Flood Study [DLWC, 1997].*
>
> *A single computer model of the Georges River study area was developed by Bewsher Consulting for Liverpool Council. This model has been used as part of further flood investigations for the 2004 floodplain management study.*
>
> *The Georges River MIKE-11 model was developed from various sources. The origin of the model was a MIKE-11 in-bank tidal model, which was first developed by the Public Works Department to study tidal behaviour between Liverpool and Botany Bay [PWD, 1992]. The tidal model was subsequently extended by Bewsher Consulting to incorporate the floodplain, by extending model cross sections and inserting additional overbank flow paths. A separate MIKE-11 model, developed as part of the Upper Georges River Flood Study [DLWC, 1998], was also added to the main model to extend it upstream of Liverpool (Bewsher, 2004)*

Council requested that a flood impact assessment be undertaken of the proposed development using a 2D floodplain model of the site and adjoin lands upstream and downstream of the site. Council provided a copy of the MIKE-11 model to assist in the assembly of the 2D floodplain model.

1.3 Proposed Development

The proposed development has been described in the following terms.

The proposed development of a marina is located on land zoned private and public recreation located on the western bank of the Georges River and is around 600 m due south of Newbridge Road.

The site is presently a sand extraction area located immediately south of a proposed residential area (ground level above the 100yr ARI), immediately east of the residential development in the former Boral quarry site and immediately north of a large elevated former landfill site (levels near and above the 100yr ARI). The Flower Power site also immediately to the north has been recently approved for development by Council. The residential area immediately to the north includes a high level bridge connection to the former Boral quarry site residential development which provides access to high ground above the PMF level. The marina development will have access to this bridge.

The flood levels at the site estimated from the 2004 Georges River Floodplain Risk Management Study are as follows:

20 yr ARI	RL 4.6 m AHD
100 yr ARI	RL 5.6m AHD
PMF	RL 10.2m AHD

The flood planning level is RL 6.1m AHD (100 yr ARI plus 0.5m).

The proposed marina development consists of floating berths, a dry storage facility incorporating amenities at the upper levels and a private marina club house. Importantly the buildings are located on the western edge of the site and are open structures below the flood planning level. The formation of the marina basin creates additional flood storage.

The lowest level of boat storage is above the FPL at RL 7.3 m AHD. The lowest amenities area in the boat storage building is at a level above the PMF level at RL 10.525 m AHD. The private marina club level is at the FPL of RL 6.1 m AHD. It is stated that these buildings provide minimum disruption to any flood conveyance because:

1. They are open structures below the FPL;
2. They do not extend eastwards of the major flood flow controllers either upstream or downstream of the site which are the residential area (and Flower Power) to the north and the former landfill site to the south.

A key issue is that stated presence of high ground immediately north of the proposed development (higher than the 100 yr ARI flood level) and a large elevated former landfill site (levels near and above the 100yr ARI) located immediately south of the proposed development. If this is the case then it would be expected that flooding of the proposed development in events up to the 100 yr ARI event would be primarily by the lateral discharge of floodwaters from the river into the site rather than longitudinal flood flows spilling into the site across its northern boundary and discharging through the southern boundary of the site. The lateral interchange of flow with the river would be then primarily controlled by the ground levels on the bank of the river (eastern side of the site) and the waterway opening through the river bank.

It is expected that in a PMF the site would be subject initially to lateral flows which would then become longitudinal flood flows as the flood level continues to rise above the 100 yr ARI flood level.

The development site is located between high ground upstream and downstream and it is expected that in events less than a 100 yr ARI event that the proposed development would have only local impacts on design flood levels and it is likely that these impacts would be minimal and would not have any significant impacts upstream of the site.

The proposed development has been detailed on an architectural plan view, included as **Figure 1-2**, and section views of the proposed site included as **Figure 1-3**, prepared by Michael Fountain Architects on 11th November 2010.

2. HYDRAULICS

Hydrological modelling was not required to support this assessment (with the exception of the Newbridge culverts, refer to **Section 2.2.1**) due to the availability of routed hydrographs that were obtained from Council's existing MIKE-11 models which were prepared as part of the 2004 Georges River Flood Risk Management Study.

In order to assess the spatial impact of the proposed development on flooding it was proposed to assemble a local 2D floodplain model of the development and a reach upstream and downstream of the site. The hydraulic study area is identified in **Figure 2-1**.

2.1 2D Floodplain Model

The TUFLOW 2D hydraulic model covers an area of about 744 ha. The model extends from upstream of Newbridge Road in the north to just downstream of the M5 Motorway to the south.

Digital Elevation Model

The Digital Elevation Model (DEM), which assigns elevation values to the 2D grid cells, has been mostly based on Aerial Laser Survey (ALS) data provided by Liverpool City Council on the 14th September 2012.

Generally, the accuracy of the ALS data is +/- 0.15m for vertical elevations on hard surfaces, but provides less reliable records in densely vegetated areas. ALS is unable to penetrate surface of water bodies and therefore provides no data on the bed geometry of the Georges River. Hence the Georges River bed geometry was represented in the DEM based on the 1D cross sections extracted from Council's existing MIKE-11 model. The bed and bank geometry were interpolated between cross sections to create a 2D DEM of the river bed and banks.

The base DEM is shown in **Figure 2-2**.

For assessment purposes a 2m x 2 m grid size was adopted in the TUFLOW 2D floodplain model.

Surface Roughness

Hydraulic surface roughness have been modelled in the 2D TUFLOW model using spatially distributed Manning roughness ("n") values based on aerial photography (NearMap, recorded 23/10/11). Areas of the floodplain were assigned a land-use category, with an associated roughness value as shown in **Table 2-1**.

The land-use breakdown for the study area is shown in **Figure 2-3**.

Boundary Conditions

Inflows to the local 2D floodplain model were based on routed hydrographs that were obtained from Council's existing MIKE-11 models which were prepared as part of the 2004 Georges River Flood Risk Management Study.

Table 2-1 Roughness Values for Different Landuse Categories

Land-Use Type	Manning Roughness Value
Water Body	0.040 – 0.045
Open Space	0.04
Floodplain	0.09
Environmental / Special Use	0.09
Road	0.02
Residential	0.12
Commercial	0.20
Industrial	0.20
Rockwall	0.10

There were three main inflows considered in this study:

- *Georges River:* The critical duration storm for the Georges River catchment is the 36 hour storm for both the 20 year and 100 year ARI events and is estimated to generate peak flows of approximately 1,570 m^3/s and 2,080 m^3/s respectively in the vicinity opposite Bankstown Airport (refer MIKE-11 Cross Section Chainage 10121 m).

- *Milperra Drain:* The Milperra Drain collects runoff from the suburbs of Georges Hall and Milperra and from Bankstown Airport. It flows in a westerly direction along the northern boundary of Bankstown Golf Course before turning south and flowing along the western boundary of the Bankstown Golf Course before then turning west and flowing under Henry Lawson Drive and eventually discharging into the Georges River. The peak flow for the 20 year and 100 year ARI 36 hour duration storms was calculated to be approximately 54 m^3/s and 66 m^3/s respectively (refer MIKE-11 Cross Section MD8470 m). The inflows from Milperra Drain were input in the 2D floodplain model in the vicinity of the northwestern corner of the Bankstown Golf Course.

- *Newbridge Road Drain:* There is a major stormwater outfall located on the southern side of Newbridge Road to the west of the BS&G site. The outfall comprises 4 x 0.65 m (H) x 1.75 m (W) box culverts. This is the outfall of a drainage network with unknown extents in the suburb of Chipping Norton to the north of the site. The potential peak discharge of the culverts in a 36 hour storm was estimated by assembling a broadscale DRAINS model for the estimated local catchment area. The peak discharges at the outfall in the 20 year and 100 year ARI 36 hour storms were estimated to be approximately 4 m^3/s and 5 m^3/s respectively. These minor inflows were considered negligible in comparison with the flows in the Georges River and Milperra Drain and therefore the discharge from the Newbridge Road outfall was not included in the TUFLOW hydraulic model.

The hydrograph inflow locations are shown in **Figure 2-2**.

The adopted downstream boundary conditions were the 20 and 100 year ARI water level time series obtained from Council's existing MIKE-11 model at a cross section located immediately downstream of the M5 Motorway (Chainage 14180 m).

2.1 Model Calibration

The calibration of the TUFLOW floodplain model was conducted iteratively by comparing the 100 yr ARI and 20 yr ARI flood levels predicted by the TUFLOW and 2004 MIKE-11 models by progressively adjusting the roughness values within acceptable ranges to achieve an acceptable fit to the design flood levels estimated in 2004.

A comparison of the design flood levels predicted by the 2004 MIKE-11 model and TUFLOW floodplain model is given in **Table 2-2**. Approximate locations of the MIKE-11 cross section locations are shown in **Figure 2-2**. Cross sections from chainages 11960 m to 12890 m are within the vicinity of the Georges Cove site with cross sections chainages 12330 m, 12500 m, and 12620 m are adjacent to the Georges Cove site.

It is concluded that the TUFLOW base model gives an acceptable representation of the MIKE-11 design flood with the level of agreement for the predicted 100 yr ARI flood levels being between – 0.02 m to +0.15 m and for the 20 yr ARI flood levels being between – 0.02 m to +0.15 m. In the vicinity of the proposed development the level of agreement for the predicted 100 yr ARI flood levels being between – 0.02 m to +0.02 m and for the 20 yr ARI flood levels being between – 0.01 m to +0.03 m.

Table 2-2 Comparison of Predicted Design Flood Levels (m AHD)

Cross Section Chainage (m)	MIKE-11 Peak WL Results (m AHD)		TUFLOW Peak WL Results (m AHD)		WL Difference (m) (TUFLOW Less MIKE-11)	
	20 year ARI	100 year ARI	20 year ARI	100 year ARI	20 year ARI	100 year ARI
10120	5.15	5.94	5.18	6.01	0.03	0.06
10290	5.05	5.85	5.11	5.94	0.06	0.09
10410	5.01	5.81	5.08	5.91	0.07	0.10
10590	4.96	5.78	5.05	5.89	0.09	0.11
10740	4.91	5.74	5.02	5.87	0.11	0.13
10890	4.86	5.70	4.99	5.85	0.13	0.15
10930	4.84	5.68	4.97	5.83	0.13	0.15
10970	4.78	5.65	4.93	5.80	0.15	0.15
11050	4.78	5.66	4.93	5.79	0.15	0.13
11140	4.79	5.67	4.94	5.80	0.15	0.14
11350	4.76	5.65	4.83	5.73	0.07	0.08
11650	4.76	5.65	4.79	5.69	0.04	0.05
11780	4.75	5.65	4.77	5.66	0.02	0.02
11960	4.74	5.64	4.74	5.63	-0.01	-0.01
12140	4.72	5.63	4.71	5.61	-0.01	-0.02
12330	4.70	5.60	4.70	5.59	0.00	-0.01
12500	4.64	5.54	4.67	5.57	0.03	0.02
12620	4.65	5.56	4.64	5.54	-0.01	-0.02
12890	4.59	5.50	4.58	5.49	-0.01	-0.01
13030	4.56	5.47	4.56	5.46	0.00	-0.01
13200	4.54	5.44	4.53	5.44	0.00	-0.01
13520	4.48	5.37	4.49	5.39	0.02	0.02
13820	4.39	5.29	4.39	5.30	-0.01	0.01
13960	4.35	5.25	4.33	5.24	-0.02	-0.01
14150	4.28	5.19	4.29	5.19	0.00	0.00
14180	4.28	5.18	4.28	5.18	0.00	0.00

The main differences between the predicted flood levels are attributed to the differences in floodplain storage associated with a 1D representation of the floodplain in comparison with a 2D representation of the floodplain at a far finer scale than the 1D model. It is expected that the 2D model provides a better definition of the floodplain topography and floodplain storage.

2.6 Benchmark (Pre-Development) Scenario

The calibrated benchmark pre-development model was adjusted to represent approved levels on a number of sites located within the study area as follows:

- There are a number of current developments near the Georges Cove site which have the potential to impact on the hydraulics of the study area. Liverpool City Council advised npc that the future finished levels of the Flower Power site, and northern portion of the BS&G site would be 6.3 m AHD. These two sites were raised to these levels in the benchmark scenario.

- There is also a residential development proposed to the west of the Georges Cove site which was flood affected in the initial TUFLOW model (refer to **Figure 2-4**). It was assumed that the landform would be modified such that there will be is no inundation on the development in either the 20 year or the 100 year ARI events.

- Following discussion with Liverpool City Council it was agreed that the levels of the Georges Cove site in the benchmark model should represent the likely rehabilitated site topography following works to remove the existing lakes and material stockpiles from the site.

  The pre-development site includes an environmental protection zone on the eastern side of the site incorporated within a 40 m riparian zone along the bank of the Georges River. The pre-development level of this area is around 1.9 m AHD and was assumed to be represented by a hydraulic roughness of 0.04. The remainder of the site is subject to a rehabilitation consent condition that requires finished levels of between 1.6 m AHD – 1.7m AHD. This area was treated as cleared land with a hydraulic roughness of 0.04.

The changes to the pre-development study area are summarised in **Figure 2-4**, the pre-development DEM is shown in **Figure 2-5**, and the pre-development land-use / roughness map is shown in **Figure 2-6**.

2.7 Post-Development Scenario

The benchmark floodplain model was subsequently modified to represent planned development site as detailed on an architectural plan view, included as **Figure 1-2**, and section views of the proposed site included as **Figure 1-3**, prepared by Michael Fountain Architects on 11th November 2010.

The main features of the proposed development are as follows:

- A marina basin located in the middle of the site with an assumed an invert level of -3.5m AHD for the marina and a hydraulic roughness of 0.04;
- A series of wetlands with a finished level of 0.6m AHD, and vegetated areas with a finished level of 1.9 m AHD located along the eastern side of the site located within the 40 m wide riparian zone of the Georges River, with an adopted hydraulic roughness of 0.09;
- A car park in the north-west corner of the site raised to 4.7 m AHD with an adopted hydraulic roughness of 0.02;

- A proposed 6 storey building on the western side of the site with car parking on the ground floor at 4.7 m AHD which is suspended above a 1.65 m AHD finished ground level at the southern end of the building, with a hydraulic roughness of 0.12; and
- A lower car park located on the southern side of the site with a ground level of at 1.65 m AHD and a hydraulic roughness of 0.02.

The post-development DEM is shown in **Figure 2-7**, the post-development land-use / roughness map included as **Figure 2-8**, and a comparison of the levels adopted in the pre-development and post-development DEMs has been included in **Figure 2-9**.

A post-development building blockage sensitivity analysis was also assessed. The sensitivity analysis was based on the assumption that the area under the southern side of the building will be 50% blocked during a rainfall event by debris and other objects. The blockage scenario sensitivity analysis DEM is shown in **Figure 2-10**. A comparison of the levels adopted in the pre-development and post-development DEMs for this option has been included in **Figure 2-11.**

3. FLOOD IMPACT ASSESSMENT

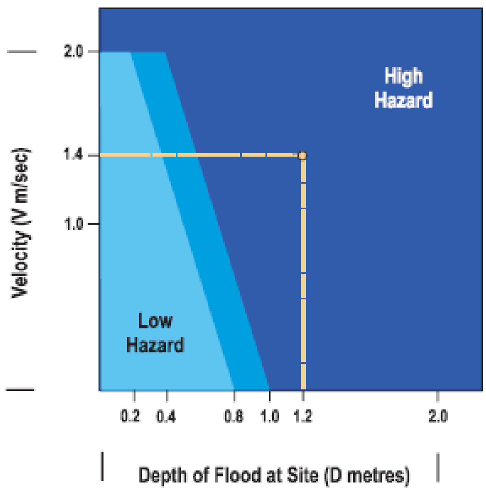
3.1 Pedestrian and Vehicular Stability

When considering pedestrian and vehicular stability, three velocity x depth criteria were identified as follows:

Velocity x Depth	Comment
≤ 0.4 m^2/s	This is typically adopted by Councils as a limit of stability for pedestrians
0.4 – 0.6 m^2/s	Unsafe for pedestrians but safe for vehicles if overland flood depths do not exceed around 0.3 m
> 0.6 m^2/s	This is typically adopted by Councils as a limit of stability for vehicles

3.2 Flood Hazards

Experience from studies of floods throughout NSW and elsewhere has allowed authorities to develop methods of assessing the hazard to life and property on floodplains. This experience has been used in developing the NSW Floodplain Development Manual to provide guidelines for managing this hazard. These guidelines are shown schematically below.



Provisional Hazard Categories
(after Figure L2, NSW Government, 2005)

To use the diagram, it is necessary to know the average depth and velocity of floodwaters at a given location. If the product of depth and velocity exceeds a critical value (as shown below), the flood flow will create a **high hazard** to life and property. There will probably be danger to persons caught in the floodwaters, and possible structural damage. Evacuation of persons would be difficult. By contrast, in **low hazard** areas people and their possessions can be evacuated safely by trucks. Between the two categories a transition zone is defined in which the degree of hazard is dependent on site conditions and the nature of the proposed development. This calculation leads to a provisional hazard rating. The provisional hazard rating may be modified by consideration of effective flood warning times, the rate of rise of floodwaters, duration of flooding and ease or otherwise of evacuation in times of flood.

3.3 Benchmark Conditions

The estimated 20 yr ARI flood levels, depths, velocities, velocity x depth and hazards under benchmark conditions are plotted in **Figures 3-1, 3-2, 3-3, 3-4** and **3-5** respectively.

The estimated 100 yr ARI flood levels, depths, velocities, velocity x depth and hazards under benchmark conditions are plotted in **Figures 3-6, 3-7, 3-8, 3-9** and **3-10** respectively.

It was concluded that:

- Peak flow velocities across the site are low;
- Velocities in the floodplain southwest of the site are even lower; and that
- The floodplain southwest of the site is flood storage in floods up to the 100 yr ARI event.

3.4 Post-Development Conditions

Two development scenarios were assessed as follows:

- Development of the Georges Cove site as given in **Figure 2-7** with no blockage factor applied under Marina building
- Development of the Georges Cove site as given in **Figure 2-10** with a 50% blockage factor applied under Marina building

The estimated 20 yr ARI flood levels, depths, velocities, velocity x depth and hazards under the Post-development conditions are plotted in **Figures 3-11, 3-12, 3-13, 3-14** and **3-15** respectively.

The estimated 100 yr ARI flood levels, depths, velocities, velocity x depth and hazards under the Post-development conditions are plotted in **Figures 3-16, 3-17, 3-18, 3-19** and **3-20** respectively.

The estimated 20 yr ARI flood levels, depths, velocities, velocity x depth and hazards under Post-development – blockage conditions are plotted in **Figures 3-21, 3-22, 3-23, 3-24** and **3-25** respectively.

The estimated 100 yr ARI flood levels, depths, velocities, velocity x depth and hazards under Post-development – blockage conditions are plotted in **Figures 3-26, 3-27, 3-28, 3-29** and **3-30** respectively.

4. FLOOD IMPACT ASSESSMENT

4.1 Post-development Condition

The post-development condition represents the marina as proposed in the applicant's Development Application.

The flood level differences in a 20 Yr ARI and 100 yr ARI events for Post-development Conditions are plotted in **Figures 4-1** and **4-2** respectively. The flood velocity differences in a 20 Yr ARI and 100 yr ARI events for Post-development Conditions are plotted in **Figures 4-5** and **4-6** respectively.

In a 20 yr ARI flood it was assessed that the planned development would locally reduce the 20 yr ARI flood levels west of the site by up to 0.03 m while a small local increase would occur in the entry channel to the marina. This local increase is confined to the waterway and does not impact on any other property. Similarly in the 100 yr ARI flood it was assessed that the planned development would locally reduce the 100 yr ARI flood levels southwest of the site by up to 0.03 m while a small local increase would occur in the entry channel to the marina and within the site. These local increases do not impact on any other property.

In the 20 yr ARI flood the peak velocity within the site of up to around 0.9 m/s occurs locally in the southern car park (Car Park B in Figure 1-2) while in the 100 yr ARI flood the peak velocity in this area increase to around 1.0 m/s.

In the 20 yr ARI flood and the 100 yr ARI flood the planned development reduces peak velocities within the site waterways and adjoining areas by greater than 0.04 m/s while in the south-west corner of the site the peak velocity is increased by up to 0.04 m/s. The peak velocity in the Newbridge Road drainage corridor increases by up to 0.04 m/s in the 20 yr ARI flood and the 100 yr ARI flood. The peak velocity in the majority of the land south-west of the site decreases by up to 0.04 m/s or more in the 20 yr ARI flood and the 100 yr ARI flood.

In a 20 yr ARI flood and the 100 yr ARI flood the impact of the planned development on peak velocity x depth is negligible except where the local raising of the ground levels to 4.7 m AHD eliminates the velocity x depth in comparison with pre-development conditions.

On the landform raised to 4.7 m AHD the velocity x depth under the building and in the elevated car park (Car Park C in Figure 1-2) is around 0.6 m^2/s in the 100 yr ARI flood (ie. the stability limit for vehicles).

In a 20 yr ARI event the impact of the planned development on flood hazard is negligible except where planned raising of the ground level on the western side of the site eliminates the flood hazard while in the 100 yr ARI flood the impact of the planned development on flood hazard is negligible.

It is concluded that the proposed development has negligible impact on the behaviour of flooding in the flood storage area located southwest of the proposed marina and nil or negligible impacts on any other adjacent properties.

4.2 Post-development – Blockage Scenario

This option blocks around 50% of the undercroft under the marina building further to the south. The flood level differences in a 20 yr ARI and 100 yr ARI events for the blockage scenario are plotted in **Figures 4-3** and **4-4** respectively. The flood velocity differences in a 20 Yr ARI and 100 yr ARI events for Post-development Conditions are plotted in **Figures 4-7** and **4-8** respectively

In a 20 yr ARI flood it was assessed that the partial blockage planned development would locally reduce the 20 yr ARI flood levels west of the site by up to 0.02 m while a small local increase would occur in the entry channel to the marina. This local increase is confined to the waterway and does not impact on any other property. Similarly in the 100 yr ARI flood it was assessed that the planned development would locally reduce the 100 yr ARI flood levels southwest of the site by up to 0.03 m while a small local increase would occur in the entry channel to the marina and within the site. While the local impact within the site is greater than under the nil blockage scenario these local increases do not impact adversely on any other property.

In the 20 yr ARI flood and the 100 yr ARI flood the planned development with partial blockage reduces peak velocities within the site waterways and adjoining areas by greater than 0.04 m/s while in the south-west corner of the site the peak velocity is increased by up to 0.04 m/s. The peak velocity in the Newbridge Road drainage corridor increases by up to 0.04 m/s in the 20 yr ARI flood and the 100 yr ARI flood. The peak velocity in the majority of the land south-west of the site decreases by up to 0.04 m/s or more in the 20 yr ARI flood and the 100 yr ARI flood.

In a 20 yr ARI flood the impact of the planned development and partial blockage on peak velocity x depth is negligible except where the local raising of the ground levels to 4.7 m AHD eliminates peak velocity x depth in comparison with pre-development conditions. In the 100 yr ARI flood planned development and partial blockage creates a zone of higher velocity x depth on the northwestern corner of the site.

In a 20 yr ARI event the impact of the planned development and partial blockage on flood hazard is negligible except where planned raising of the ground level on the western side of the site eliminates the flood hazard while in the 100 yr ARI flood the impact of the planned development and partial blockage on flood hazard is negligible.

5. CONCLUSIONS

The main features of the proposed development are as follows:

- A marina basin located in the middle of the site with an assumed an invert level of -3.5m AHD for the marina and a hydraulic roughness of 0.04;
- A series of wetlands with a finished level of 0.6m AHD, and vegetated areas with a finished level of 1.9 m AHD located along the eastern side of the site located within the 40 m wide riparian zone of the Georges River, with an adopted hydraulic roughness of 0.09;
- A car park in the north-west corner of the site raised to 4.7 m AHD with an adopted hydraulic roughness of 0.02;
- A proposed 6 storey building on the western side of the site with car parking on the ground floor at 4.7 m AHD which is suspended above a 1.65 m AHD finished ground level at the southern end of the building, with a hydraulic roughness of 0.12; and
- A lower car park located on the southern side of the site with a ground level of at 1.65 m AHD and a hydraulic roughness of 0.02.

The post-development DEM is shown in **Figure 2-7** while the post-development land-use / roughness map included as **Figure 2-8**. A post-development with 50% blockage of the undercroft was also assessed.

5.1 Post Development

It was concluded from the assessment of Post Development Conditions that:

(i) In a 20 yr ARI flood the planned development would locally reduce the 20 yr ARI flood levels west of the site by up to 0.03 m while a small local increase would occur in the entry channel to the marina.

(ii) Similarly in the 100 yr ARI flood the planned development would locally reduce the 100 yr ARI flood levels southwest of the site by up to 0.03 m while a small local increase would occur in the entry channel to the marina and within the site.

(iii) The local increases in 20 yr ARI and 100 yr ARI are in a very limited area only and do not impact on any other property;

(iv) In the 20 yr ARI flood and the 100 yr ARI flood the planned development reduces peak velocities within the site waterways and adjoining areas by greater than 0.04 m/s while in the south-west corner of the site the peak velocity is increased by up to 0.04 m/s;

(v) The peak velocity in the Newbridge Road drainage corridor increases by up to 0.04 m/s in the 20 yr ARI flood and the 100 yr ARI flood while the peak velocity in the majority of the land south-west of the site decreases by up to 0.04 m/s or more in the 20 yr ARI flood and the 100 yr ARI flood;

(vi) In a 20 yr ARI flood the velocity x depth on land southwest of the site is reduced in some areas while in the 100 yr ARI flood the impact of the planned development on peak velocity x depth is negligible except where the local raising of the ground levels to 4.7 m AHD eliminates the velocity x depth in comparison with pre-development conditions;

(vii) On the landform raised to 4.7 m AHD the velocity x depth under the building and in the elevated car park (Car Park C in Figure 1-2) is around 0.6 m2/s in the 100 yr ARI flood (ie. the stability limit for vehicles).

(viii) In a 20 yr ARI event the impact of the planned development on flood hazard is negligible except where planned raising of the ground level on the western side of the site eliminates the flood hazard while in the 100 yr ARI flood the impact of the planned development on flood hazard is negligible; and

(ix) The proposed development has negligible impact on the behaviour of flooding in the flood storage area located southwest of the proposed marina and nil or negligible impacts on any other adjacent properties.

5.2 Post Development with Blockage

It was concluded from the assessment of Post Development conditions with 50% blockage of the undercroft that:

(i) In a 20 yr ARI flood the planned development would locally reduce the 20 yr ARI flood levels west of the site by up to 0.02 m while a small local increase would occur in the entry channel to the marina.

(ii) Similarly in the 100 yr ARI flood the planned development would locally reduce the 100 yr ARI flood levels southwest of the site by up to 0.03 m while a small local increase would occur in the entry channel to the marina and within the site.

(iii) The local increases in 20 yr ARI and 100 yr ARI are in a very limited area only and do not impact on any other property;

(iv) In the 20 yr ARI flood and the 100 yr ARI flood the planned development with partial blockage reduces peak velocities within the site waterways and adjoining areas by greater than 0.04 m/s while in the south-west corner of the site the peak velocity is increased by up to 0.04 m/s;

(v) The peak velocity in the Newbridge Road drainage corridor increases by up to 0.04 m/s in the 20 yr ARI flood and the 100 yr ARI flood while the peak velocity in the majority of the land south-west of the site decreases by up to 0.04 m/s or more in the 20 yr ARI flood and the 100 yr ARI flood;

(vi) In a 20 yr ARI flood the impact of the planned development with partial blockage on peak velocity x depth is minor except where the local raising of the ground levels to 4.7 m AHD eliminates the peak velocity x depth in comparison with pre-development conditions;

(vii) In the 100 yr ARI flood the impact of the planned development with partial blockage on peak velocity x depth is negligible except where the local raising of the ground levels to 4.7 m AHD reduces significantly the peak velocity x depth in comparison with pre-development conditions;

(viii) In a 20 yr ARI event the impact of the planned development with partial blockage on flood hazard is negligible except where planned raising of the ground level on the western side of the site eliminates the flood hazard while in the 100 yr ARI flood the impact of the planned development on flood hazard is negligible.

(ix) The proposed development with partial blockage has negligible impact on the behaviour of flooding in the flood storage area located southwest of the proposed marina and nil or negligible impacts on any other adjacent properties.

Yours faithfully

Brett C. Phillips

……………………………….

Dr Brett C. Phillips
Director, Water Engineering
for **Cardno**

Georges Cove
Flood Impact Assessment

Figure 1-1
Subject Site

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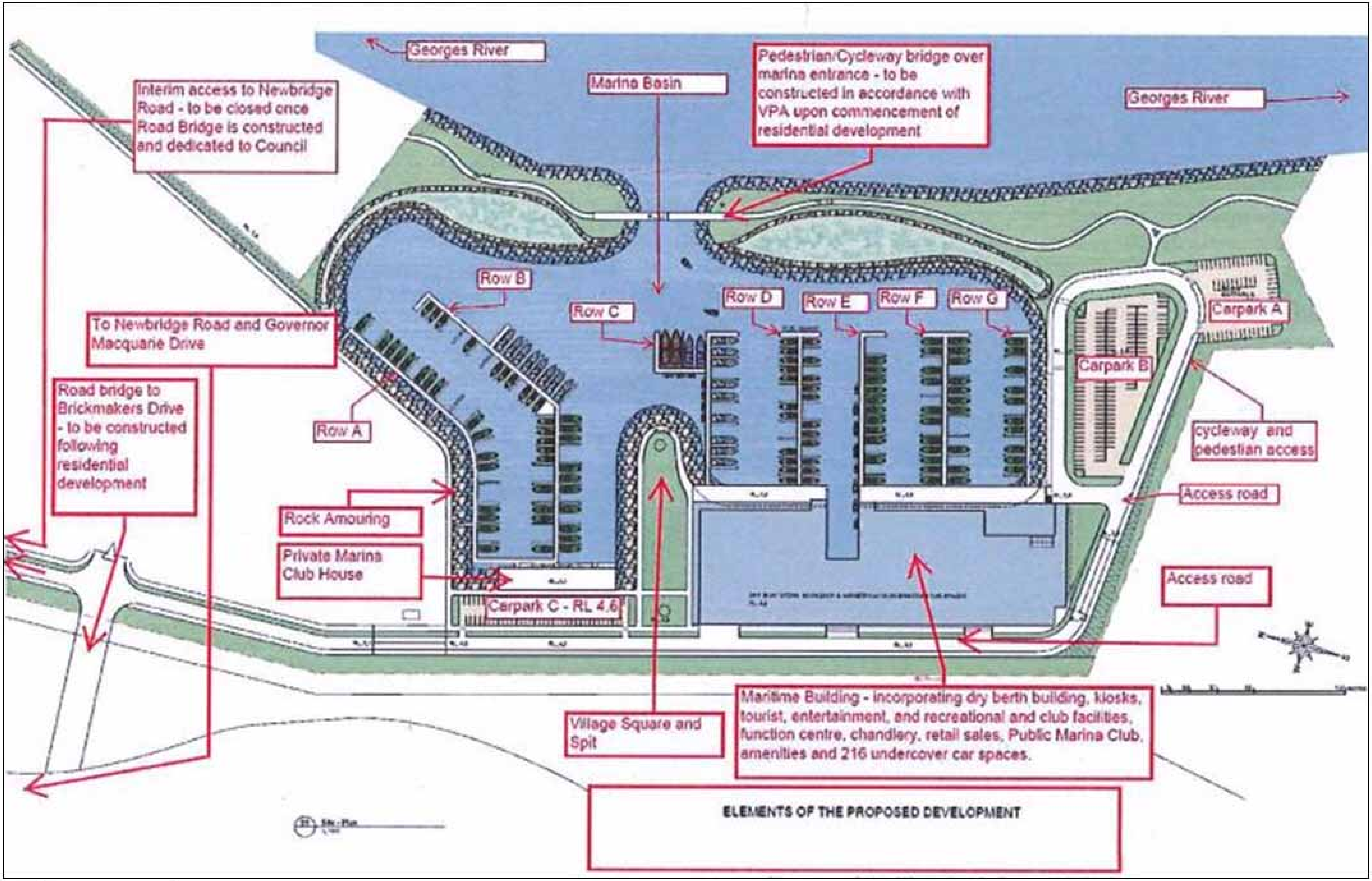
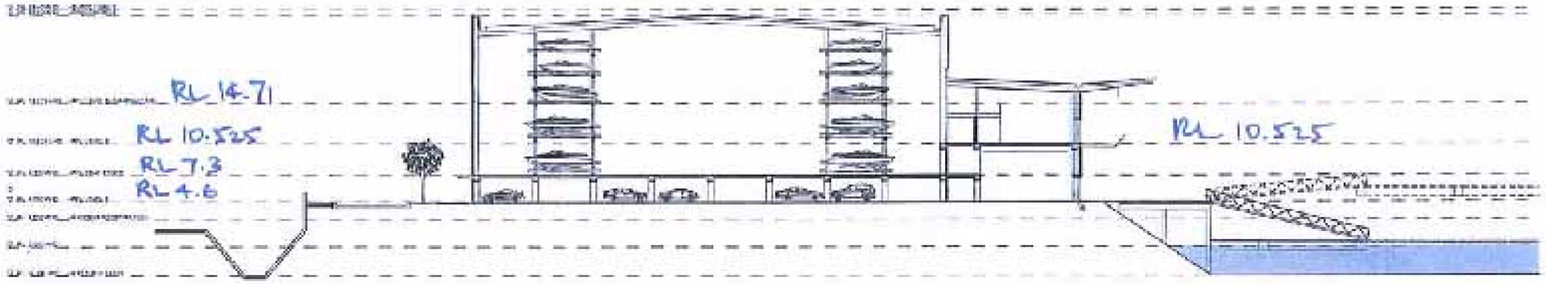


Figure 1-2
Proposed Development
Plan View

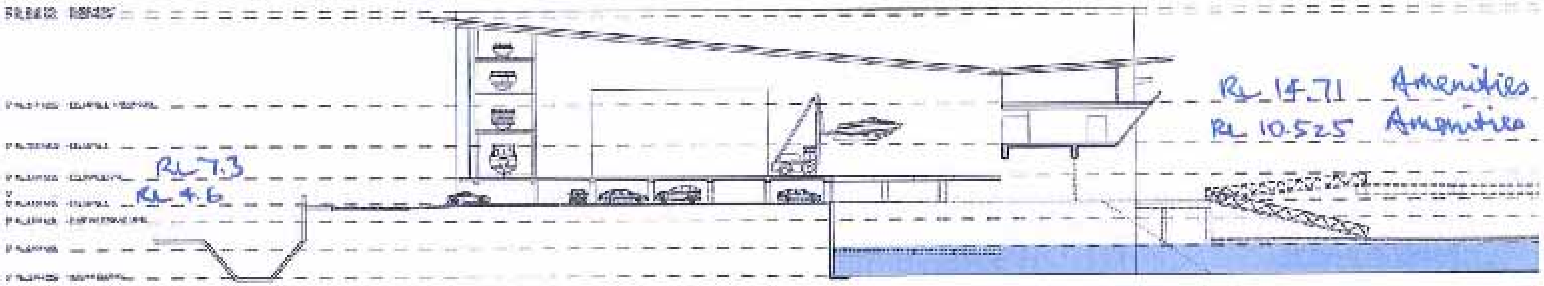
According to Flood Consultant Worley Parsons:
the 1 in 100 year floor level is 5.6 AHD
the flood planning level is 6.1 AHD,
all habitable development should be at or above this level.

We have therefore assumed the following levels.

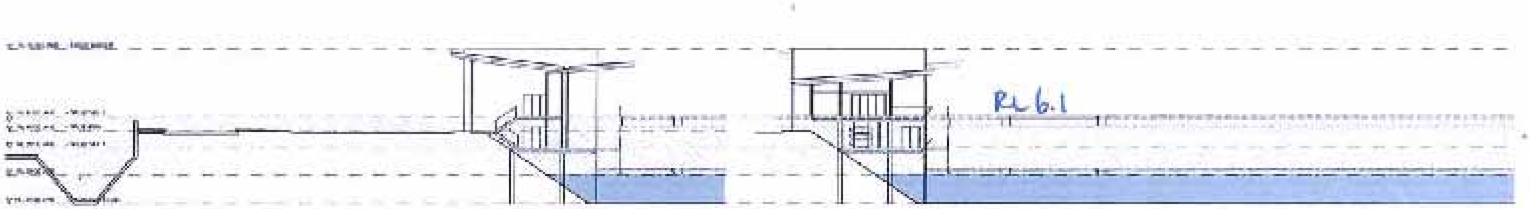
6.10	Commercial spaces, dry berth stores and the hardstand
4.60	Dry store carpark and Northern car park
2.8	Southern car park
0.98	Higher high water springs
	(All levels are AHD)



Section - Dry Berth Store



Section - Dry Berth Store

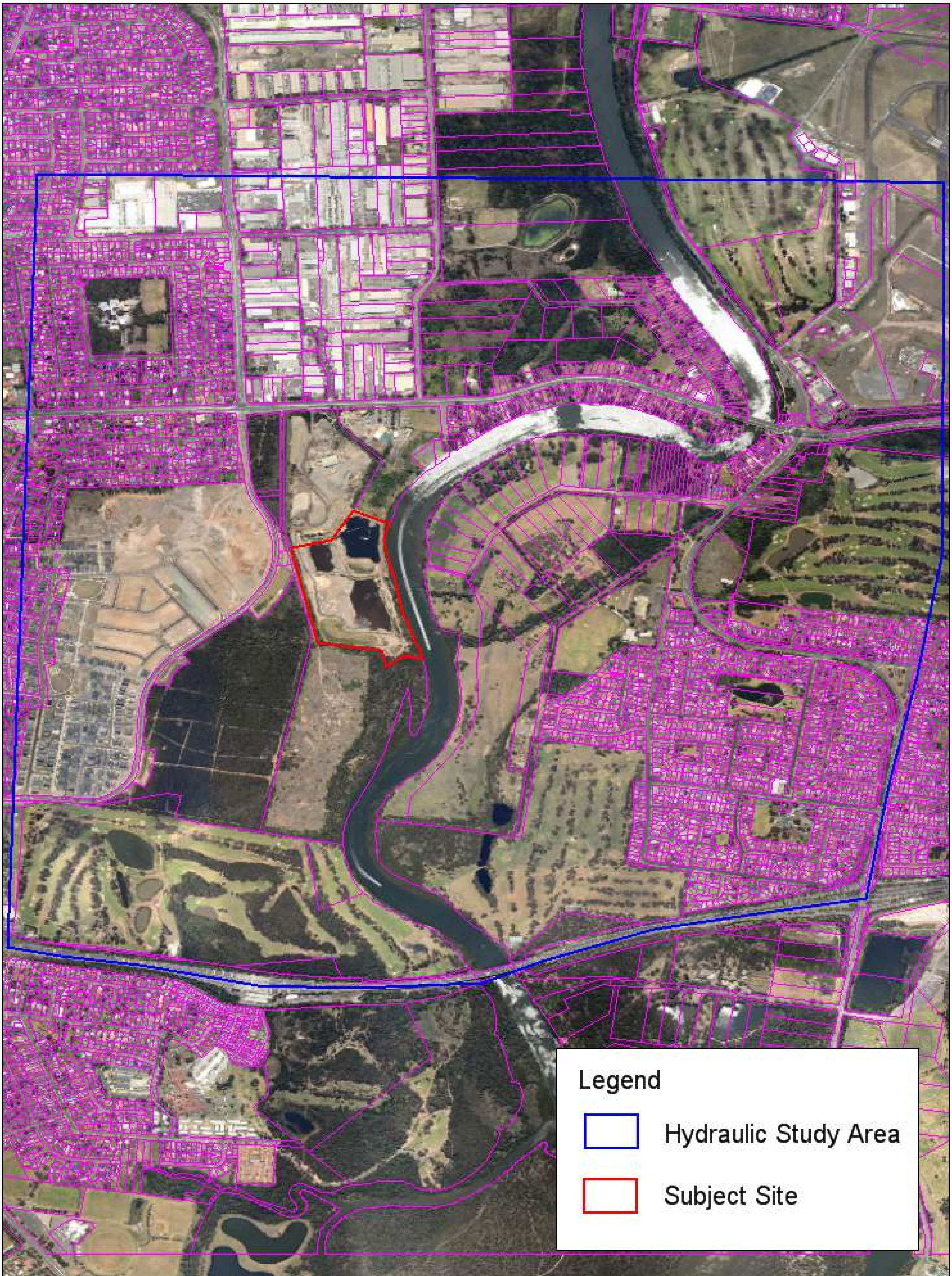


Sections - Private Marina Clubhouse

Prepared by:
MICHEAL FOUNTAIN ARCHITECTS
2/5 Narabang Way Belrose NSW 2085
Tel: 02 9450 2070 Email: mfa@mfa.com.au
Web: www.mfa.com.au

Issue:
TO CLIENT
Date:
11th November 2010

Project:
GEORGES COVE MARINA
NEWBRIDGE ROAD, MOOREBANK
Client:
BENEDICT INDUSTRIES PTY LTD



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Georges Cove
Flood Impact Assessment

Figure 2-1
Hydraulic Study Area

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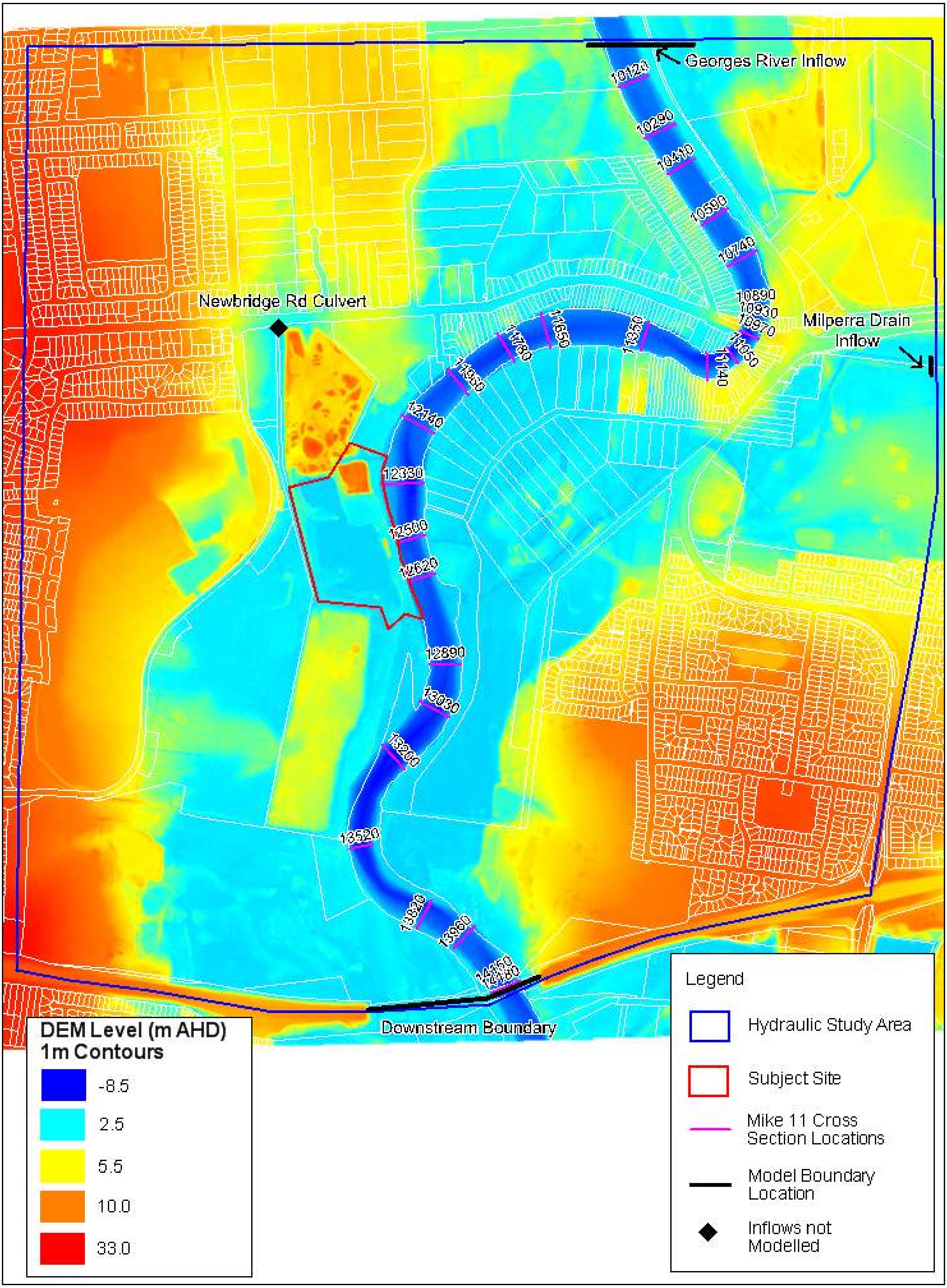

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Flood Impact Assessment

Figure 2-2
Base Model Set-up

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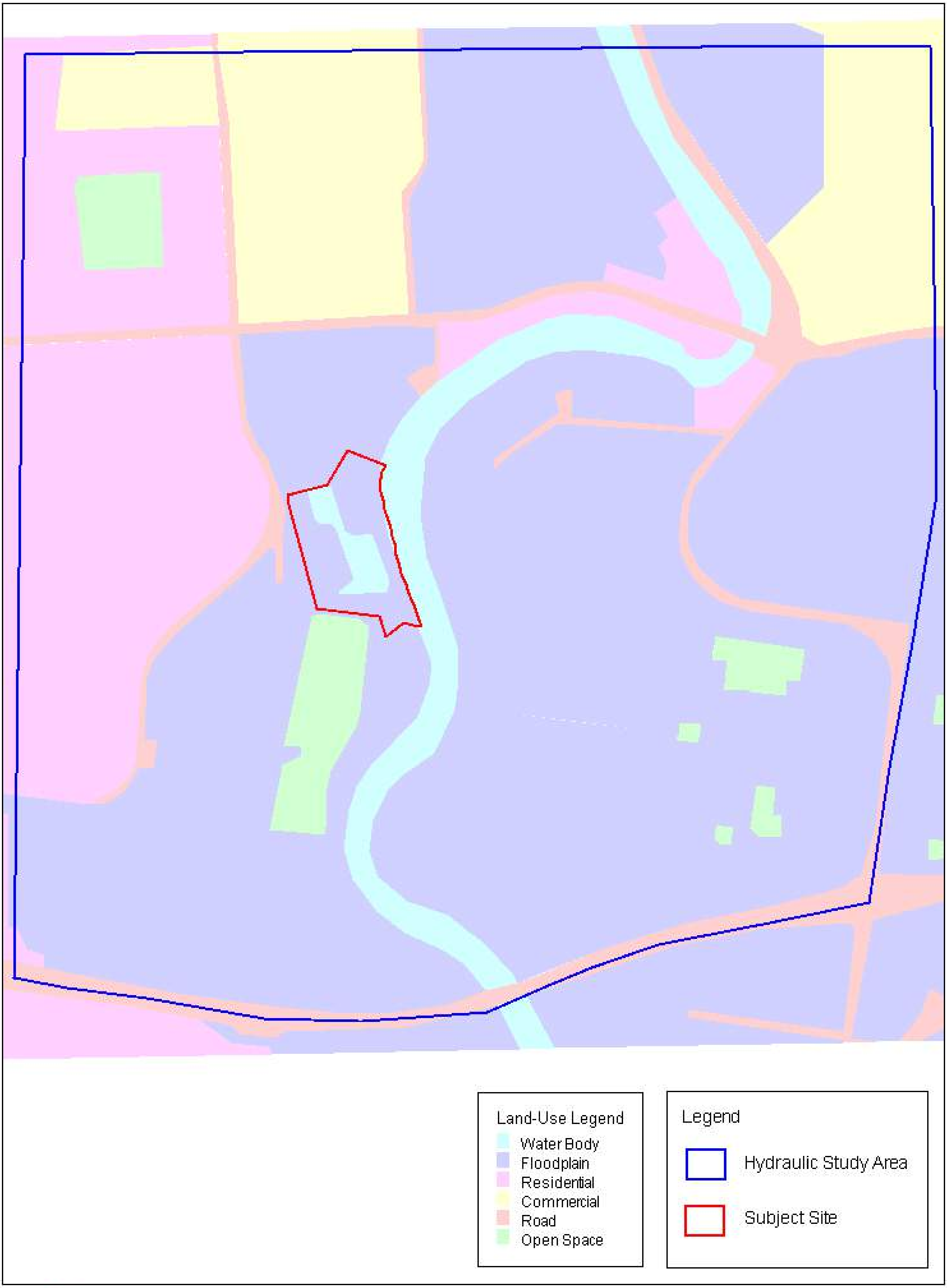


Georges Cove
Flood Impact Assessment

Figure 2-3
Land-Use

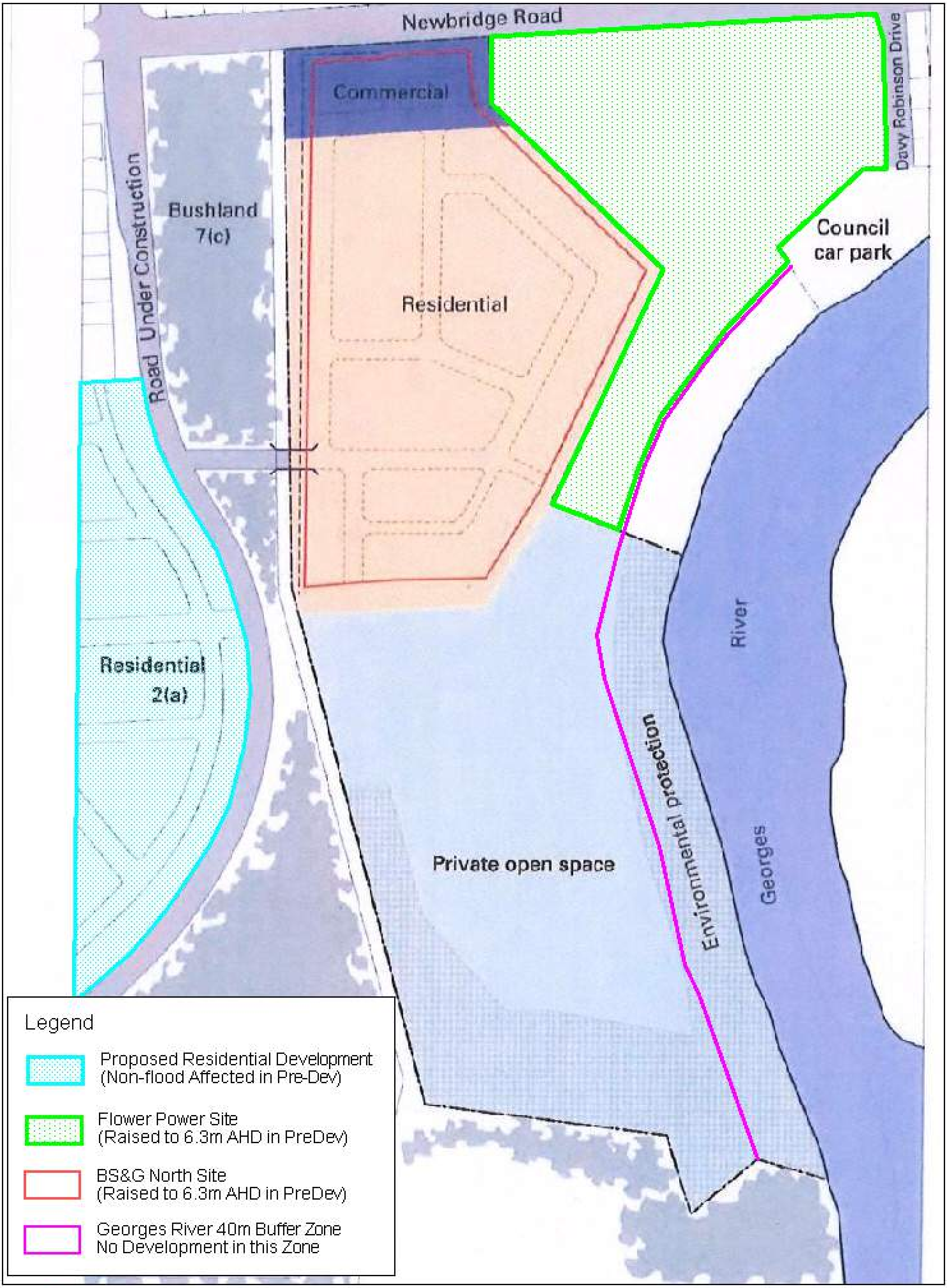
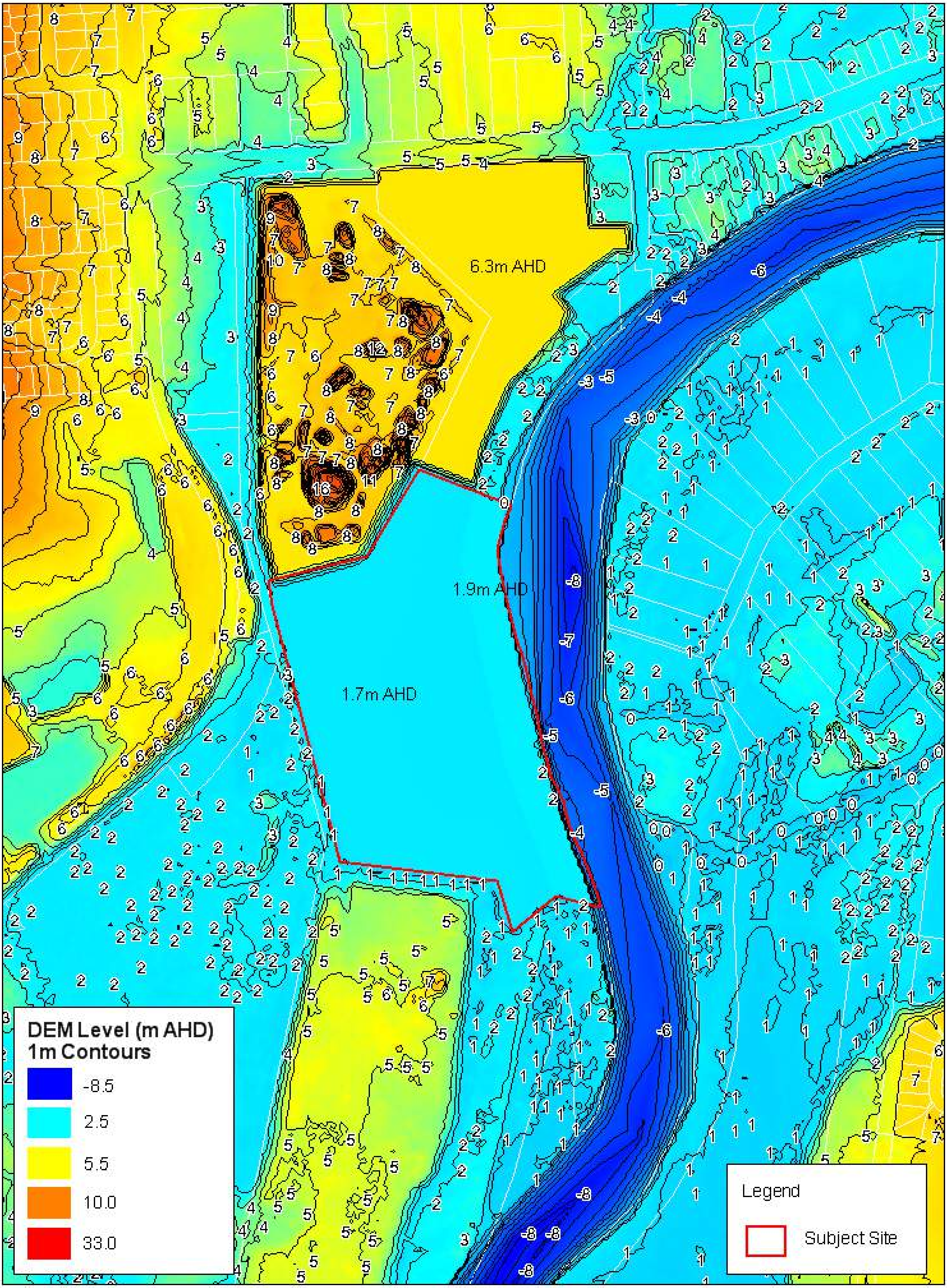


Figure 2-4
Pre-Development Scenario
Development in the Vicinity
of the Site



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Flood Impact Assessment

Figure 2-5
Benchmark (Pre-Development) DEM

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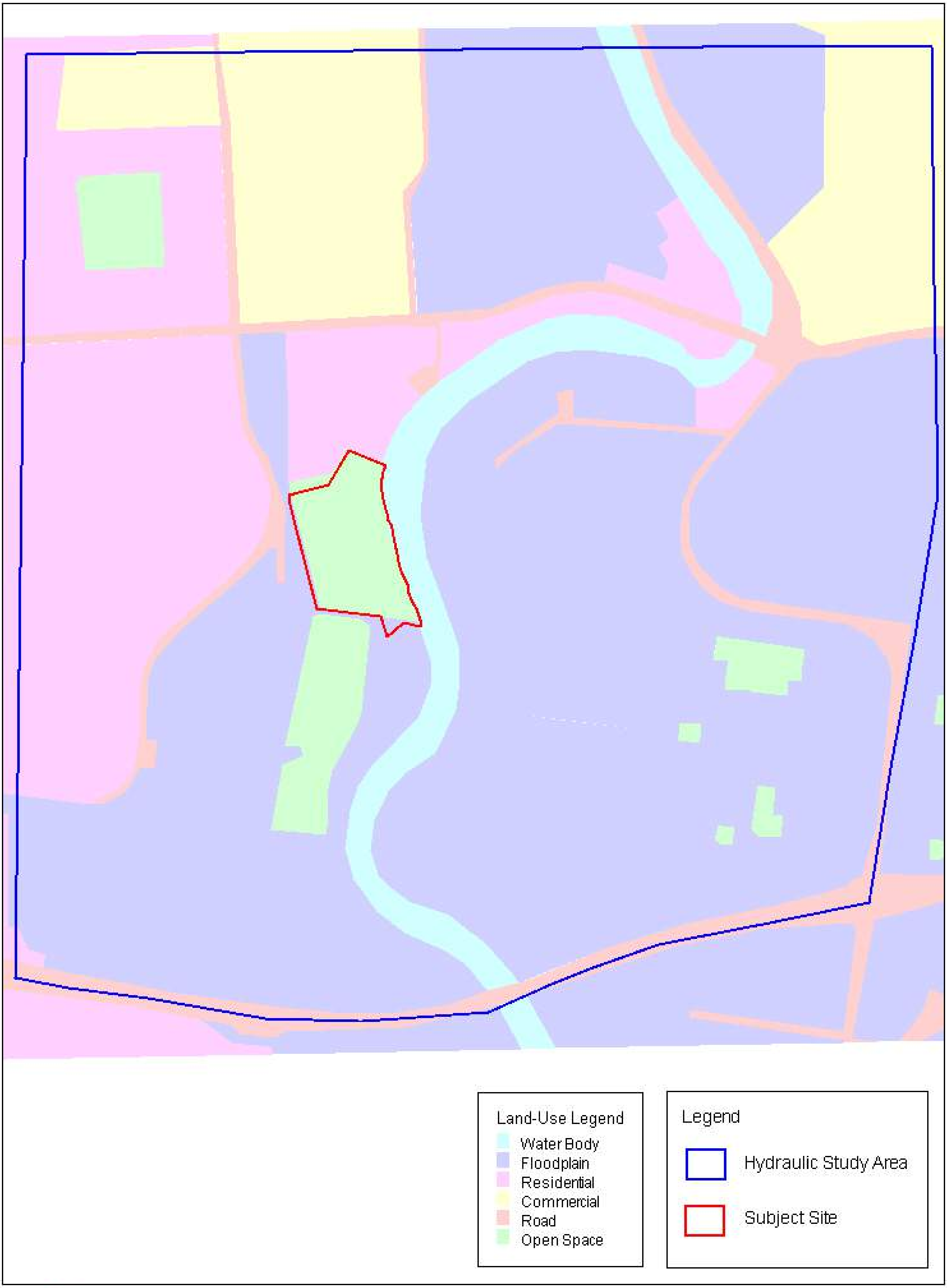
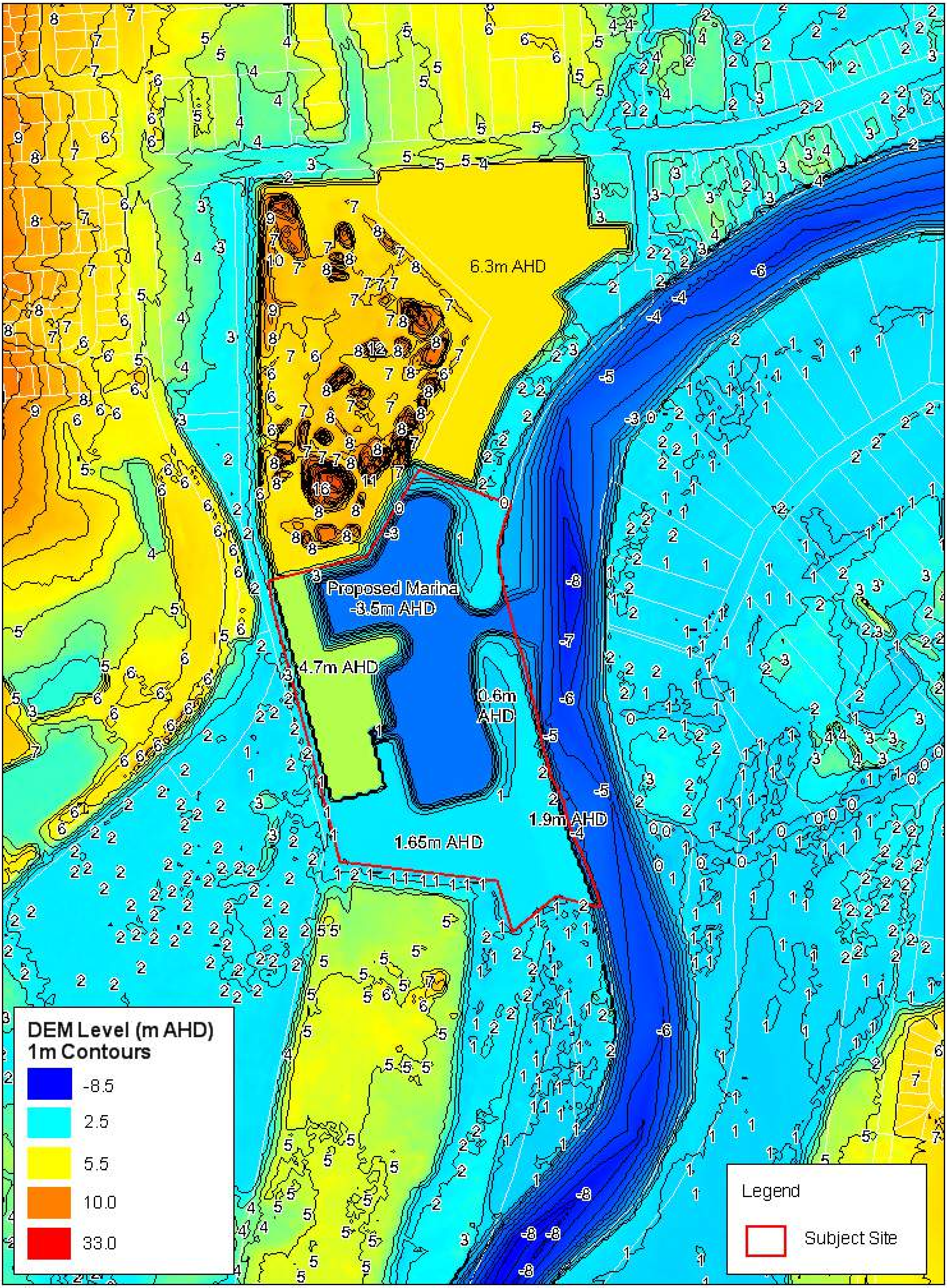

Land-Use Legend
Water Body
Floodplain
Residential
Commercial
Road
Open Space
Legend
Hydraulic Study Area
Subject Site



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Flood Impact Assessment

Figure 2-7
DEM
Post-Development

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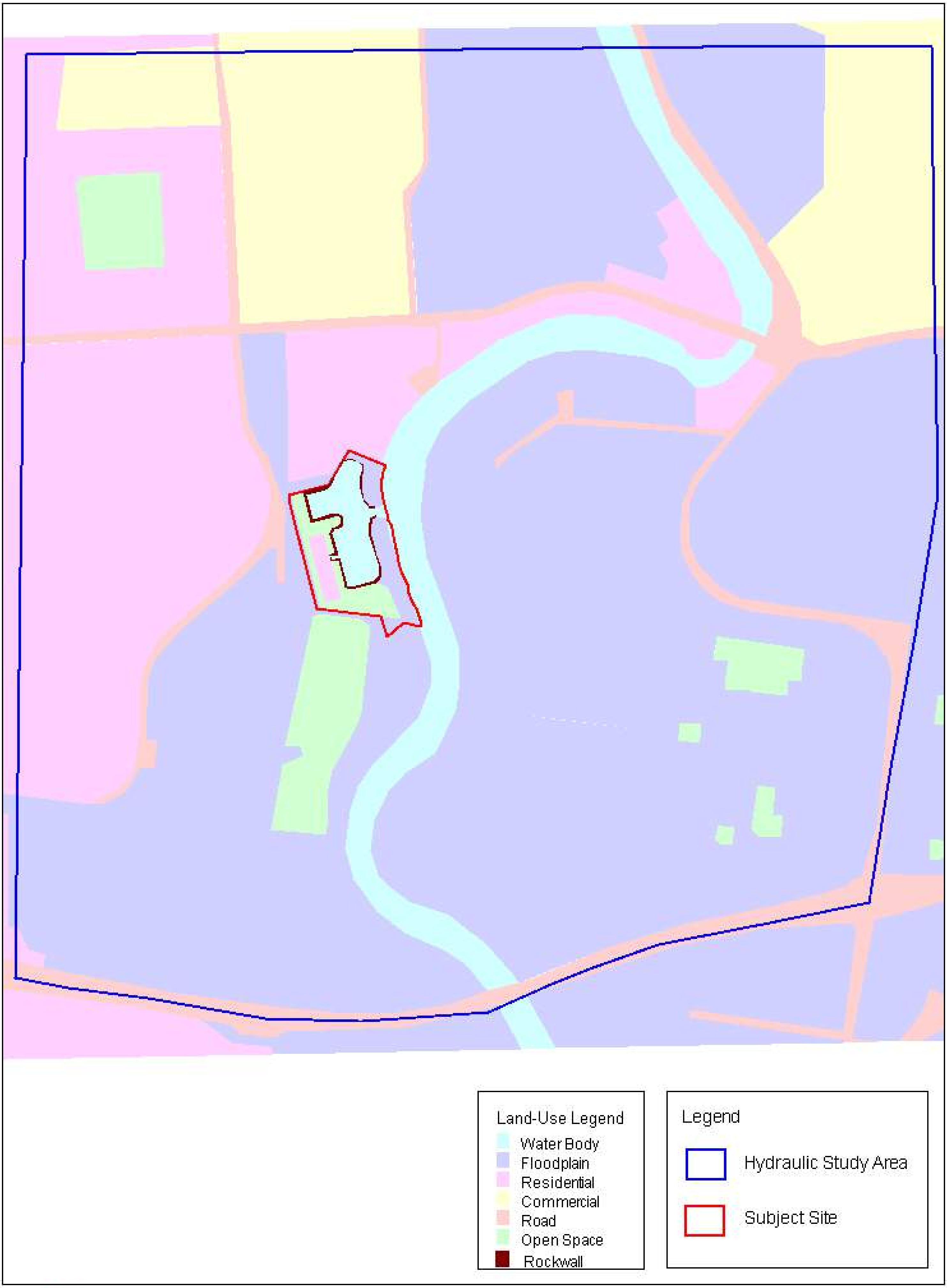
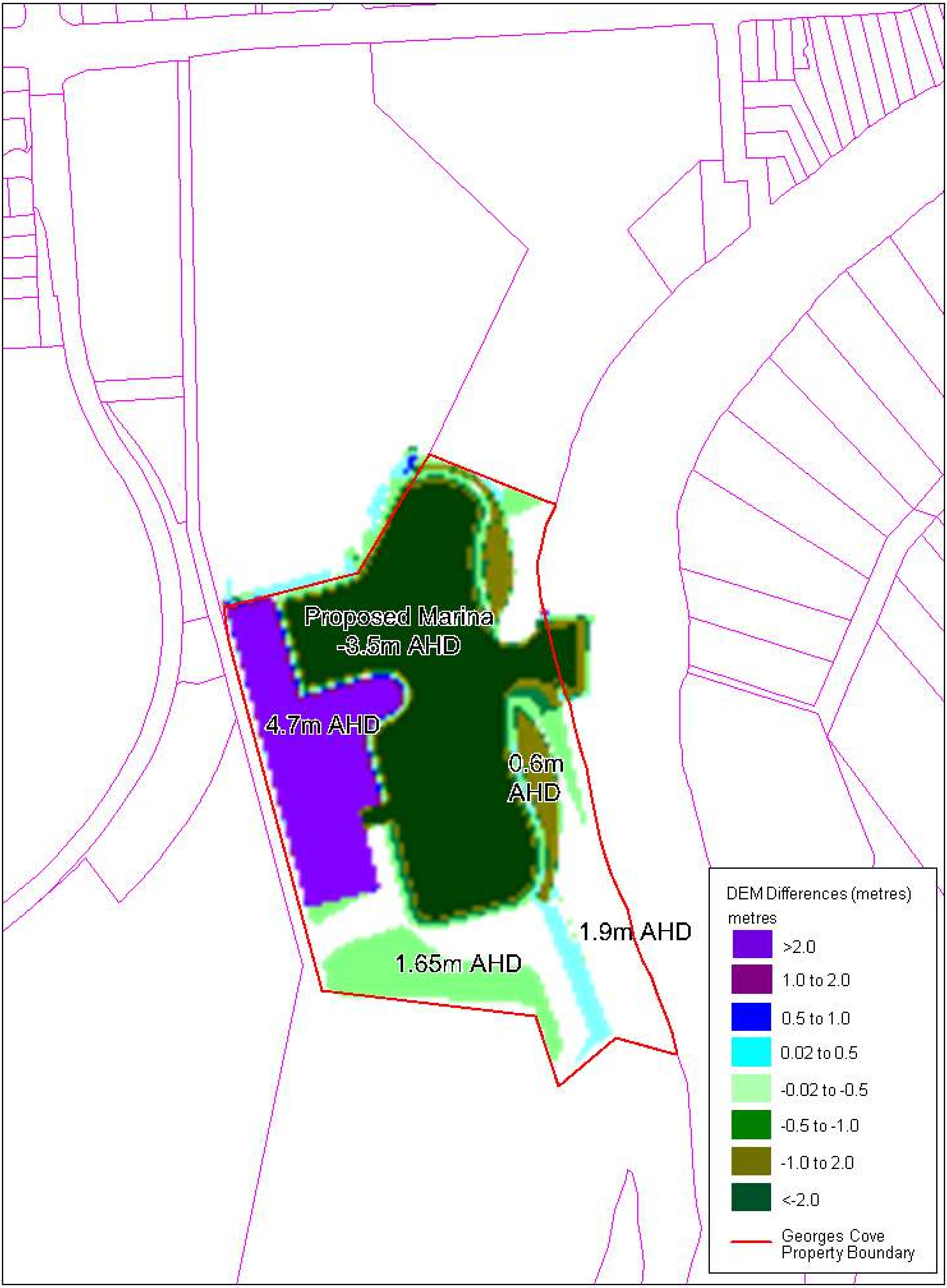


Land-Use Legend
Water Body
Floodplain
Residential
Commercial
Road
Open Space
Rockwall
Legend
Hydraulic Study Area
Subject Site



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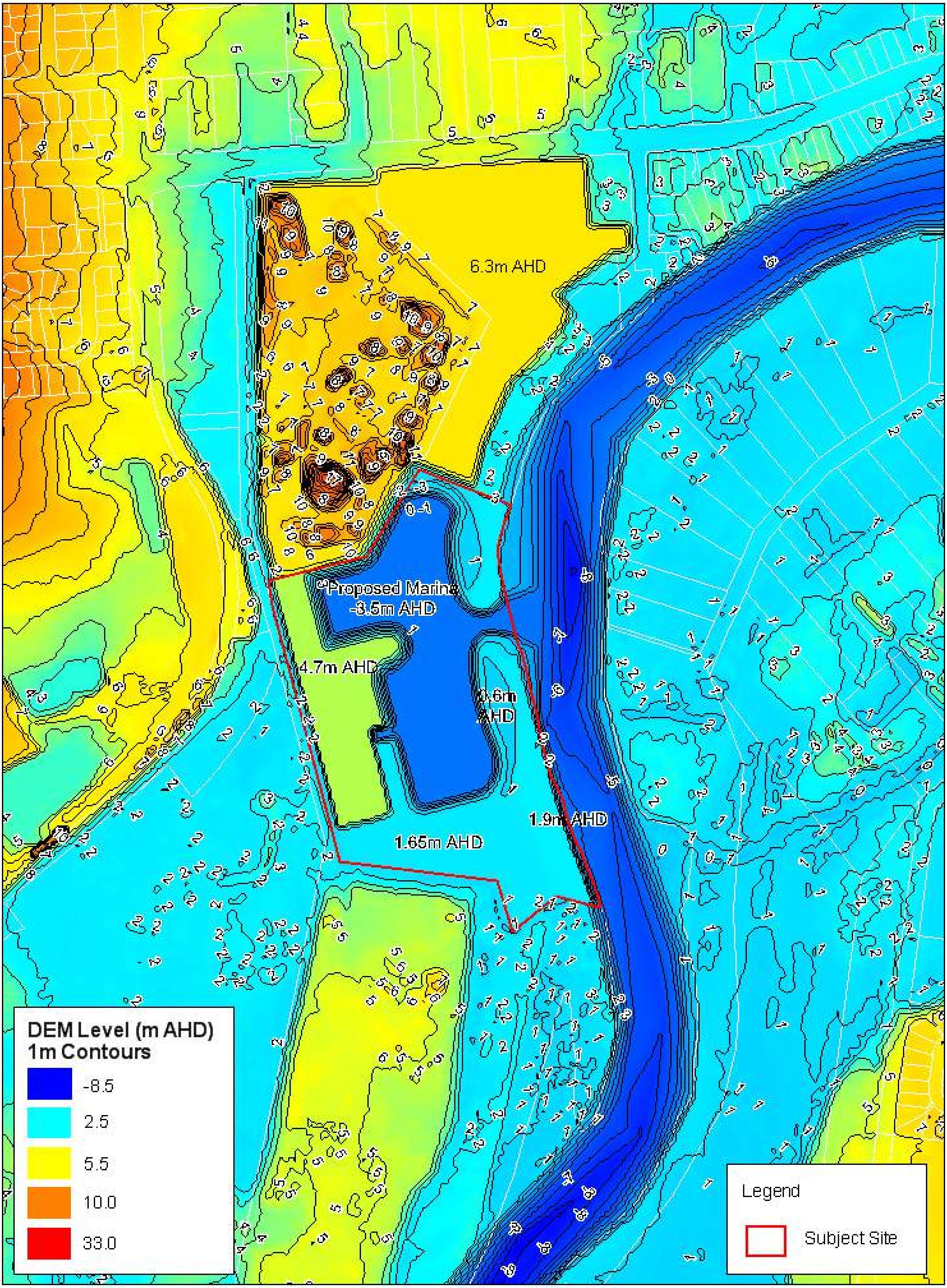
Flood Impact Assessment

Figure 2-9
DEM Difference
Post-development Less Predevelopment



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Flood Impact Assessment

Figure 2-10
DEM
Post-Development-Blockage

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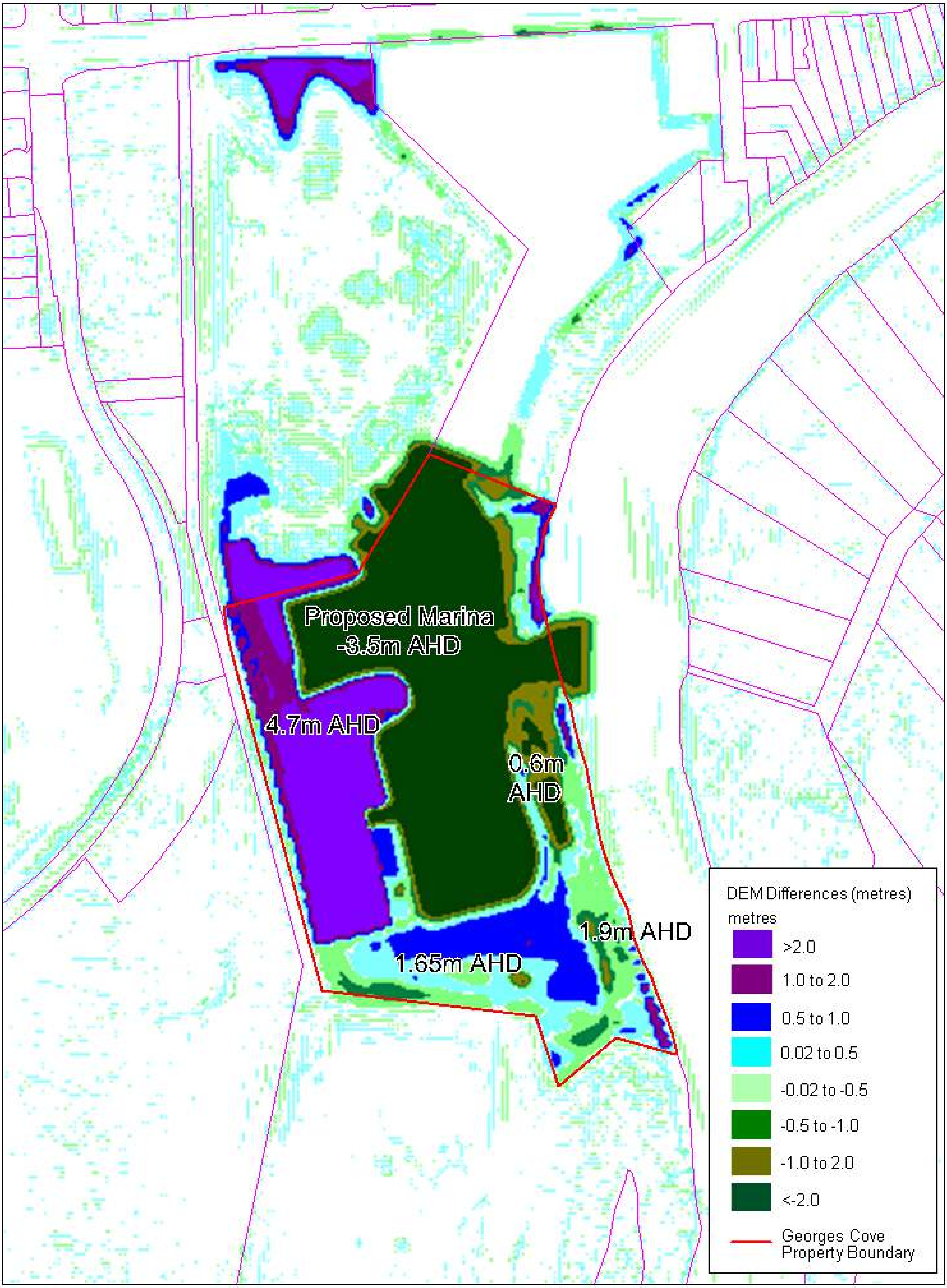

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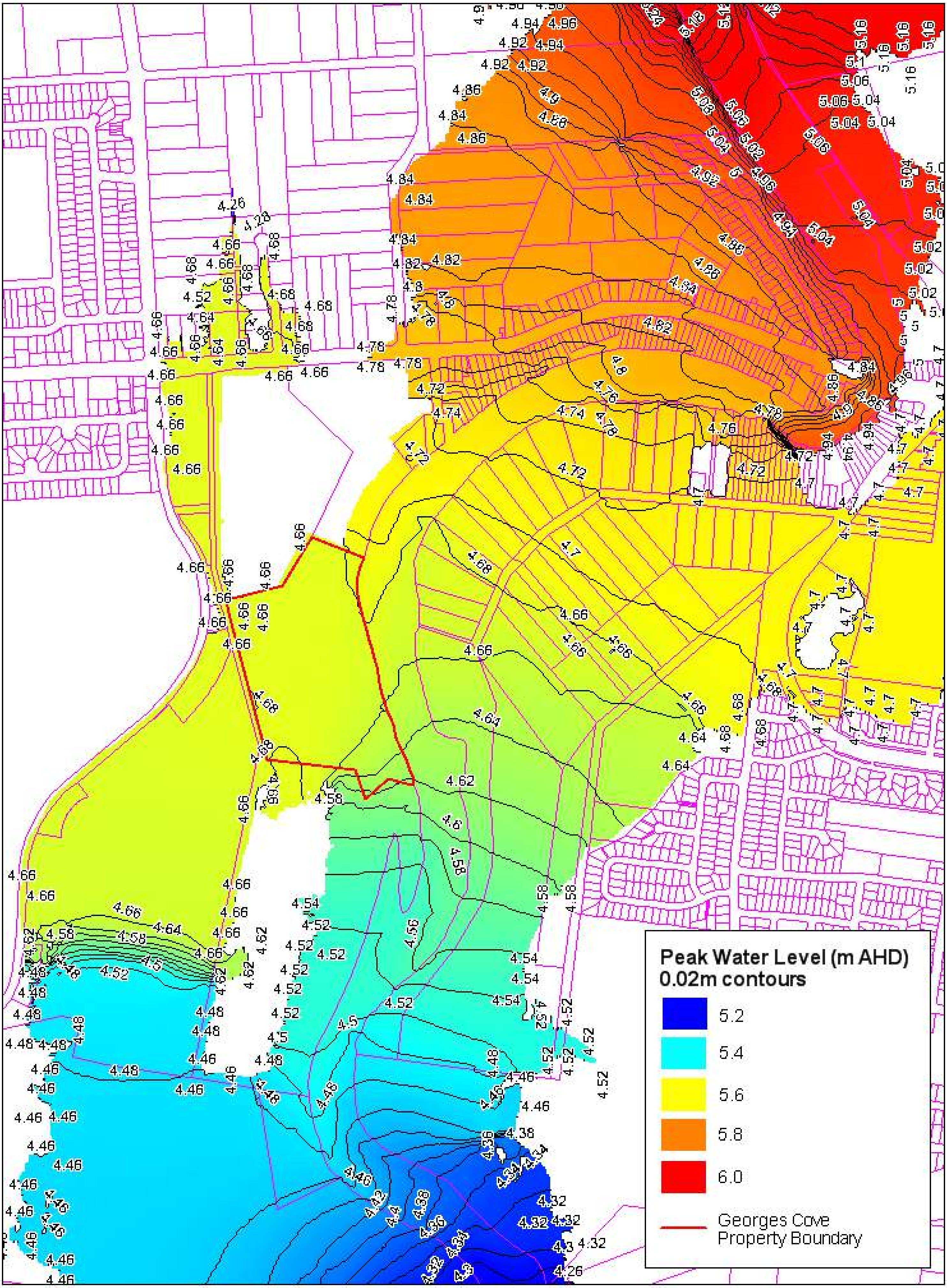
Georges Cove
Flood Impact Assessment

Figure 2-11
DEM Difference
Post-development-Blockage Less Predevelopment



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Georges Cove
Flood Impact Assessment

Figure 3-1
20 year ARI
Peak Water Level
Pre-Development

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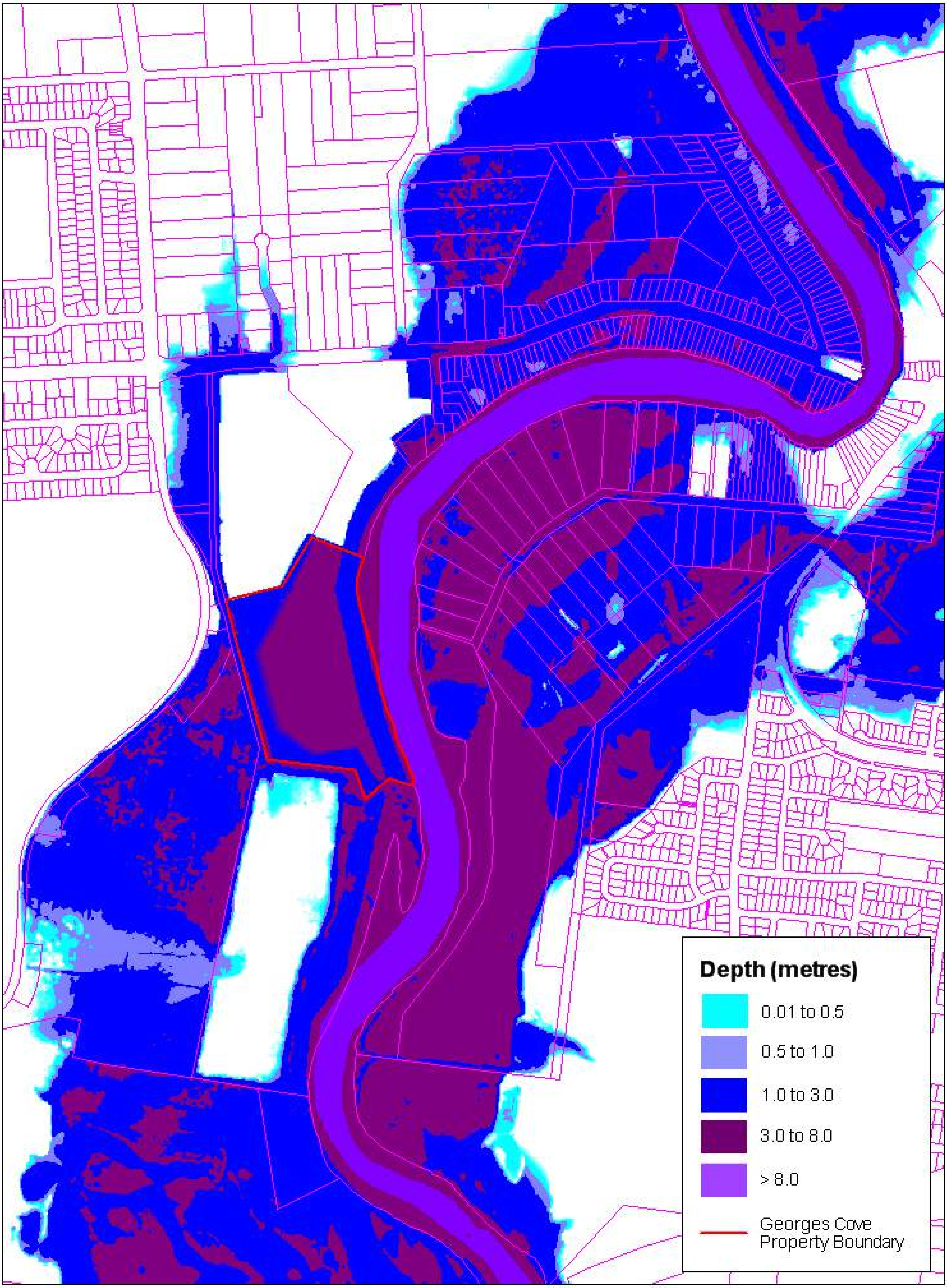
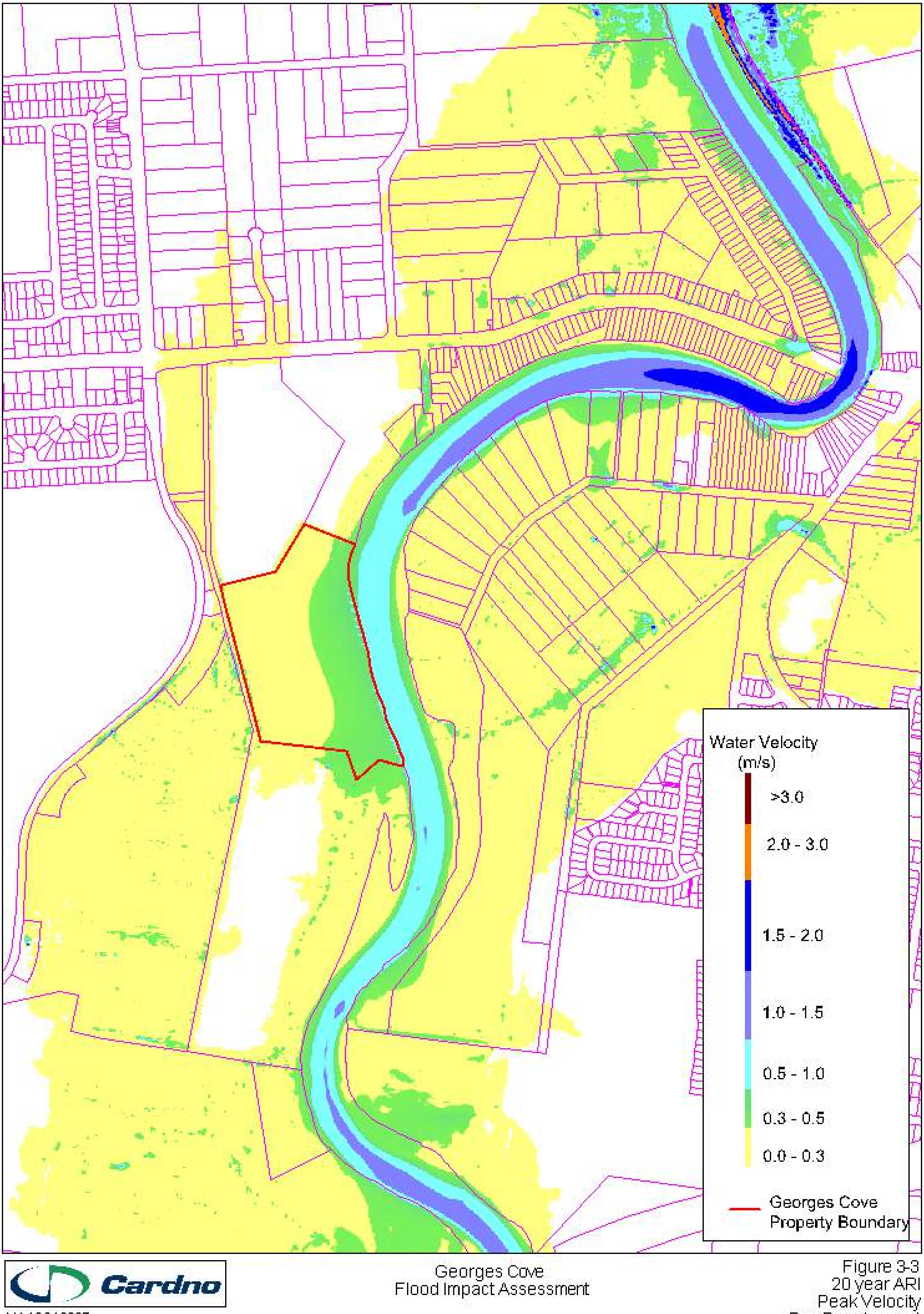


Depth (metres)
0.01 to 0.5
0.5 to 1.0
1.0 to 3.0
3.0 to 8.0
> 8.0
Georges Cove Property Boundary



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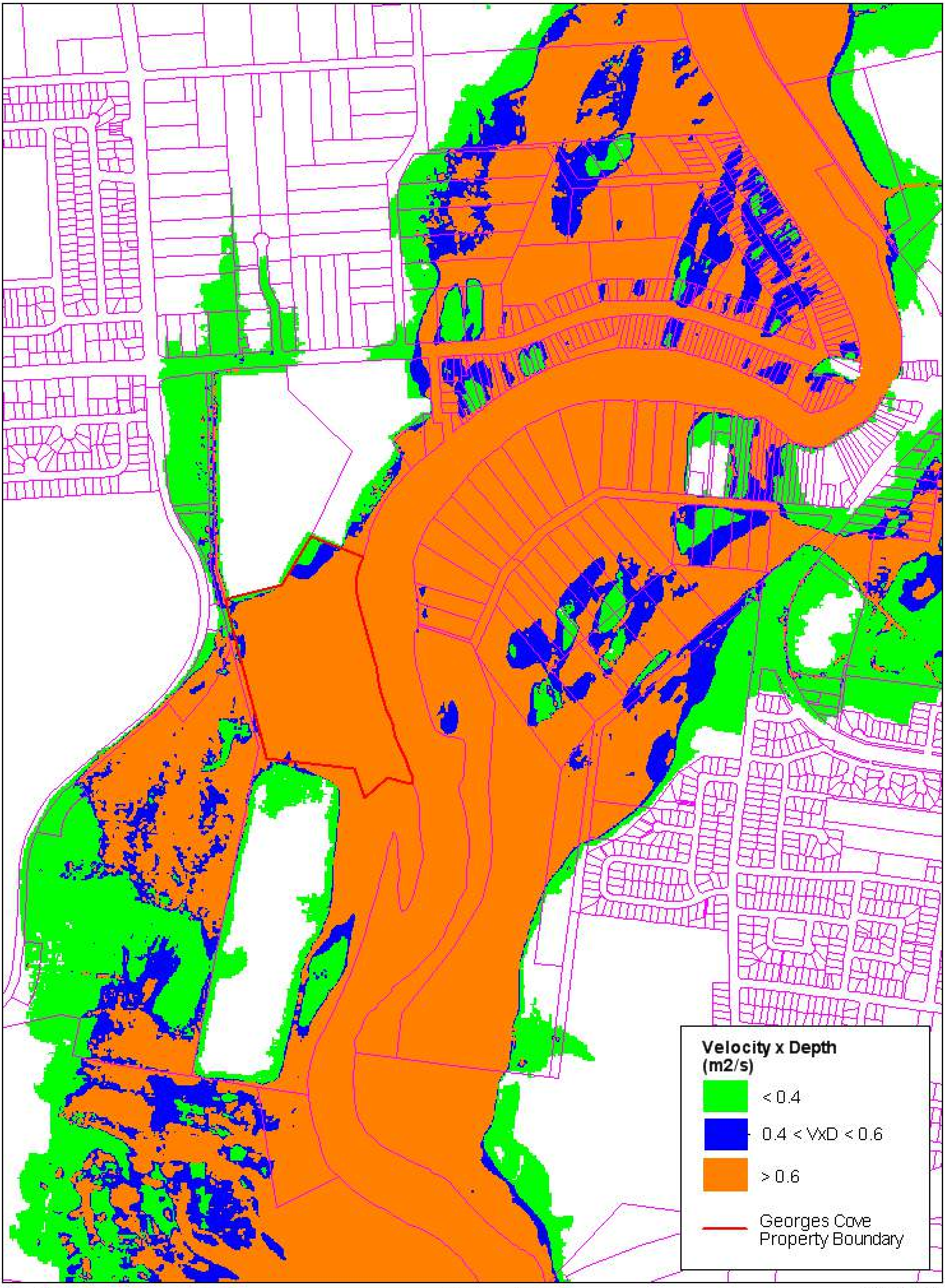
Georges Cove
Flood Impact Assessment

Figure 3-3
20 year ARI
Peak Velocity
Pre-Development



Georges Cove
Flood Impact Assessment

Figure 3-4
20 year ARI
Peak Velocity x Depth
Pre-Development

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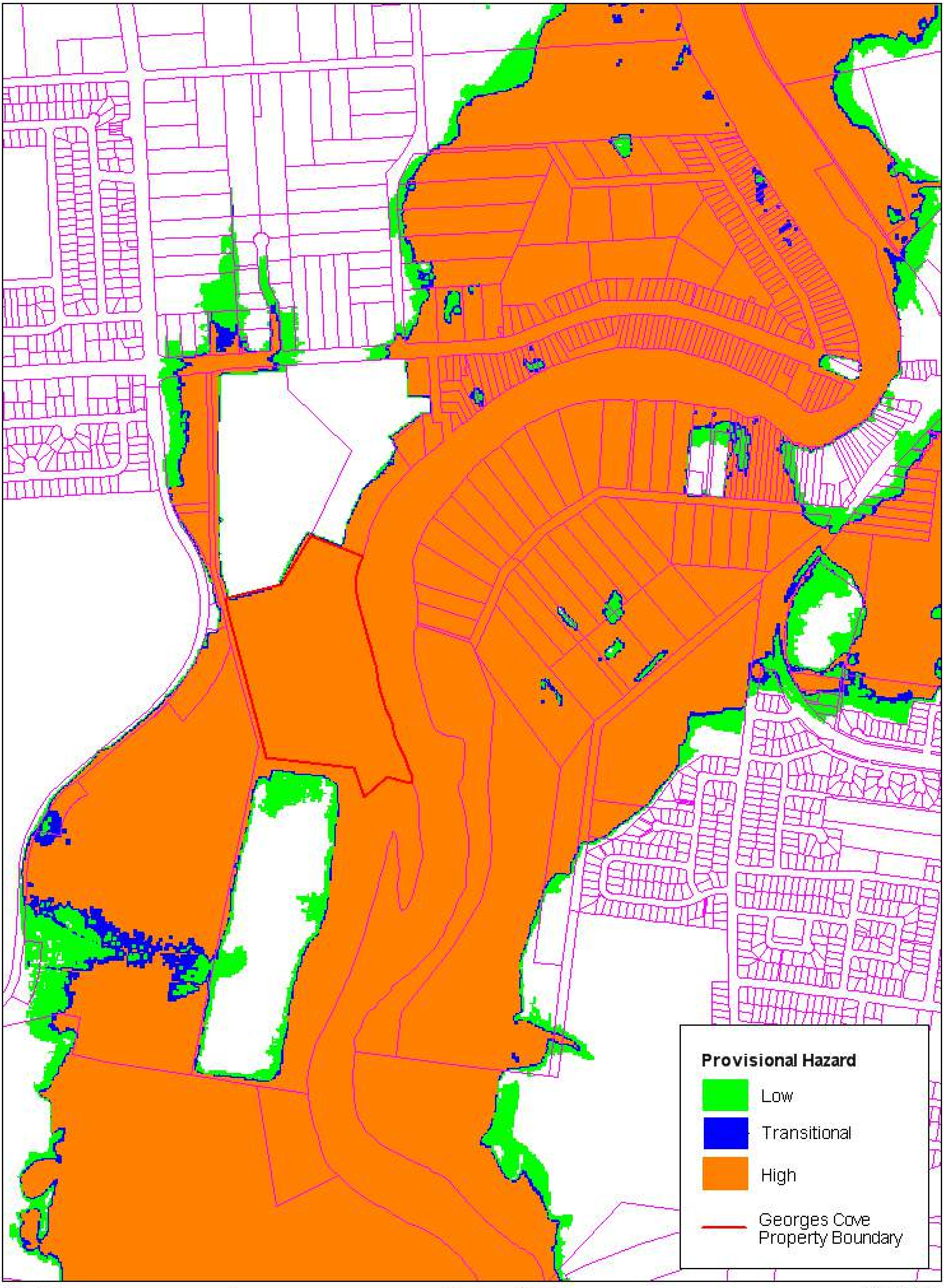

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Georges Cove
Flood Impact Assessment

Figure 3-5
20 year ARI
Provisional Hazard
Pre-Development

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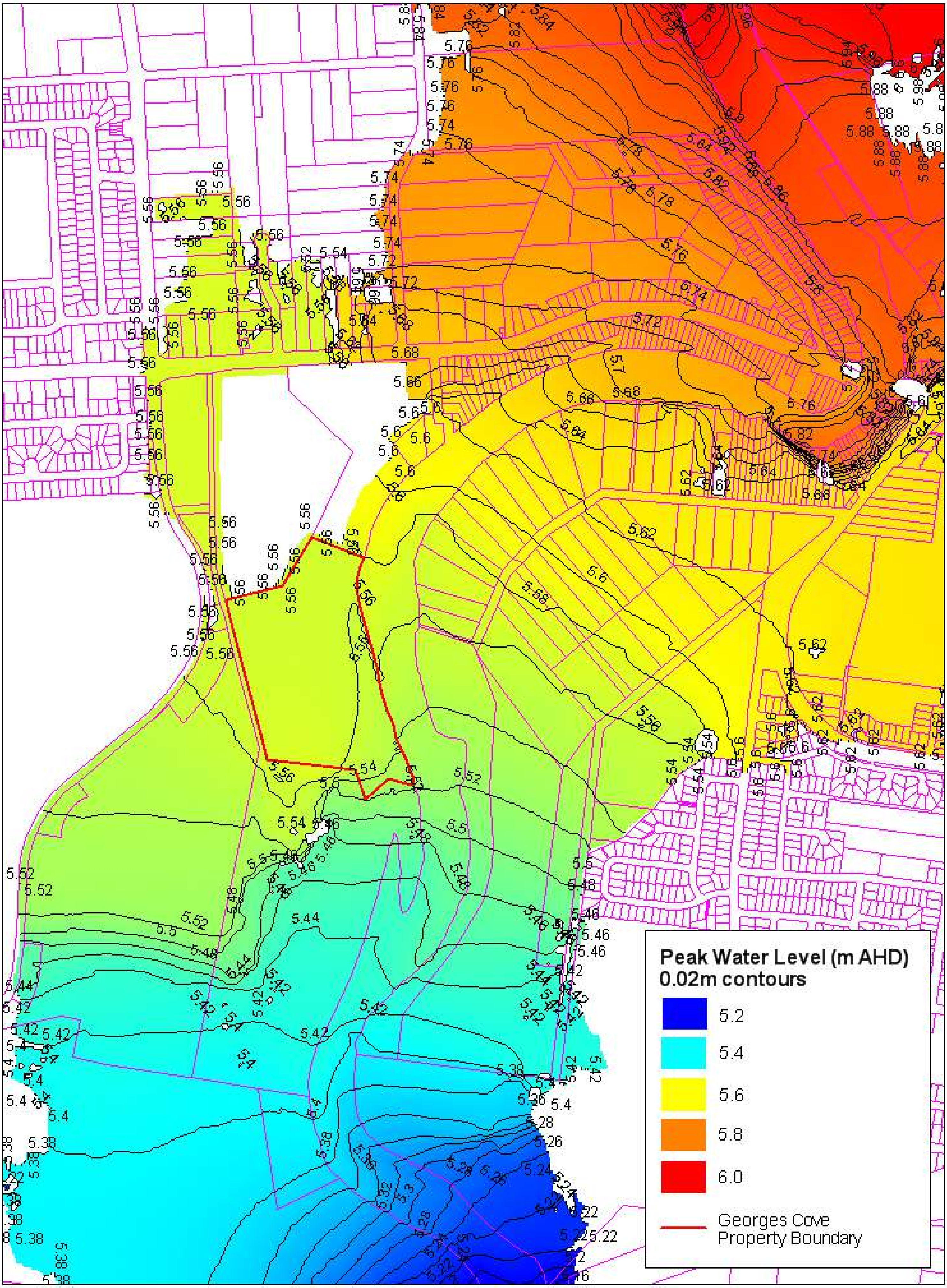

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Georges Cove
Flood Impact Assessment

Figure 3-6
100 year ARI
Peak Water Level
Pre-Development

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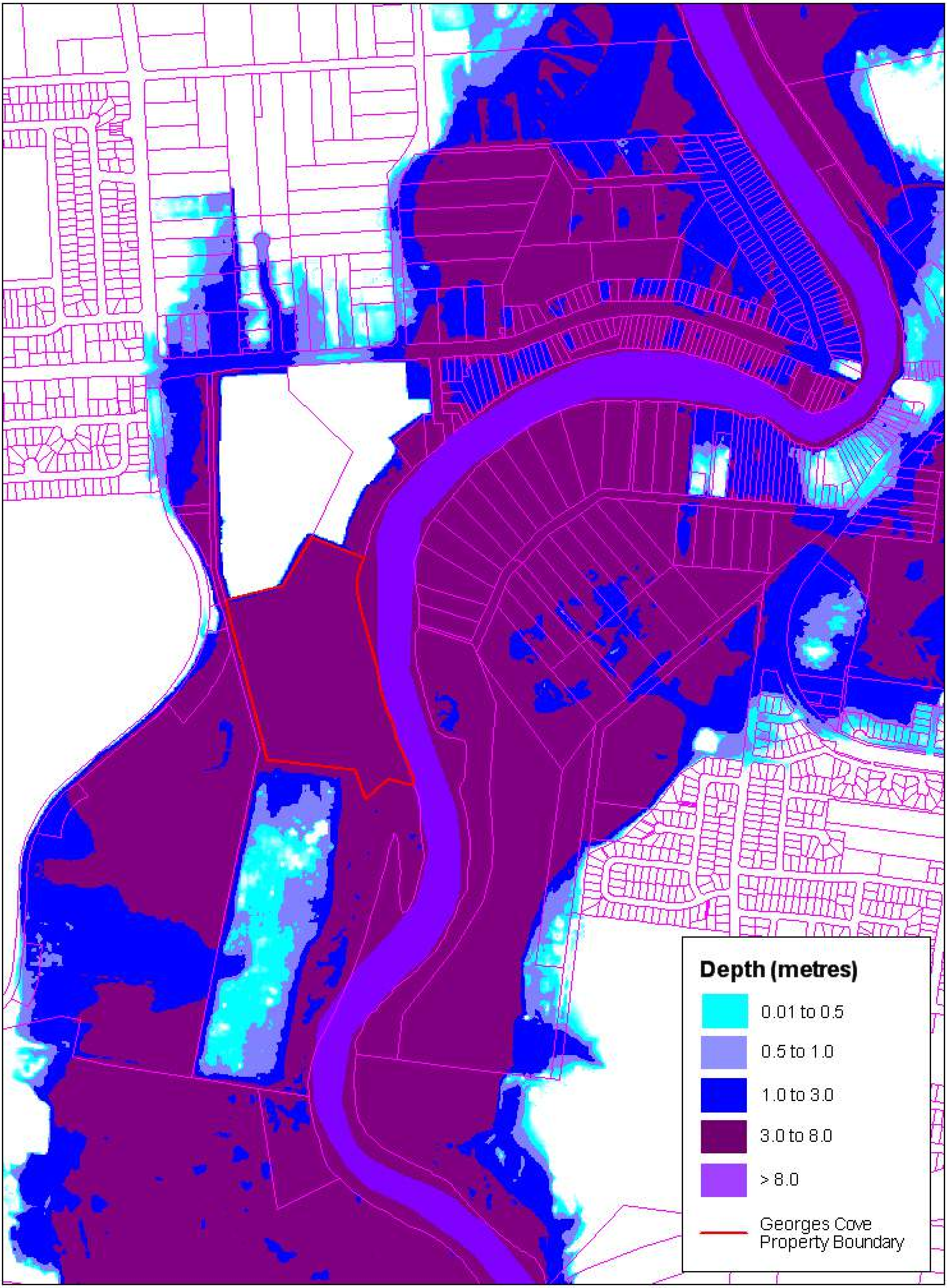

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Georges Cove
Flood Impact Assessment

Figure 3-7
100 year ARI
Peak Depth
Pre-Development

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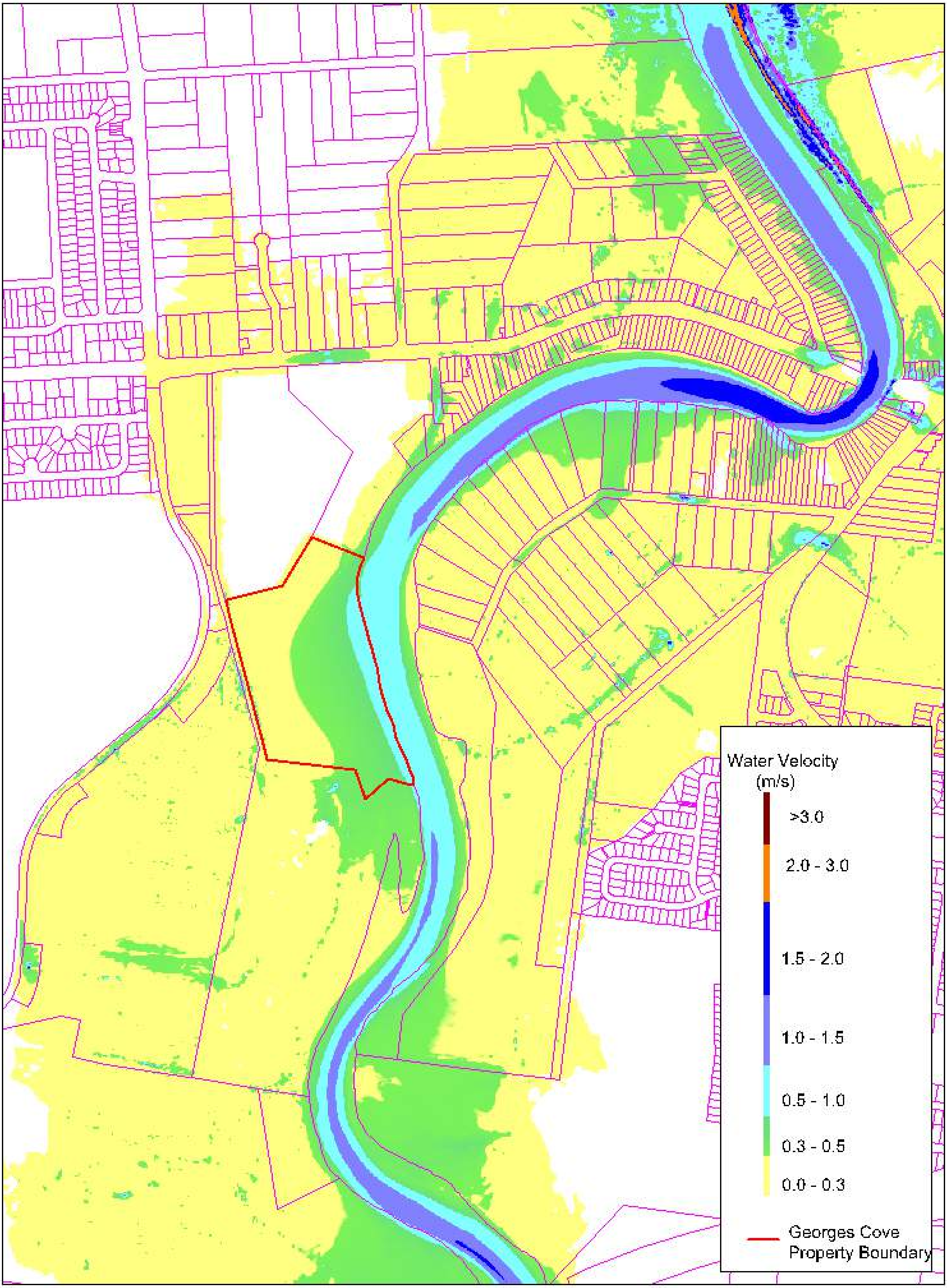

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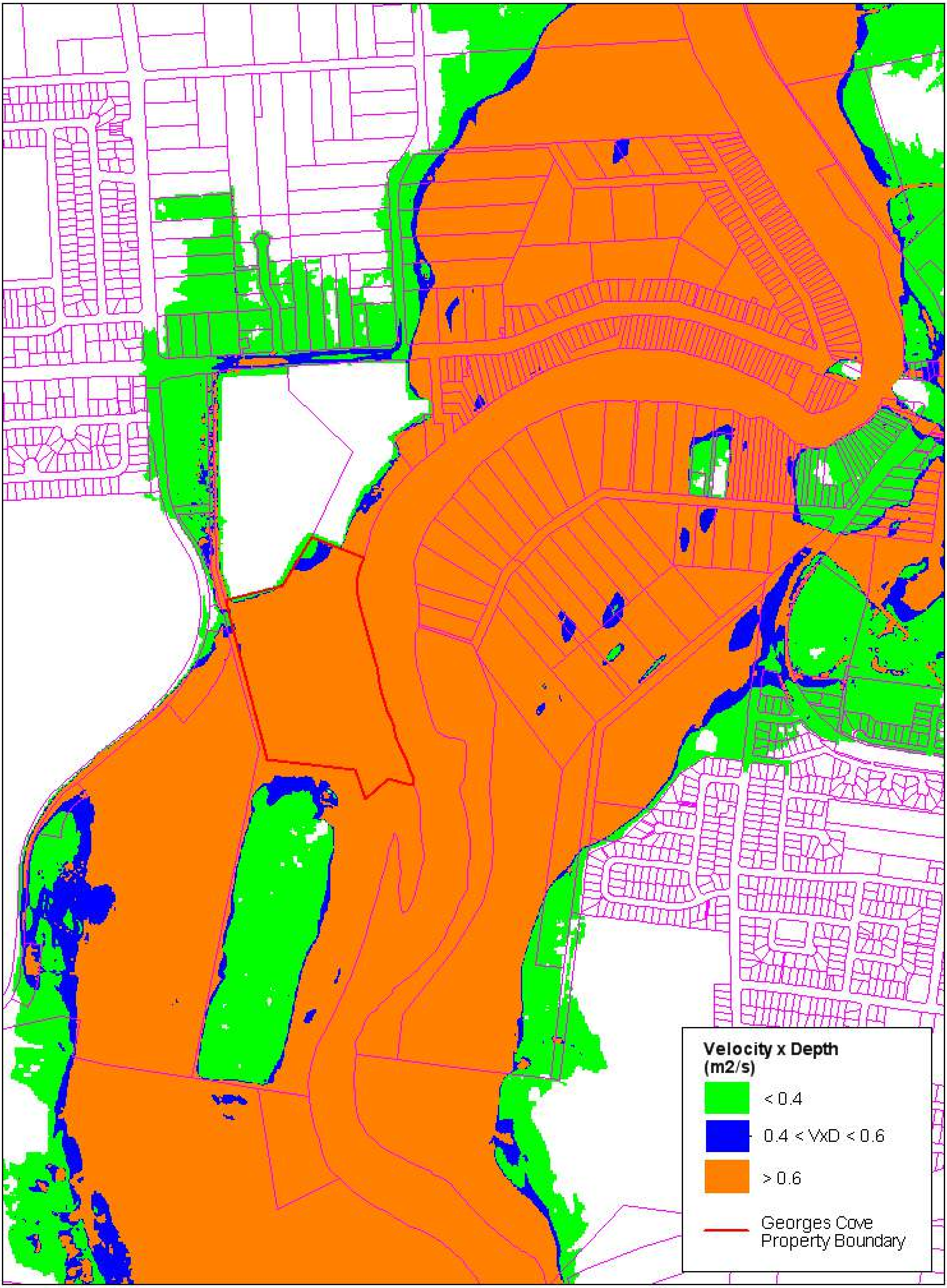
Georges Cove
Flood Impact Assessment

Figure 3-8
100 year ARI
Peak Velocity
Pre-Development

NA49913037
January 2013



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Georges Cove
Flood Impact Assessment

Figure 3-9
100 year ARI
Peak Velocity x Depth
Pre-Development

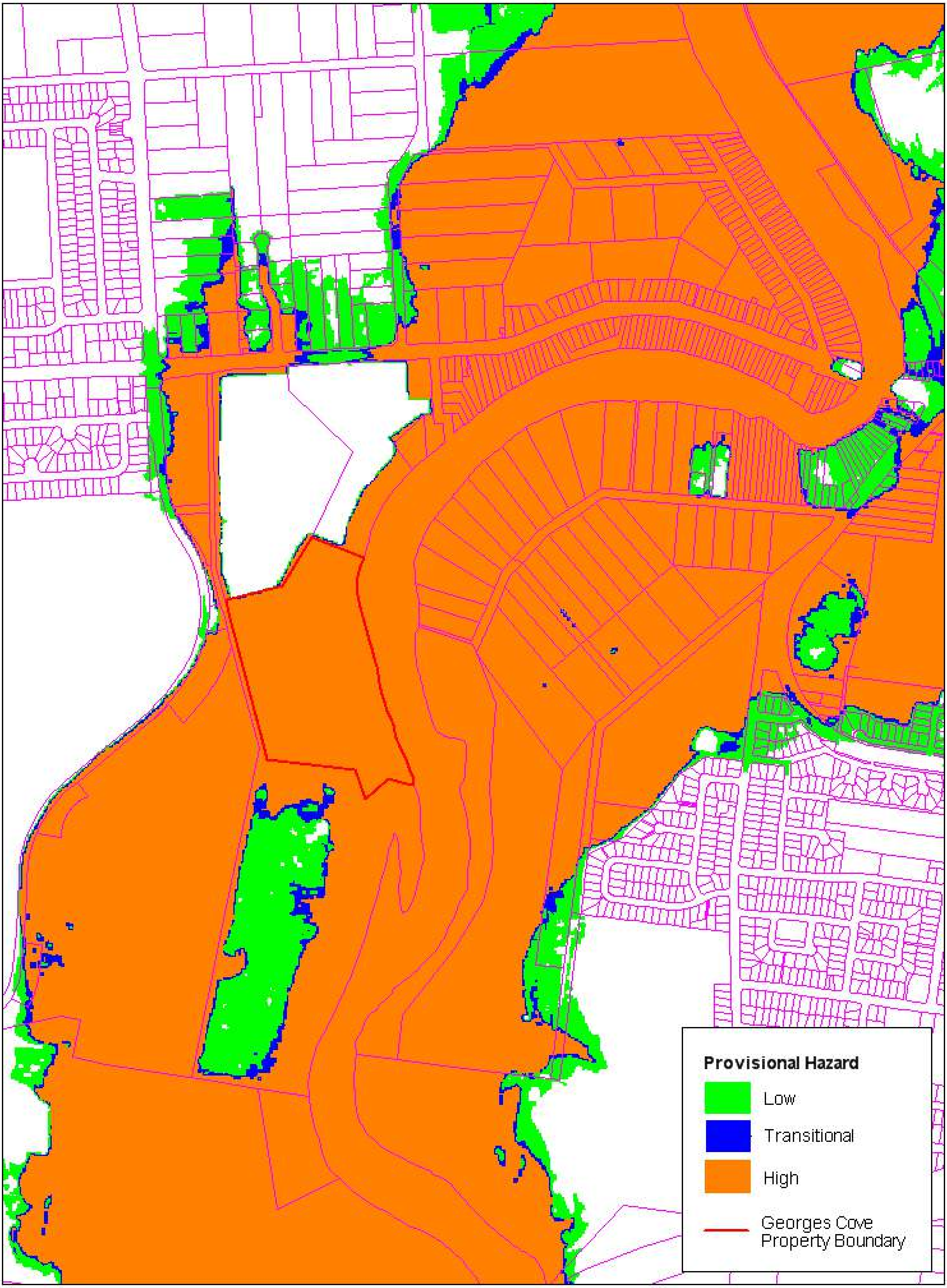
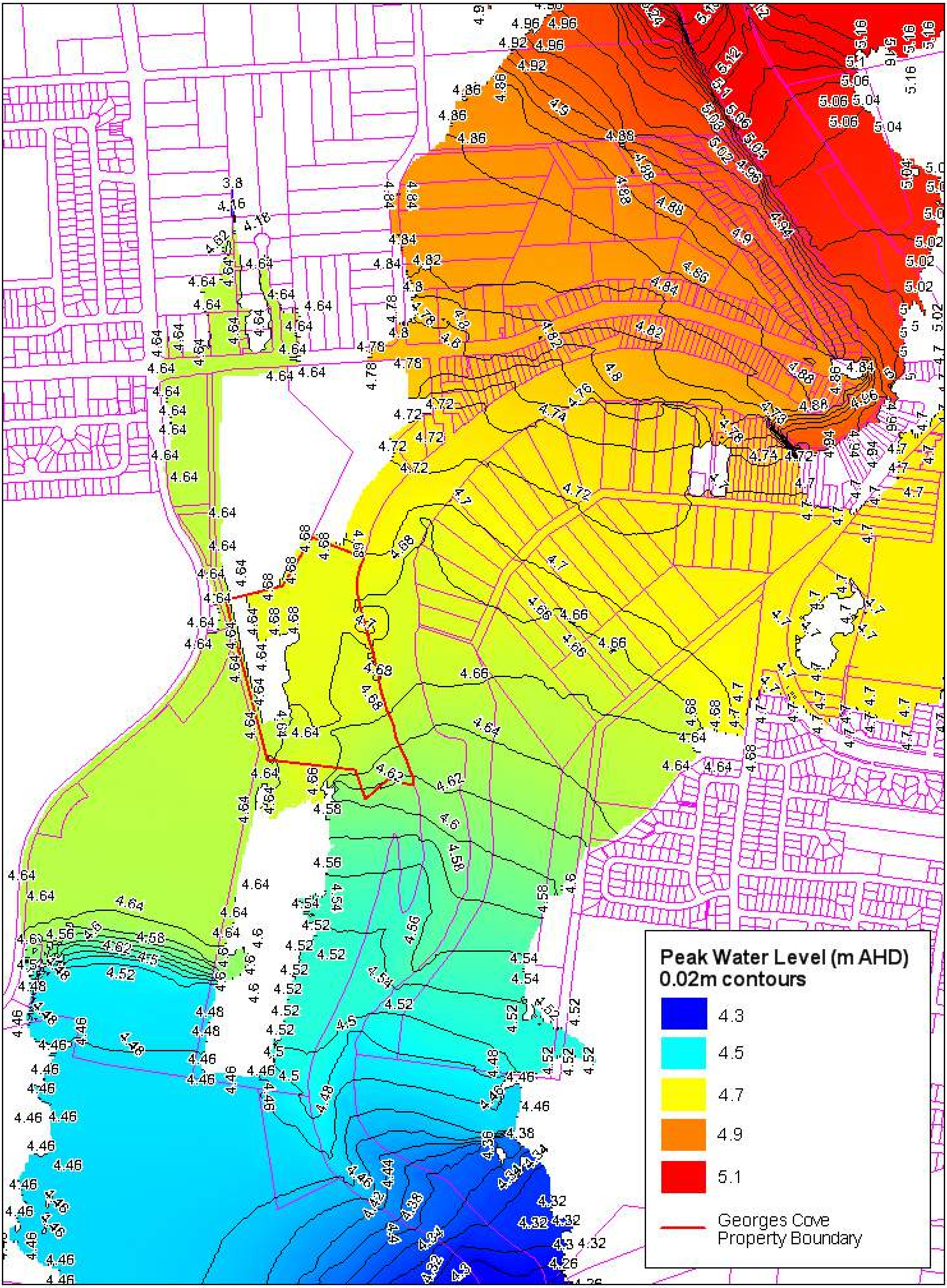
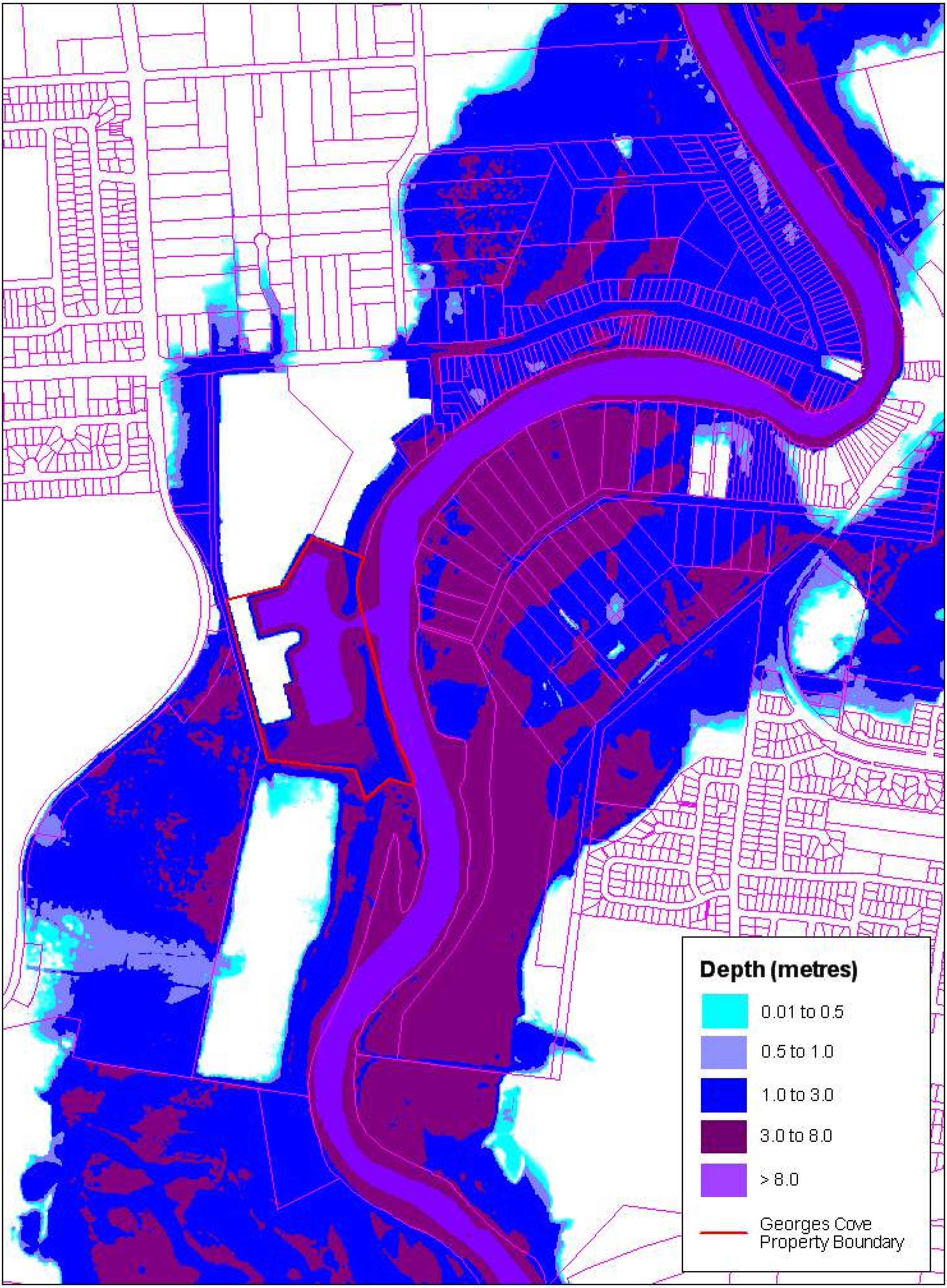


Figure 3-10
100 year ARI
Provisional Hazard
Pre-Development



Georges Cove
Flood Impact Assessment

Figure 3-11
20 year ARI
Peak Water Level
Post-development



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Georges Cove
Flood Impact Assessment

Figure 3-12
20 year ARI
Peak Depth
Post-development

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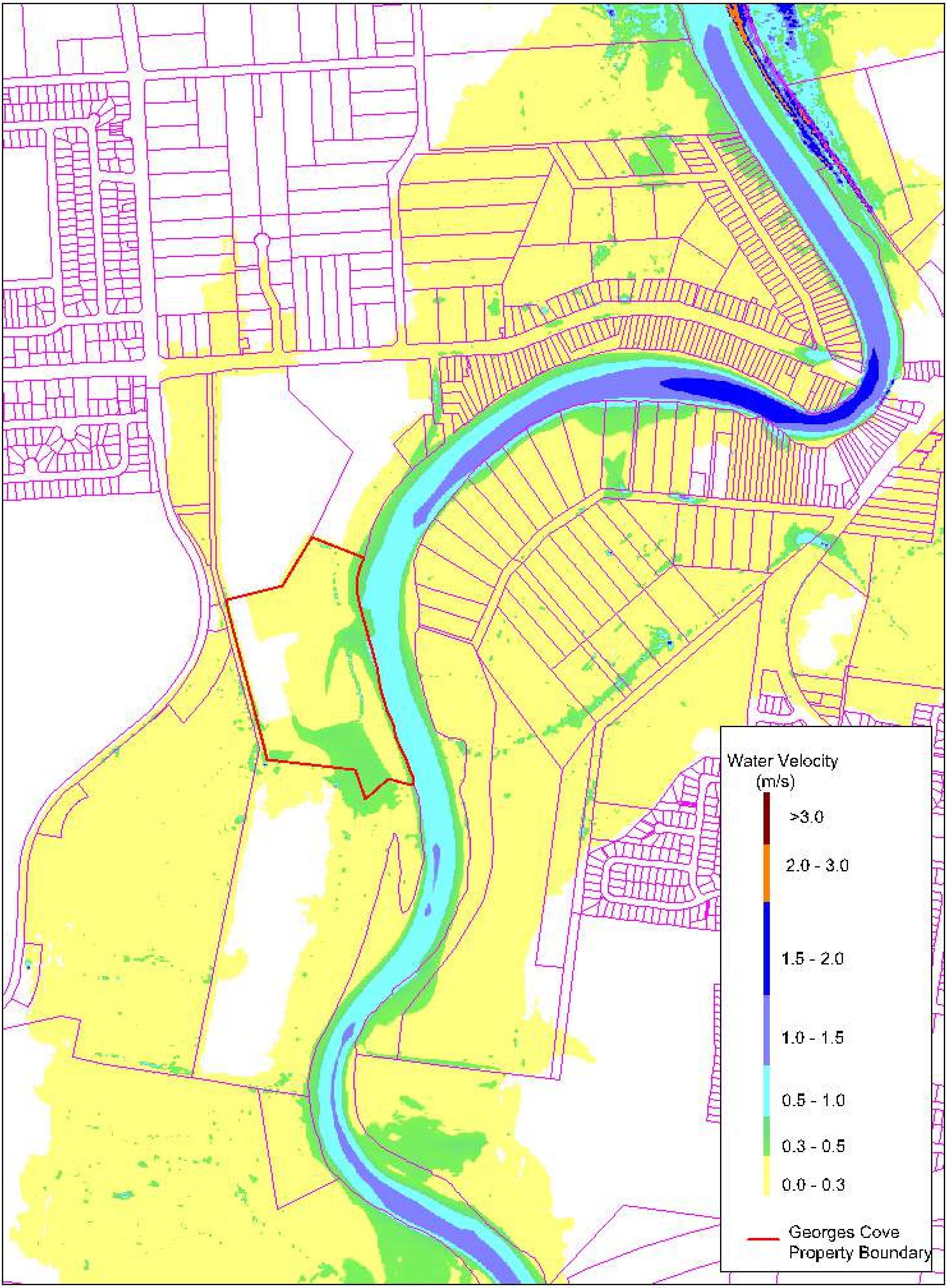

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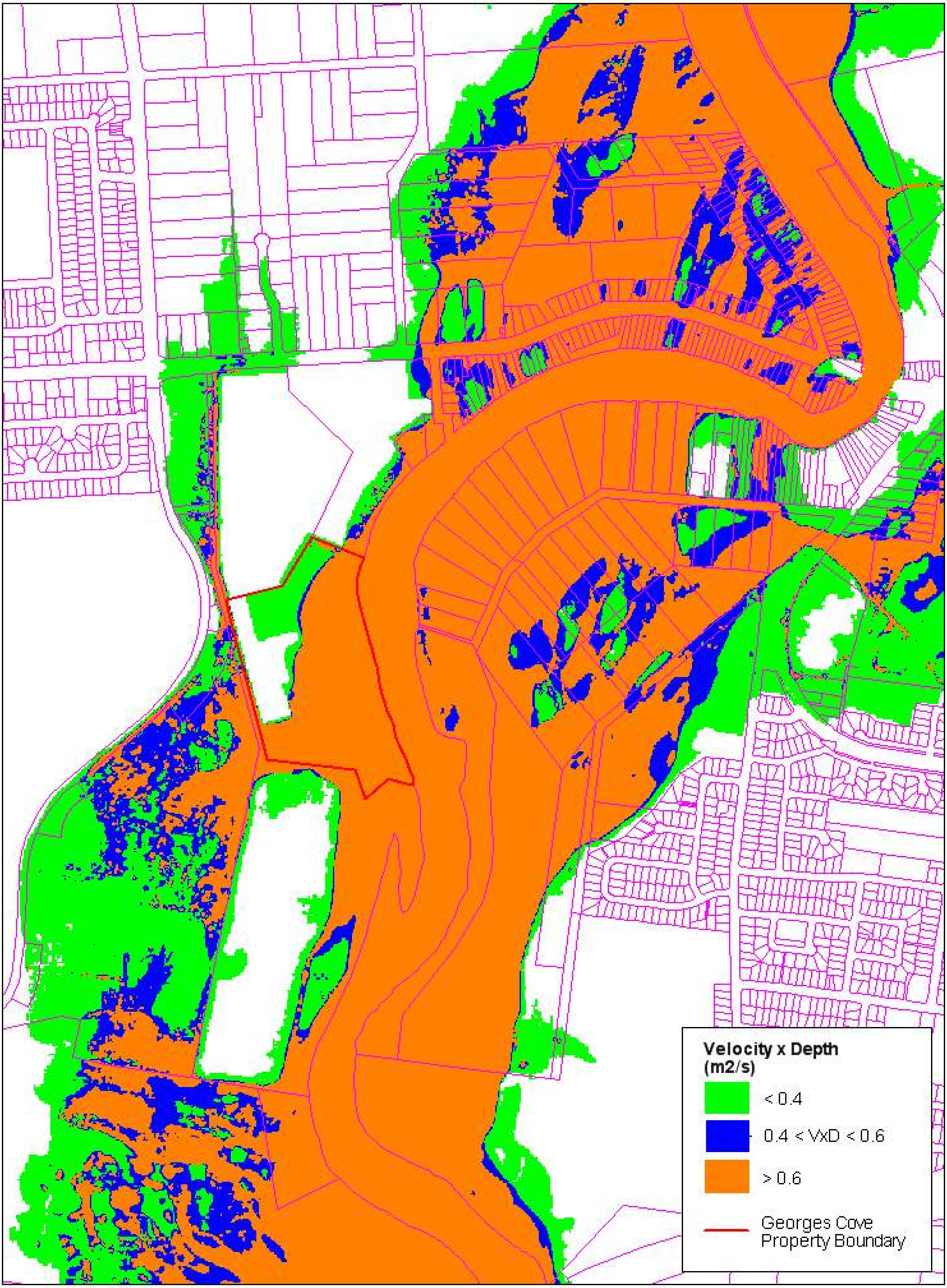
Georges Cove
Flood Impact Assessment

Figure 3-13
20 year ARI
Peak Velocity
Post-development



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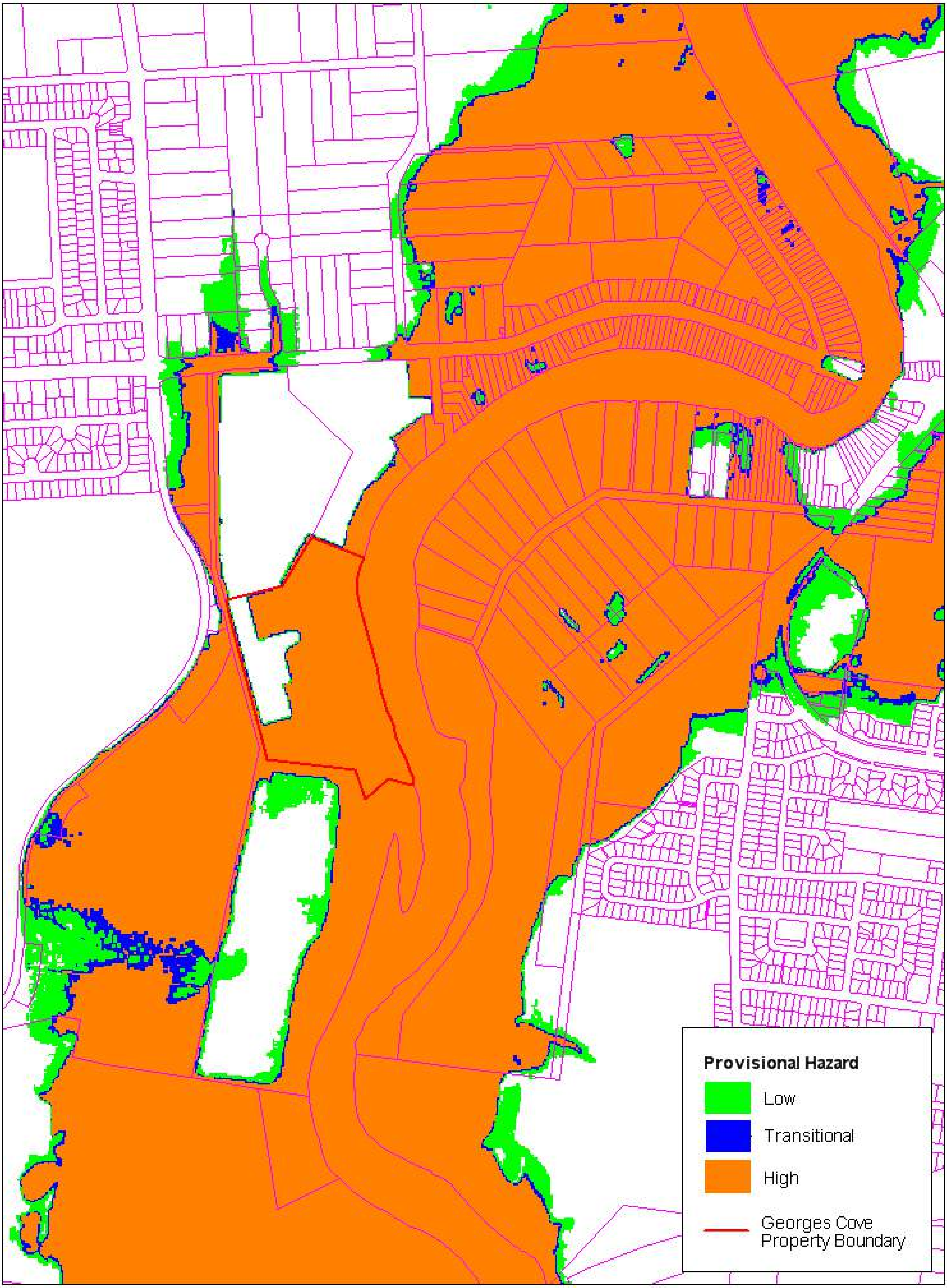
Georges Cove
Flood Impact Assessment

Figure 3-14
20 year ARI
Peak Velocity x Depth
Post-development



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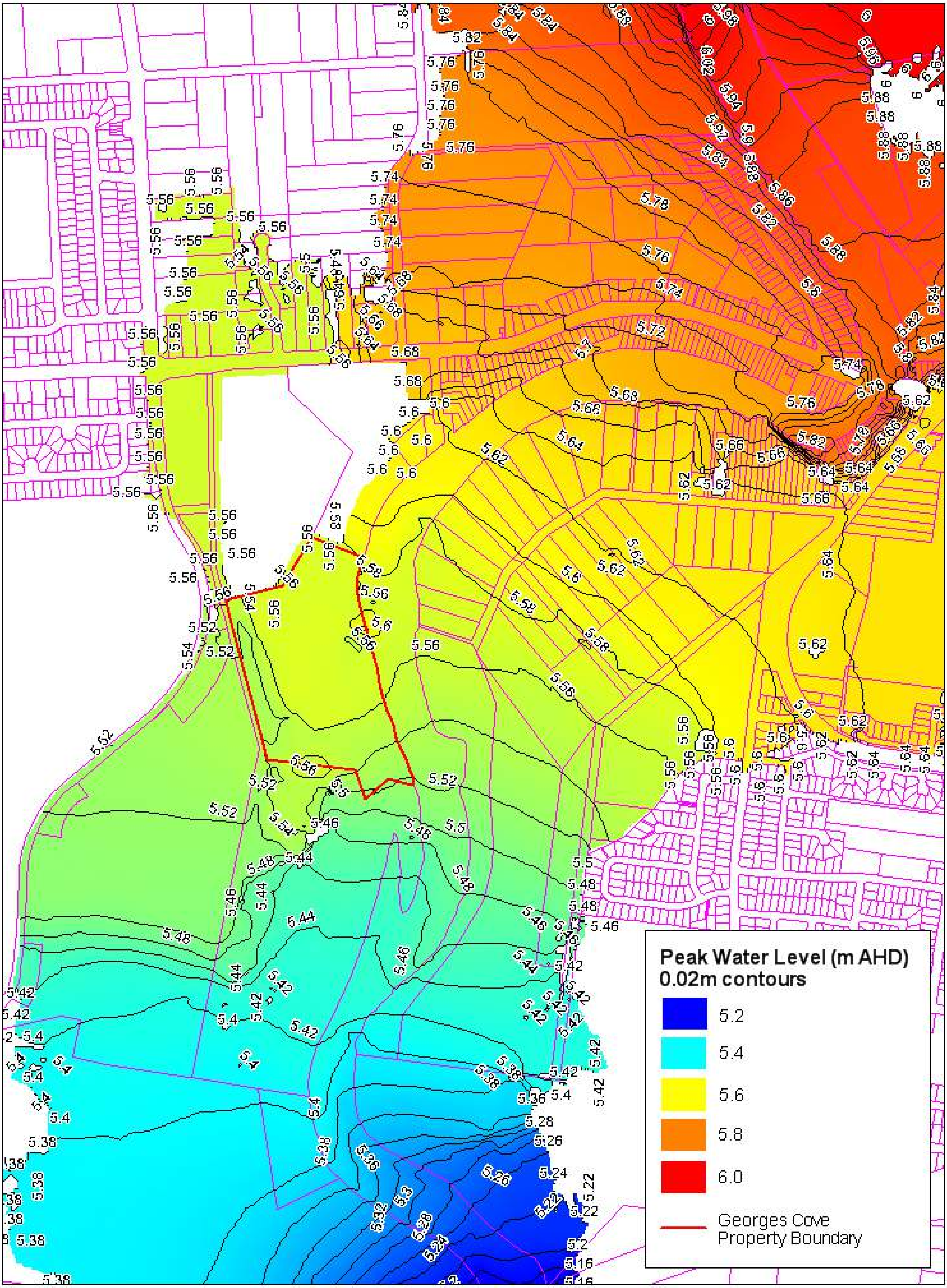
Georges Cove
Flood Impact Assessment

Figure 3-15
20 year ARI
Provisional Hazard
Post-development



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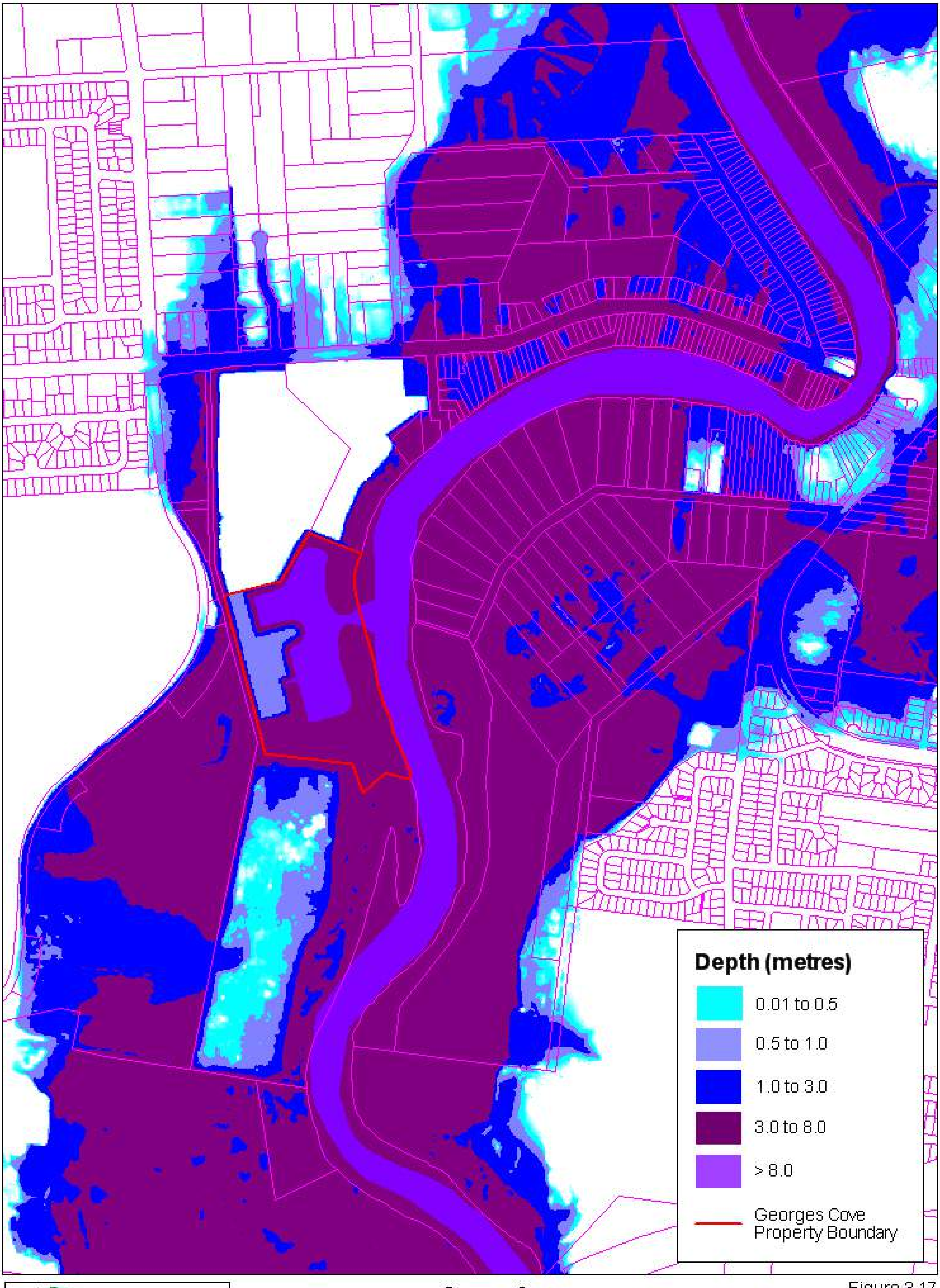
Georges Cove
Flood Impact Assessment

Figure 3-16
100 year ARI
Peak Water Level
Post-development



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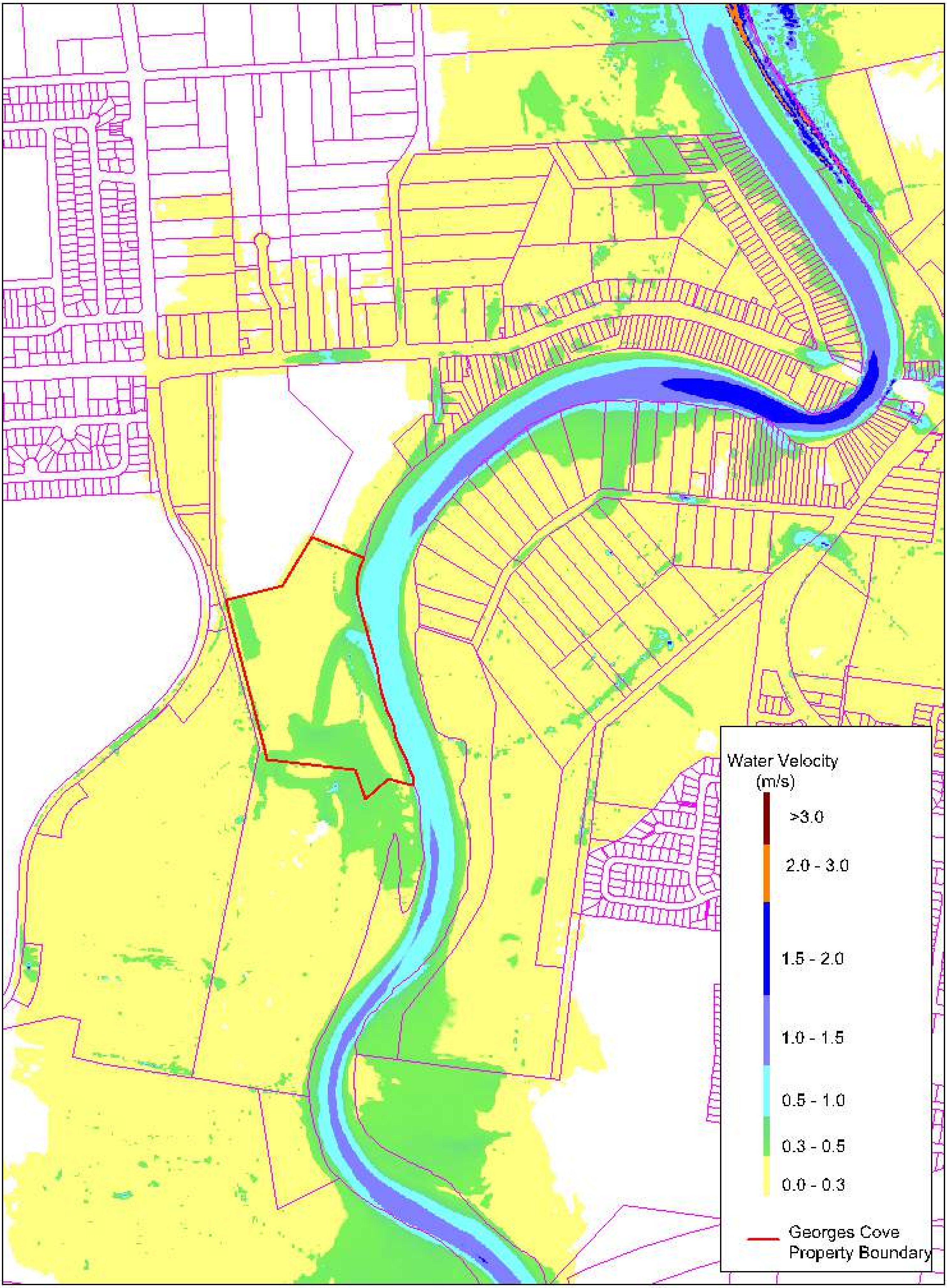
Georges Cove
Flood Impact Assessment

Figure 3-17
100 year ARI
Peak Depth
Post-development



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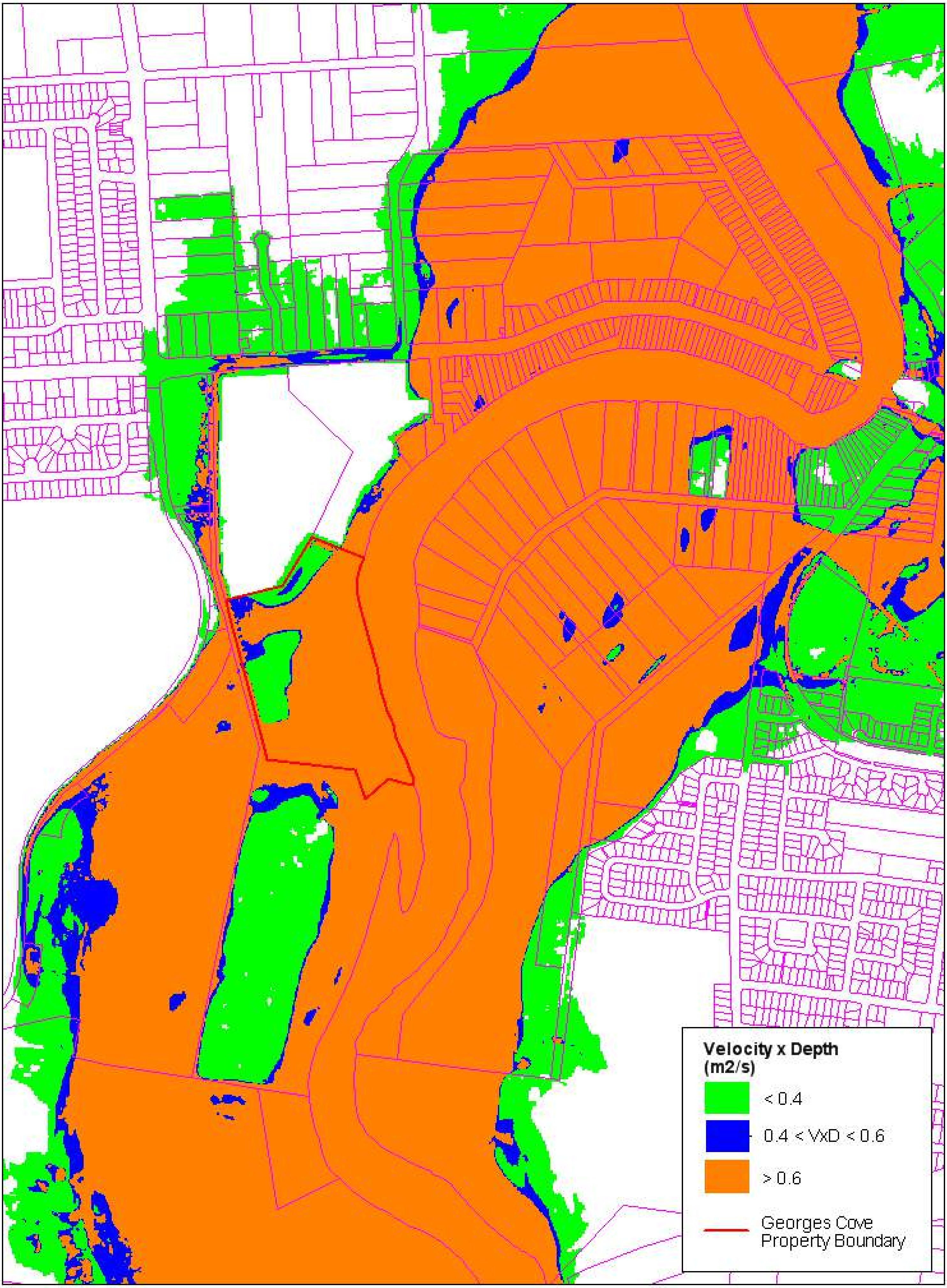
Georges Cove
Flood Impact Assessment

Figure 3-18
100 year ARI
Peak Velocity
Post-development



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Georges Cove
Flood Impact Assessment

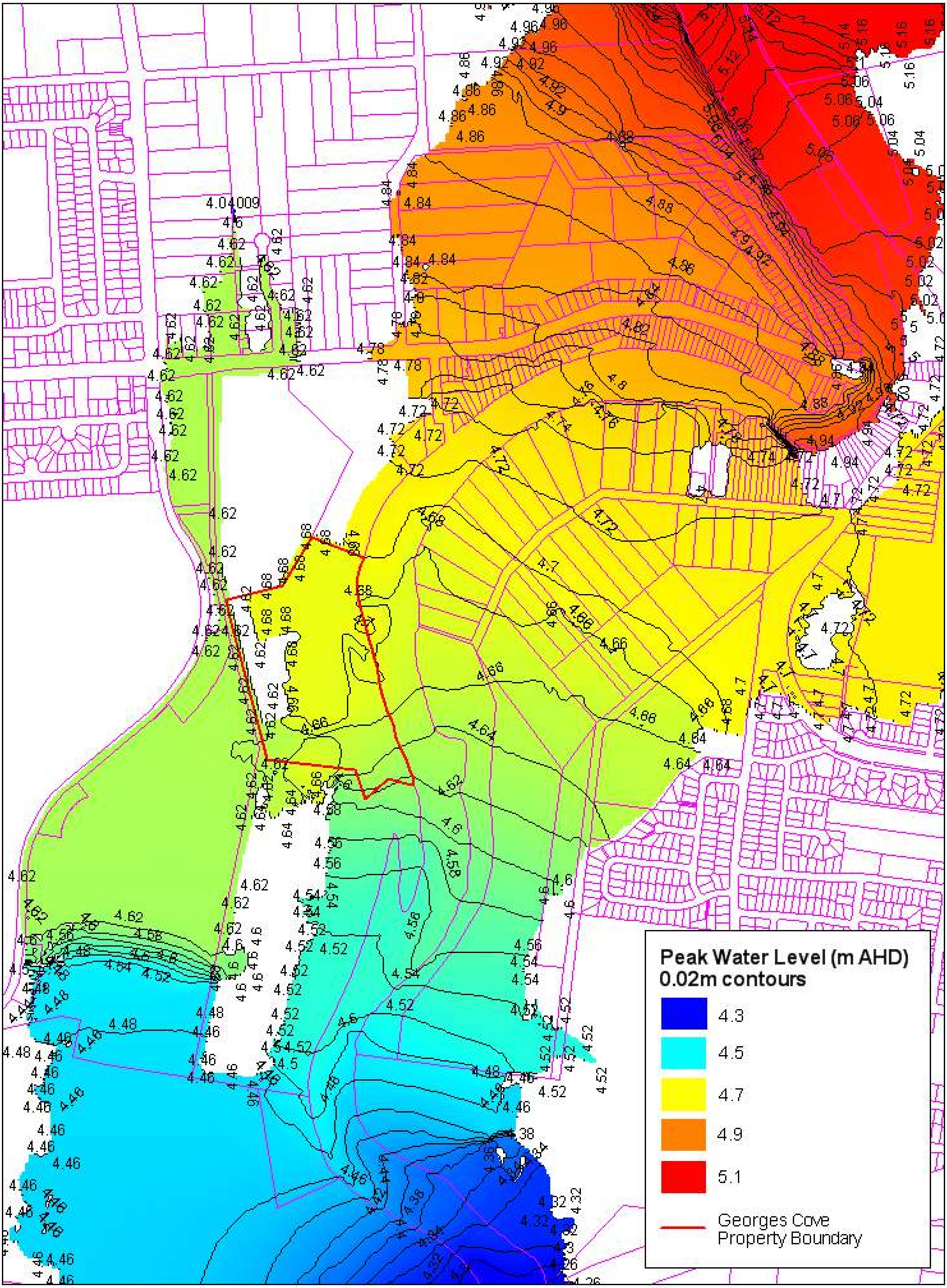
Figure 3-19
100 year ARI
Peak Velocity x Depth
Post-development

Cardno

Georges Cove
Flood Impact Assessment

Figure 3-20
100 year ARI
Provisional Hazard
Post-development

NA49913037
January 2013



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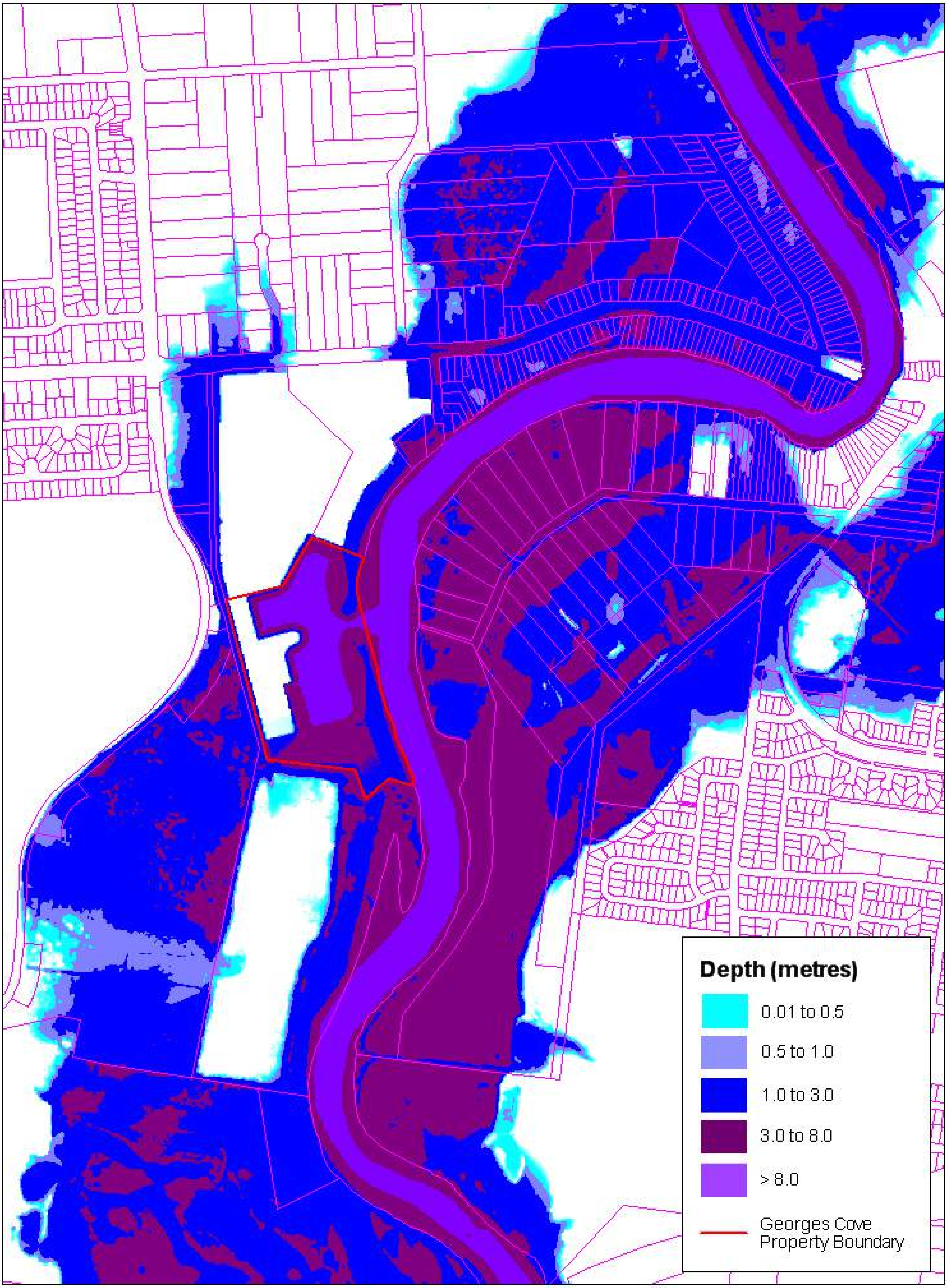
January 2013

Georges Cove
Flood Impact Assessment

Figure 3-21
20 year ARI
Peak Water Level
Post-development-Blockage



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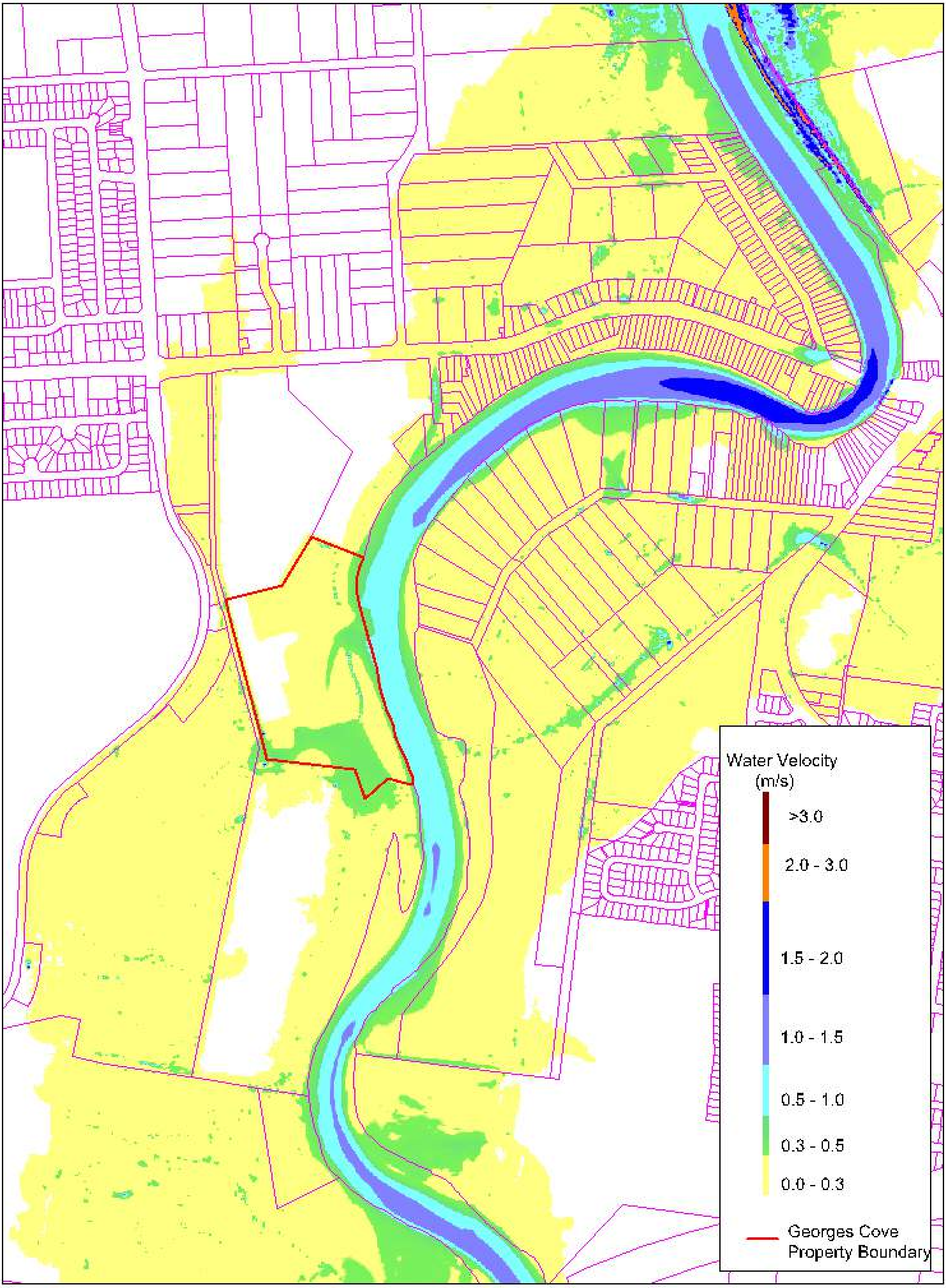
January 2013

Georges Cove
Flood Impact Assessment

Figure 3-22
20 year ARI
Peak Depth
Post-development-Blockage



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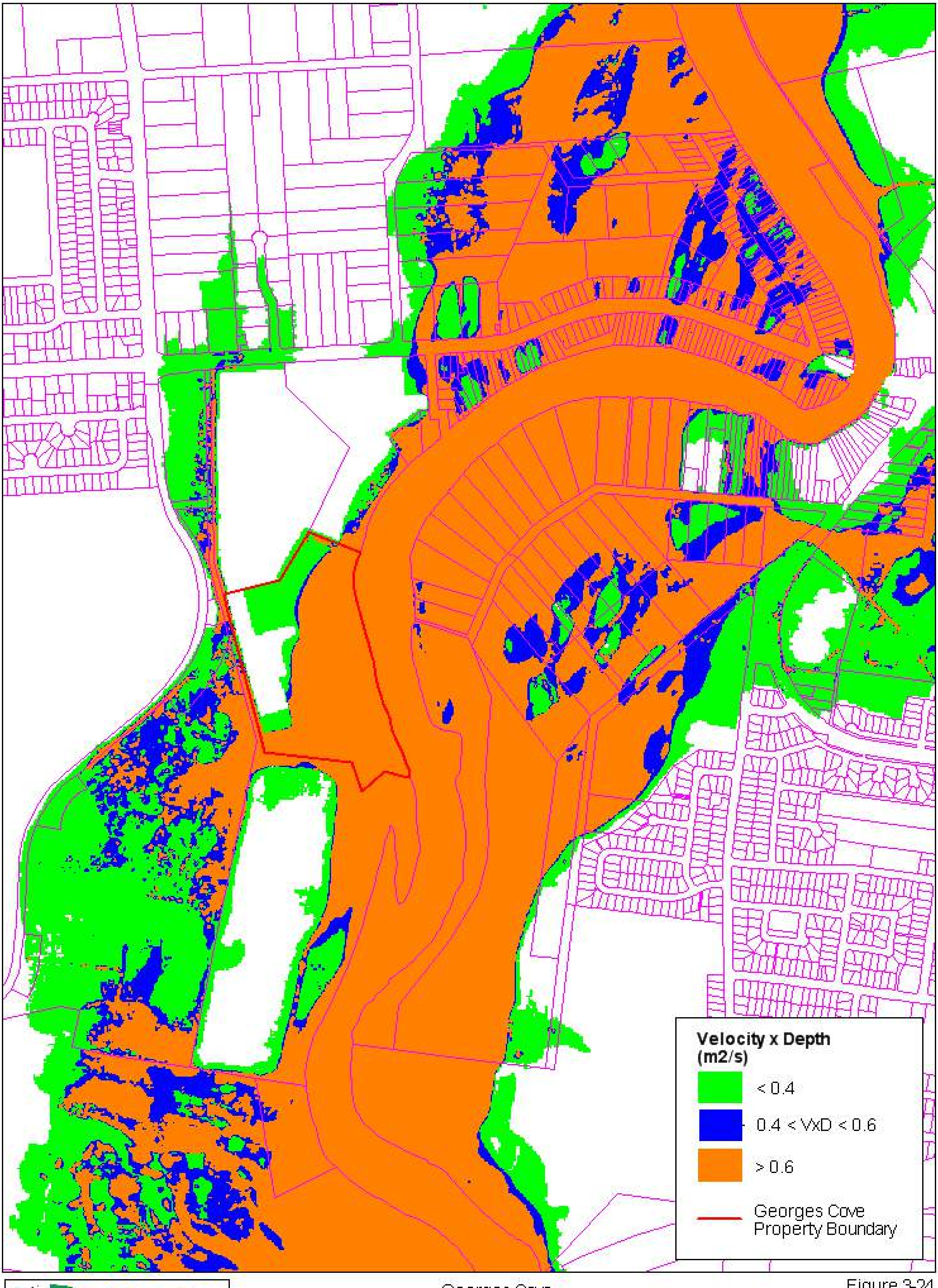
Georges Cove
Flood Impact Assessment

Figure 3-23
20 year ARI
Peak Velocity
Post-development-Blockage

NA49913037
January 2013



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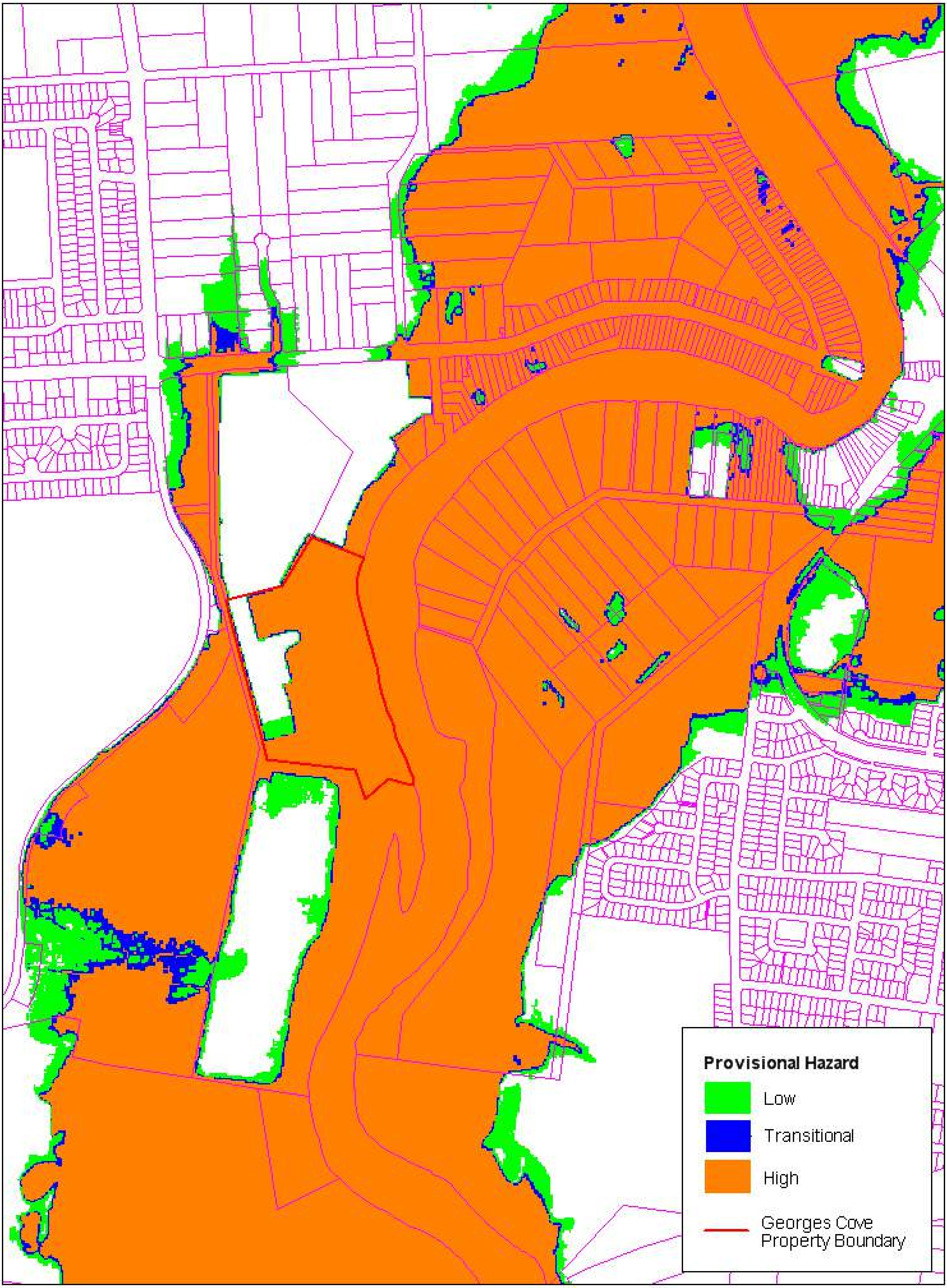
January 2013

Georges Cove
Flood Impact Assessment

Figure 3-24
20 year ARI
Peak Velocity x Depth
Post-development-Blockage



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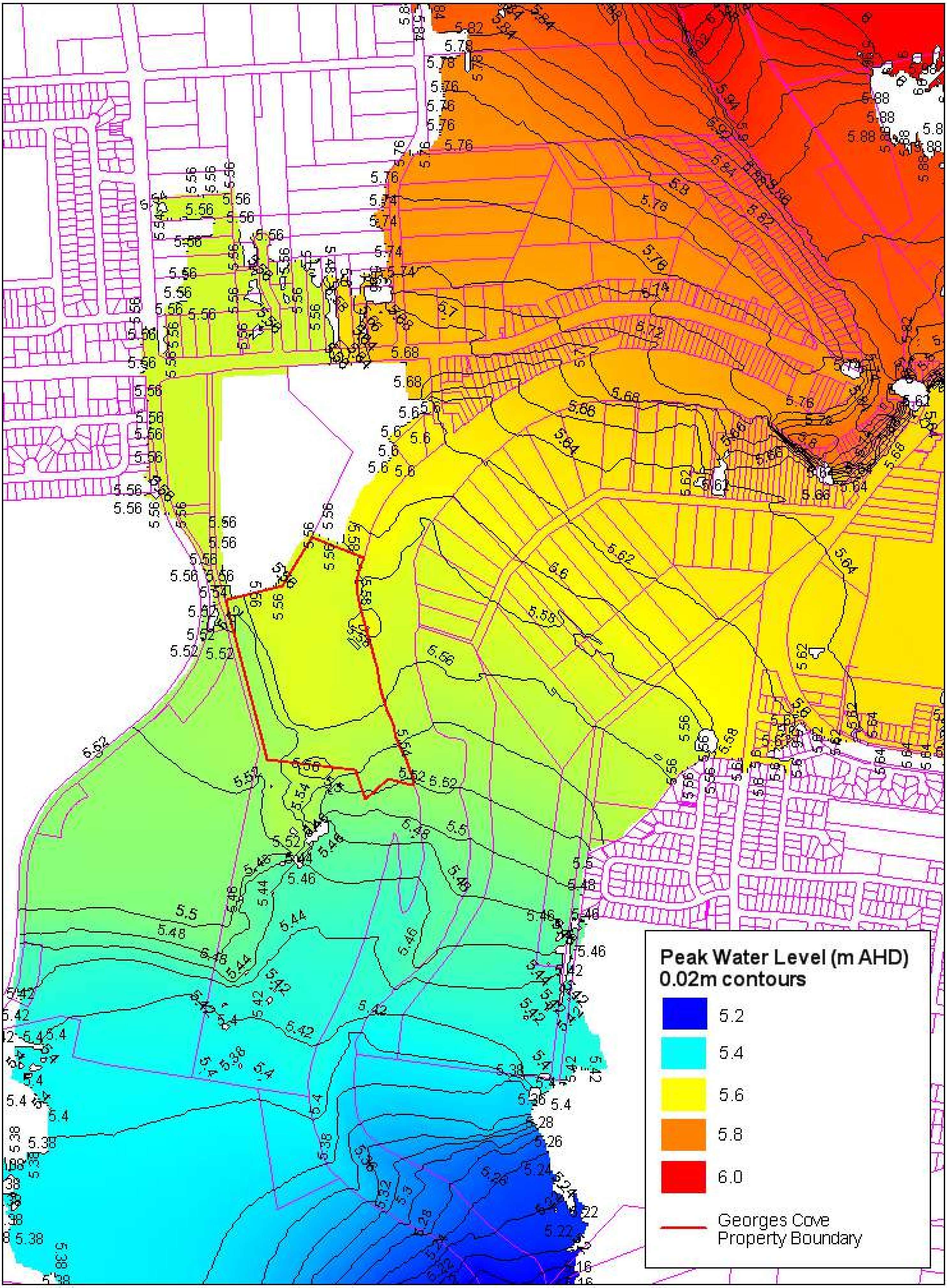
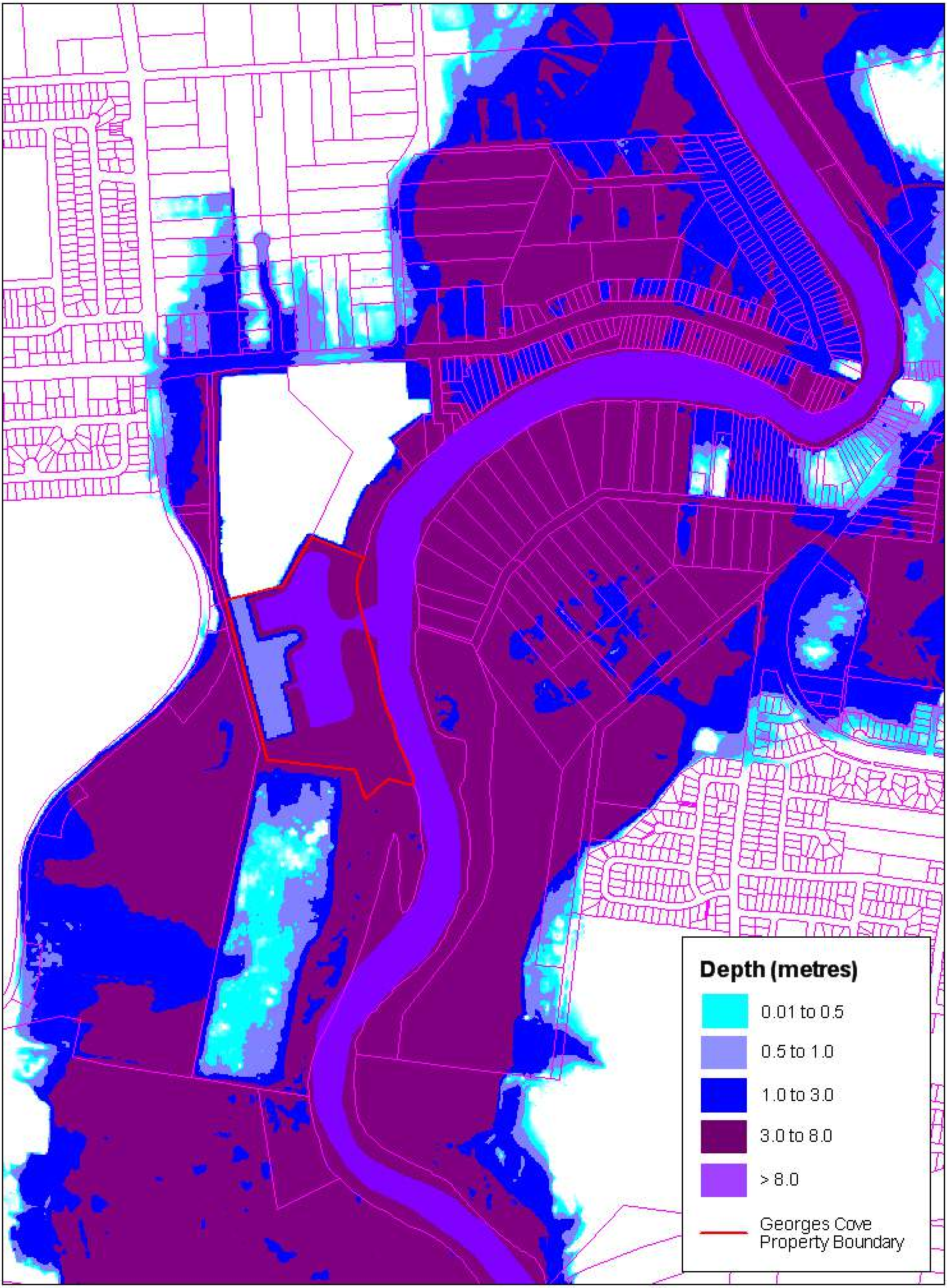
NA49913037
January 2013

Georges Cove
Flood Impact Assessment

Figure 3-25
20 year ARI
Provisional Hazard
Post-development-Blockage



Peak Water Level (m AHD)
0.02m contours
5.2
5.4
5.6
5.8
6.0
Georges Cove
Property Boundary



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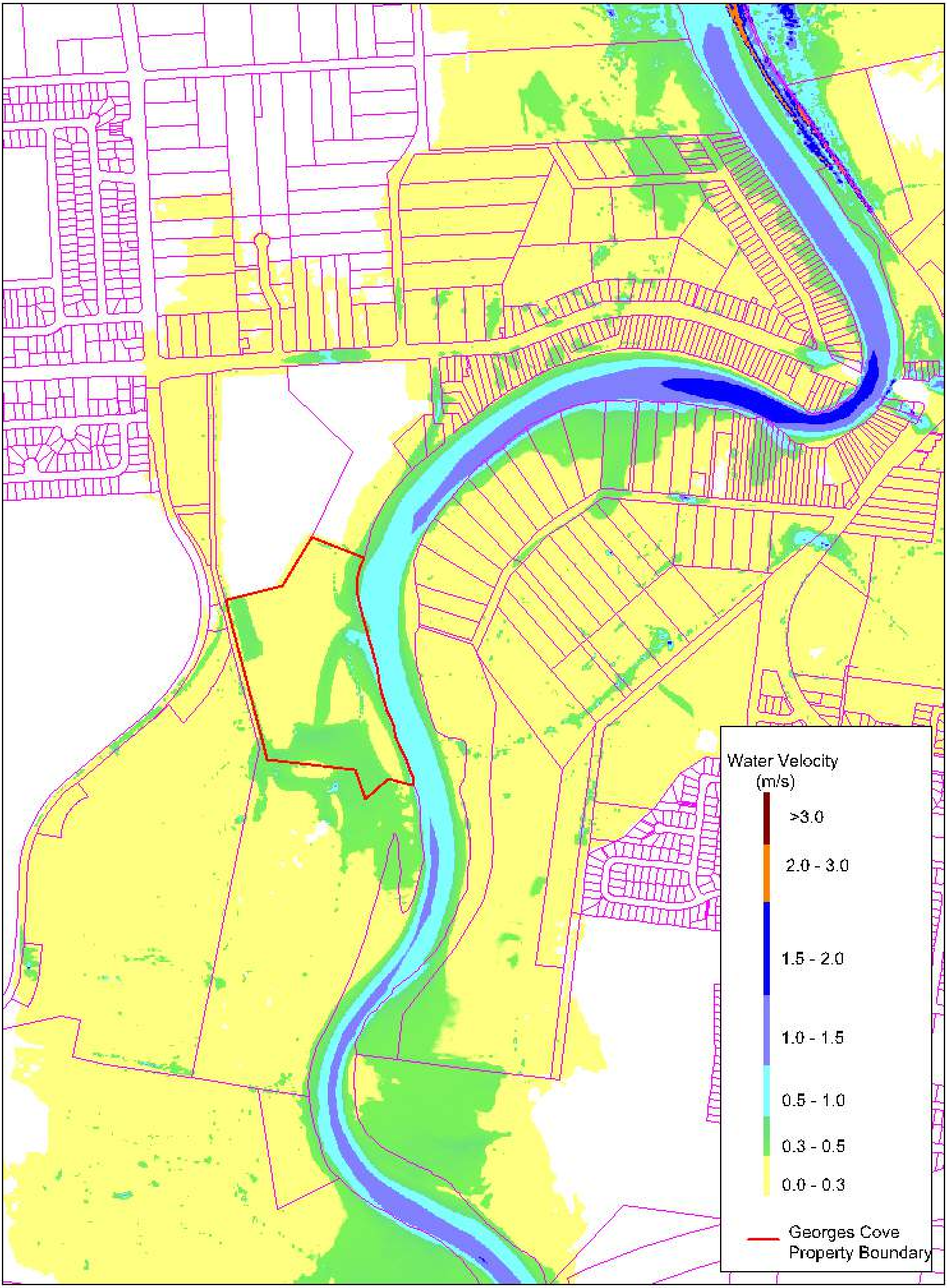
Georges Cove
Flood Impact Assessment

Figure 3-27
100 year ARI
Peak Depth
Post-development-Blockage

NA49913037
January 2013



Cardno

Georges Cove
Flood Impact Assessment

Figure 3-28
100 year ARI
Peak Velocity
Post-development-Blockage

NA49913037
January 2013

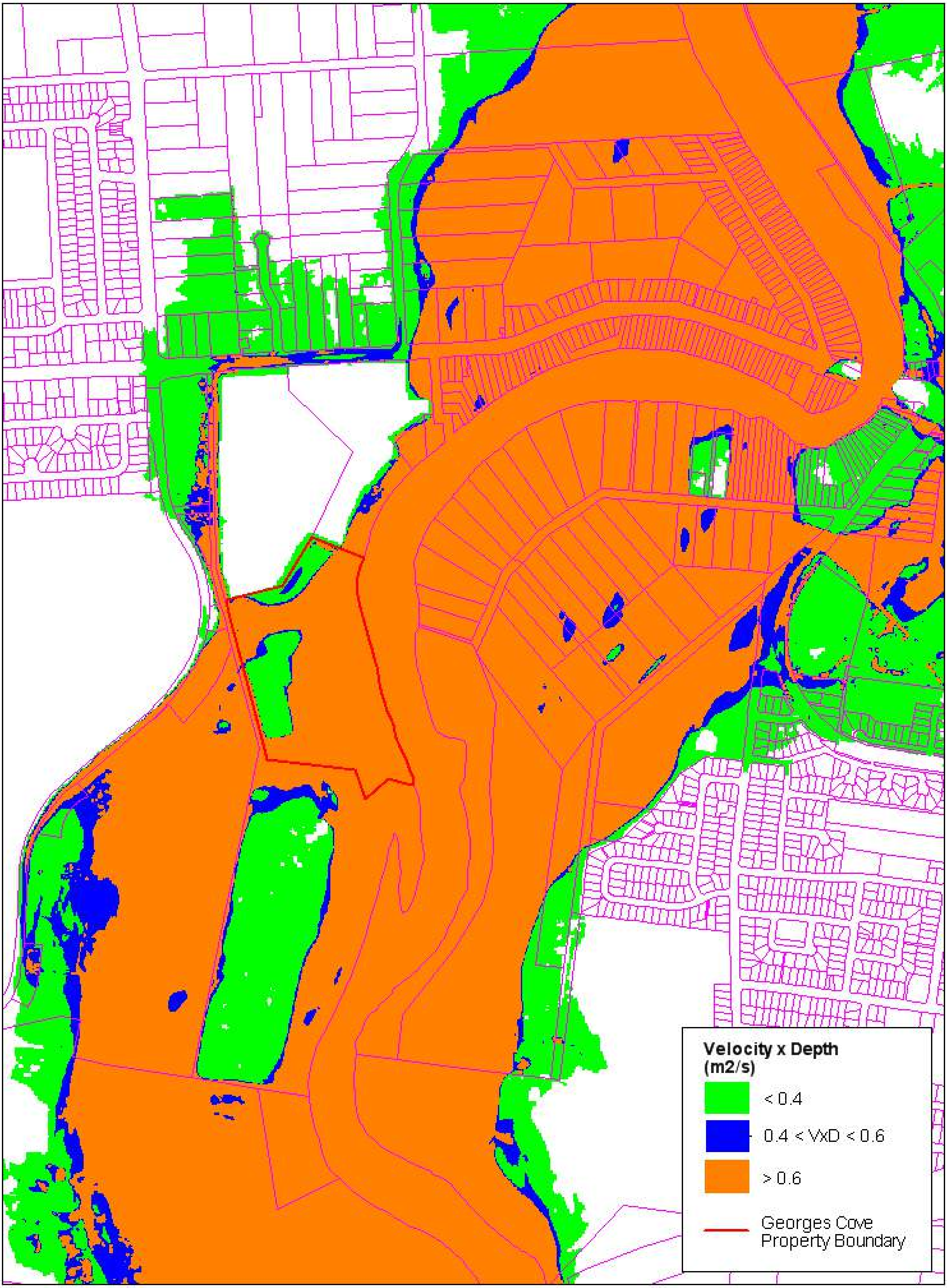


Figure 3-29
100 year ARI
Peak Velocity x Depth
Post-development-Blockage

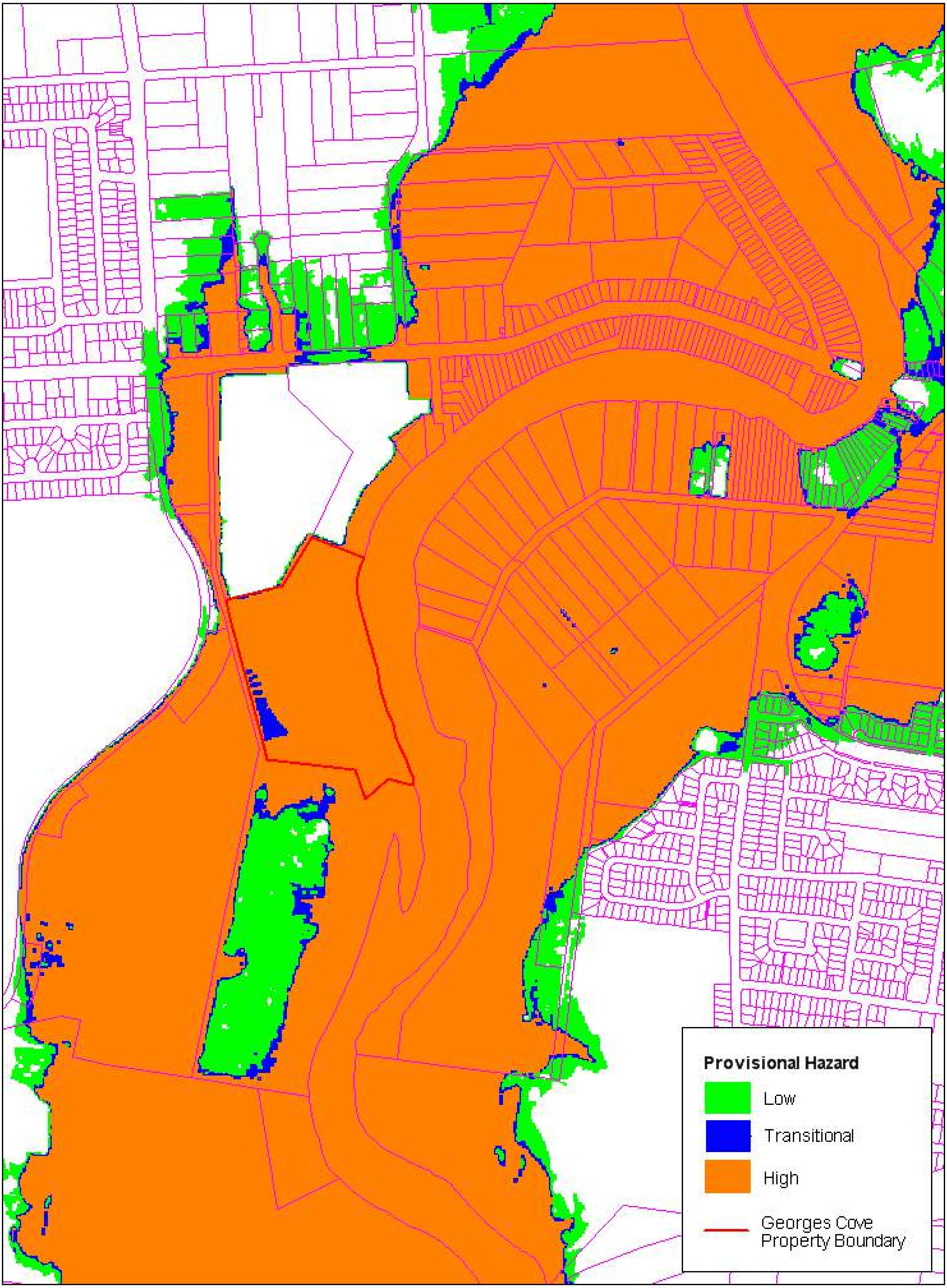
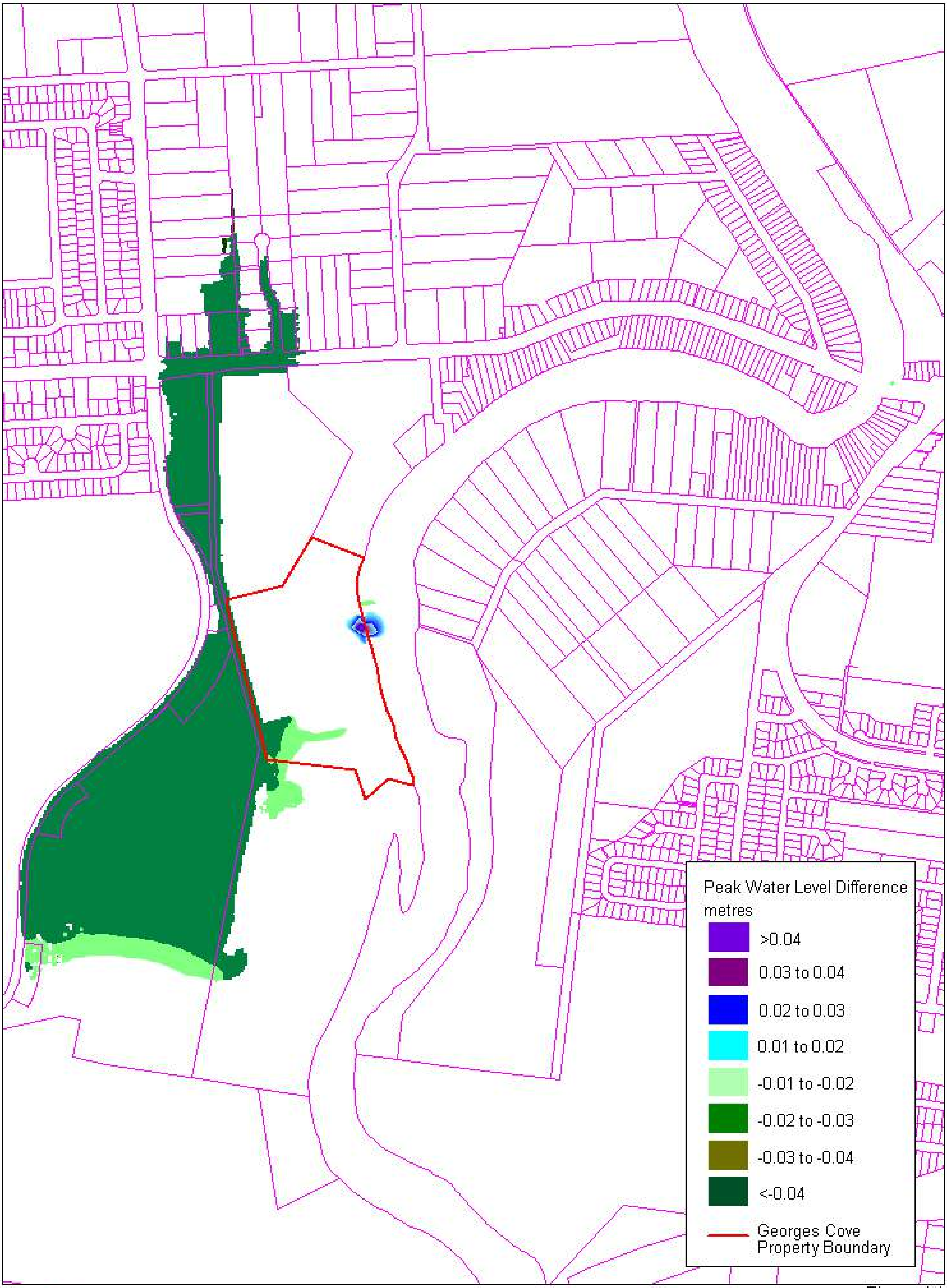


Figure 3-30
100 year ARI
Provisional Hazard
Post-development-Blockage



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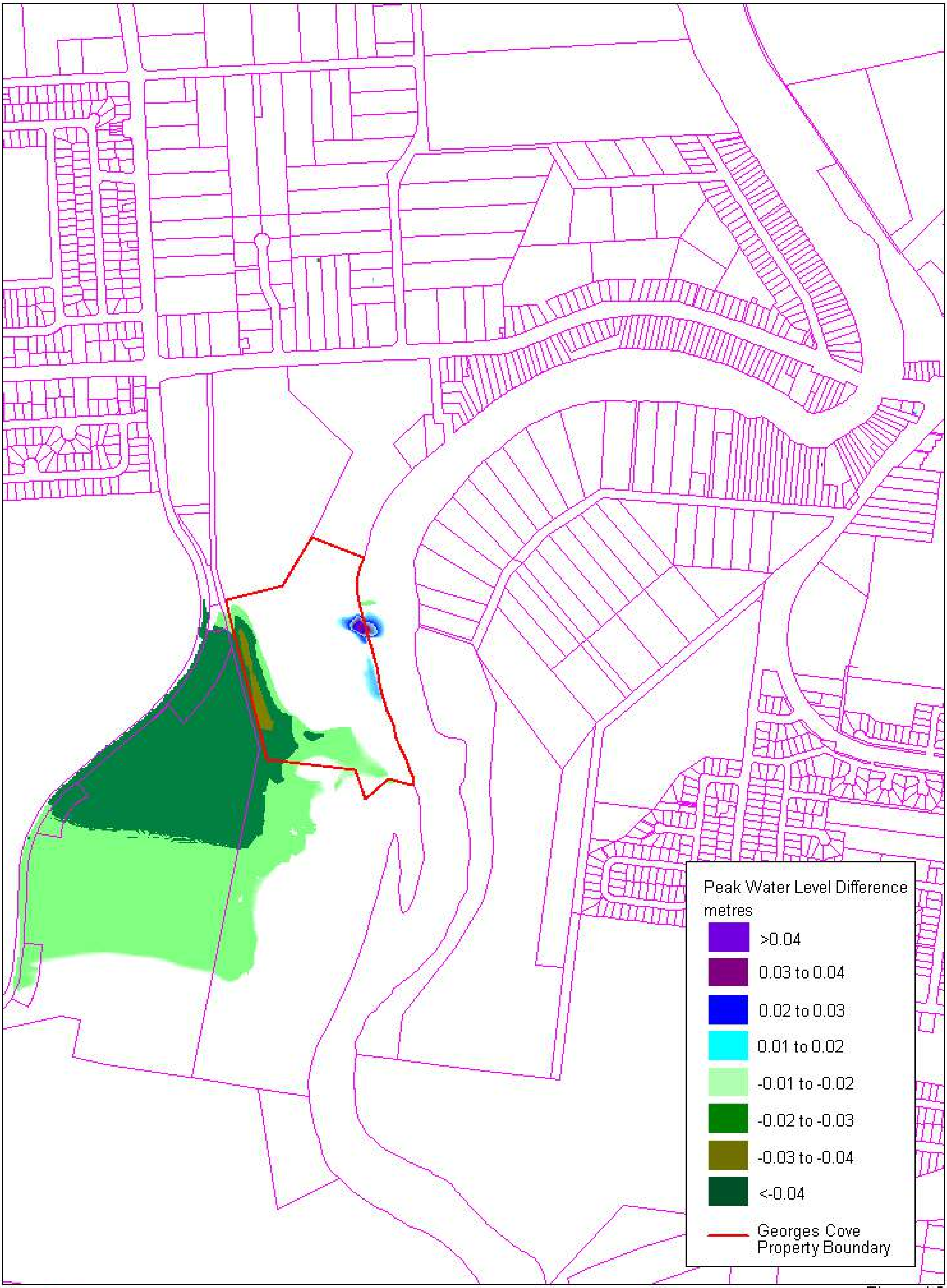
Georges Cove
Flood Impact Assessment

Figure 4-1
20 year ARI
Peak Water Level
Post-development Less Predevelopment



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Georges Cove
Flood Impact Assessment

Figure 4-2
100 year ARI
Peak Water Level
Post-development Less Predevelopment

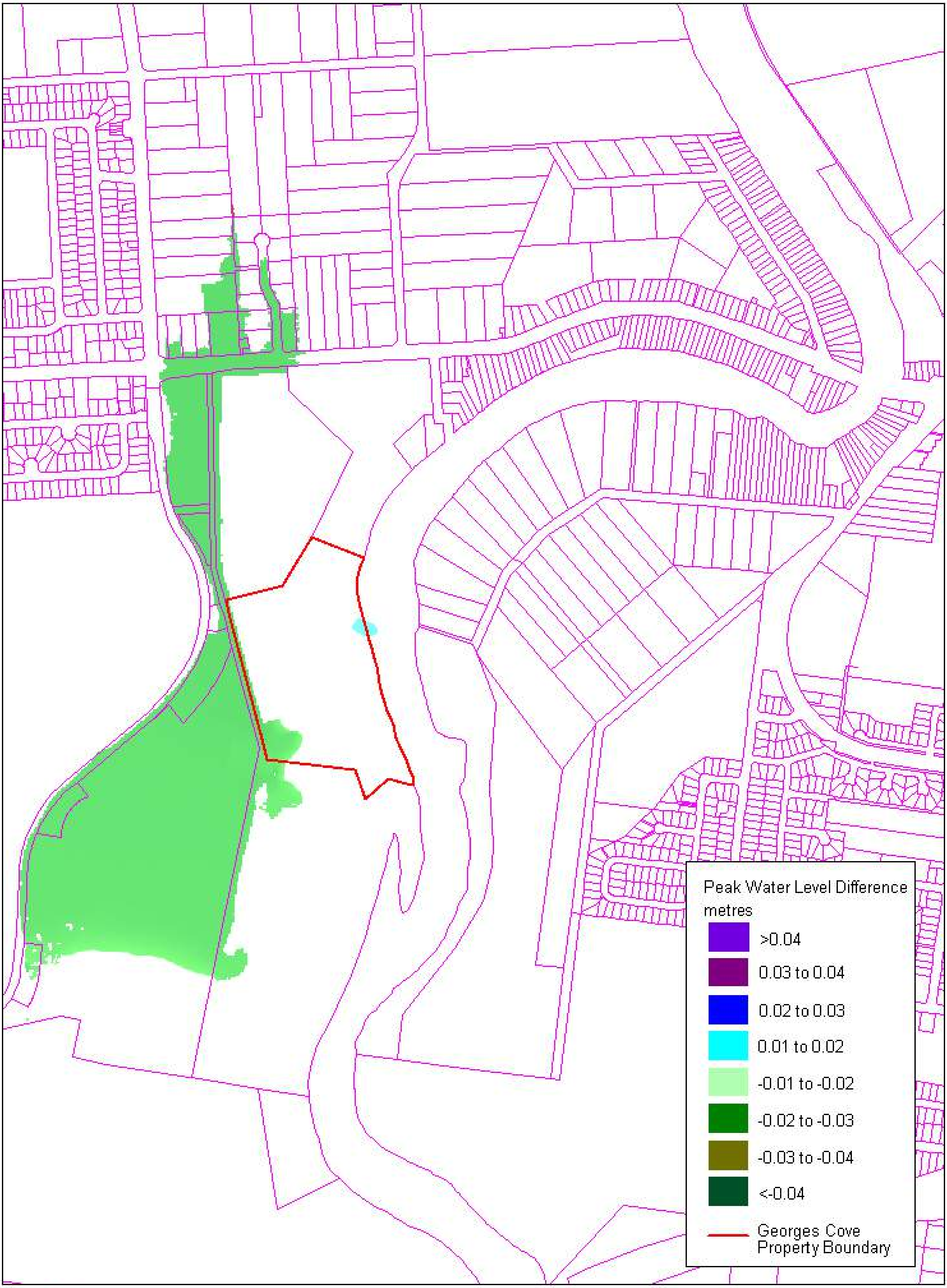

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Figure 4-3
20 year ARI
Peak Water Level
Post-development-Blockage Less Predevelopment

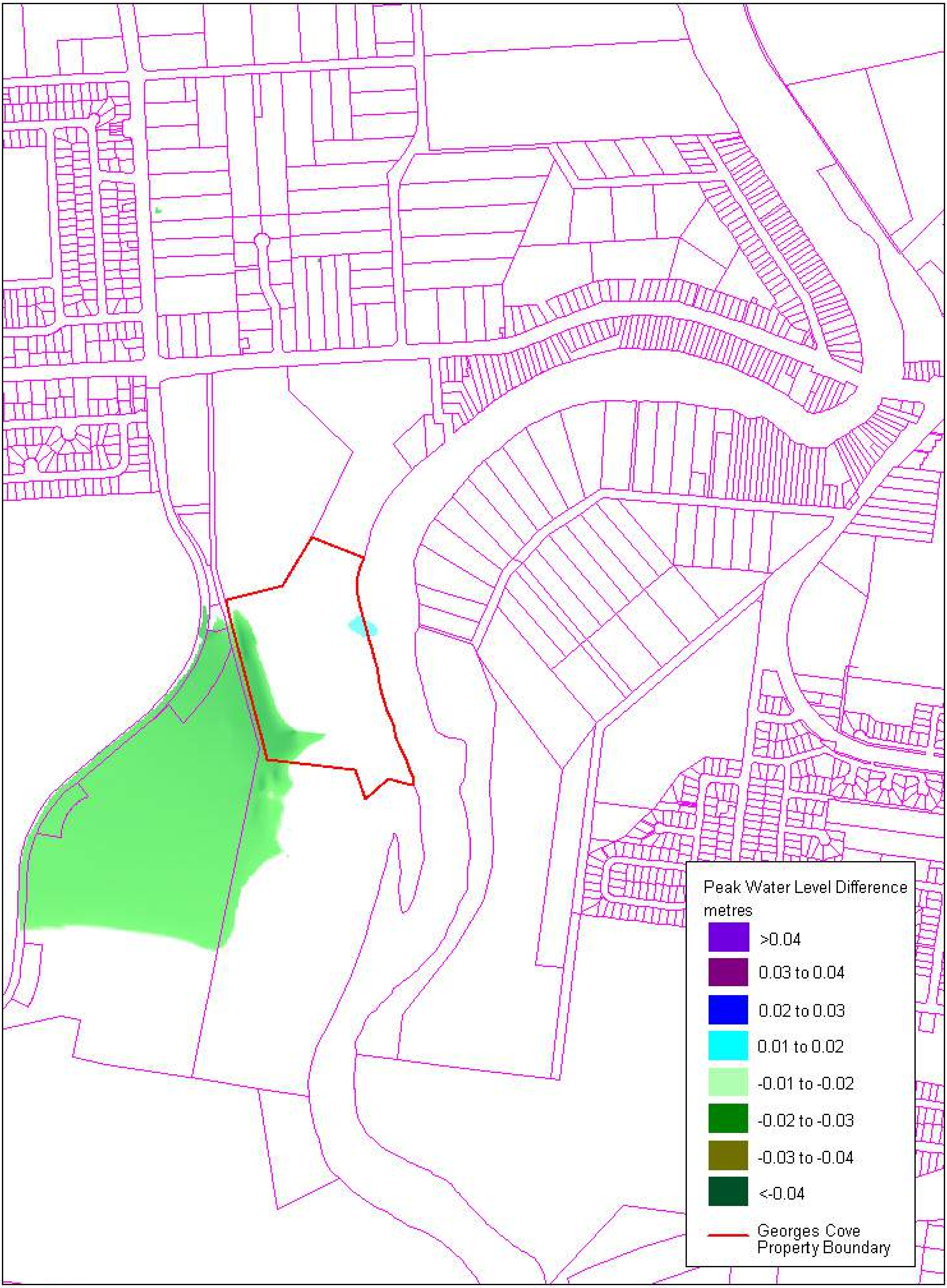

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Figure 4-4
100 year ARI
Peak Water Level
Post-development-Blockage Less Predevelopment

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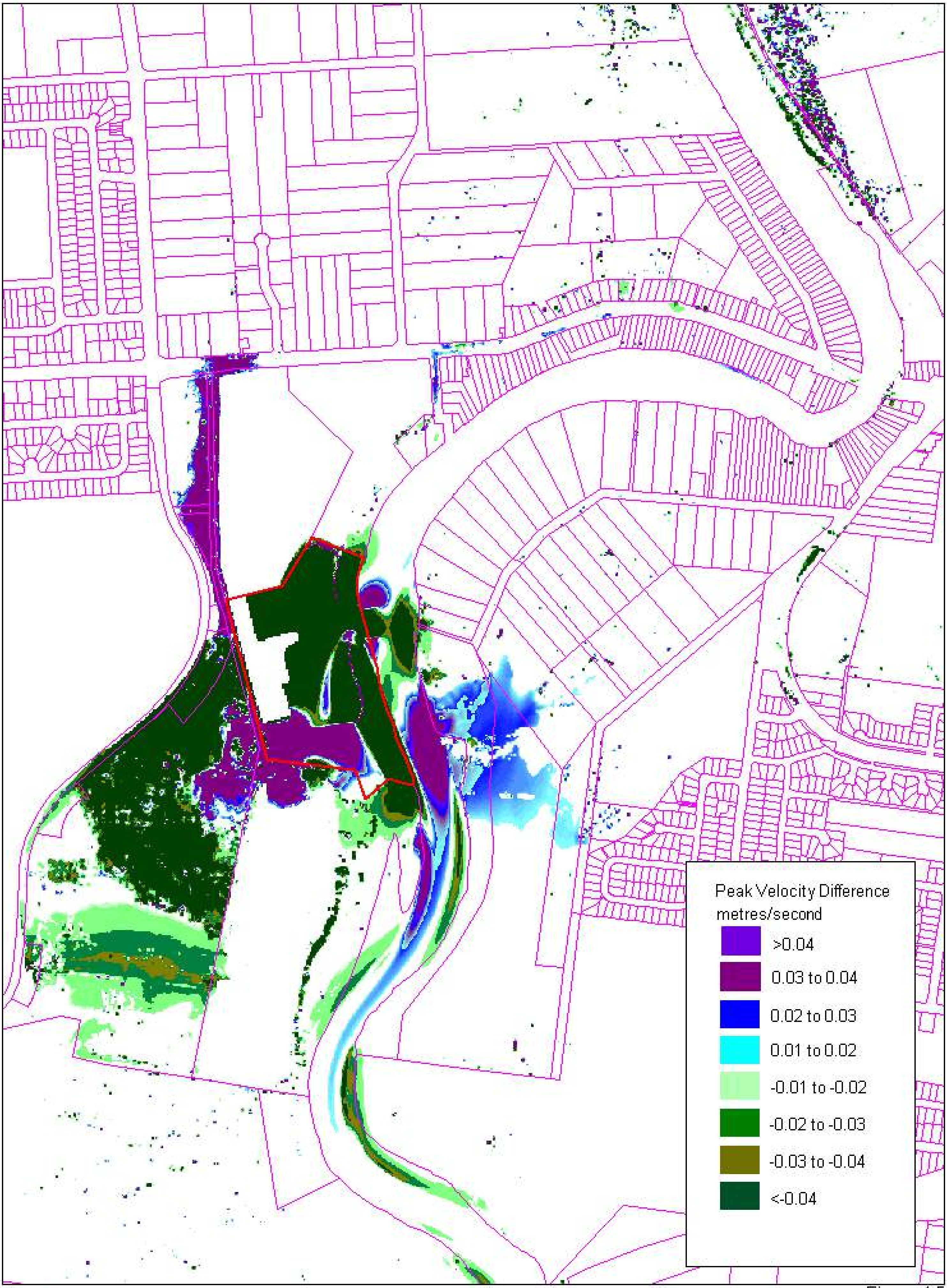

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Figure 4-5
20 year ARI
Peak Water Velocity
Post-development Less Predevelopment

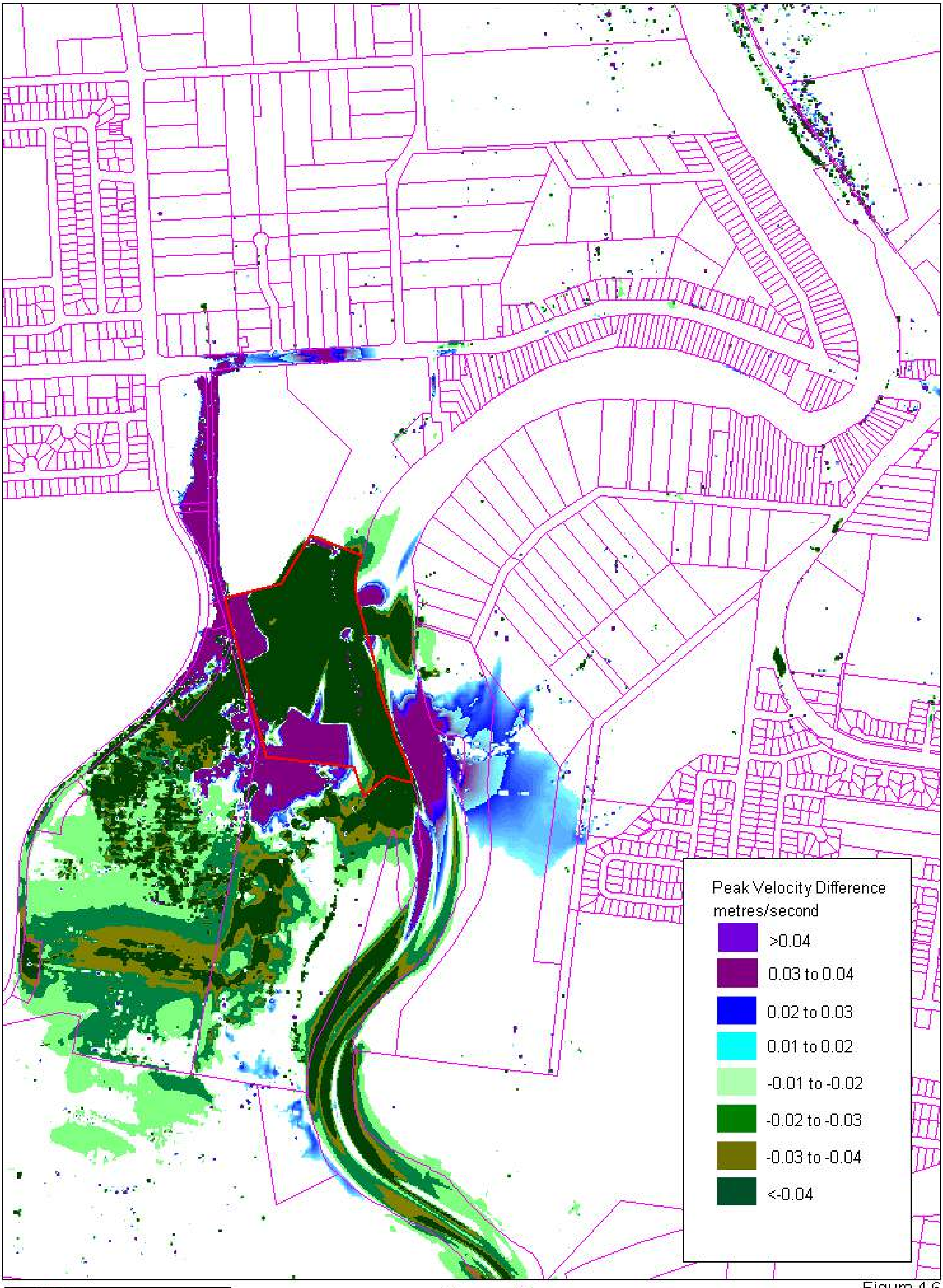

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Figure 4-6
100 year ARI
Peak Water Velocity
Post-development Less Predevelopment

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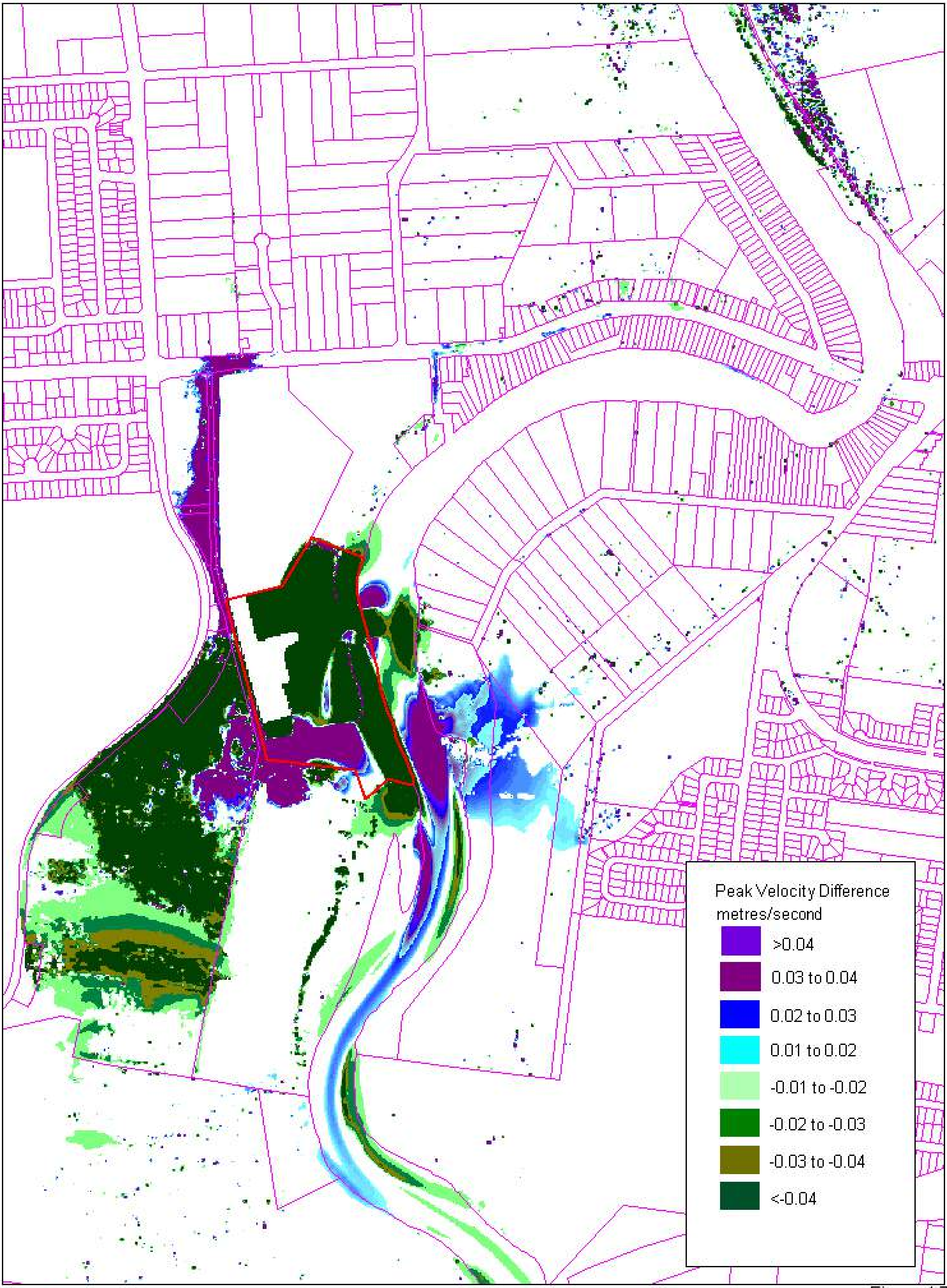

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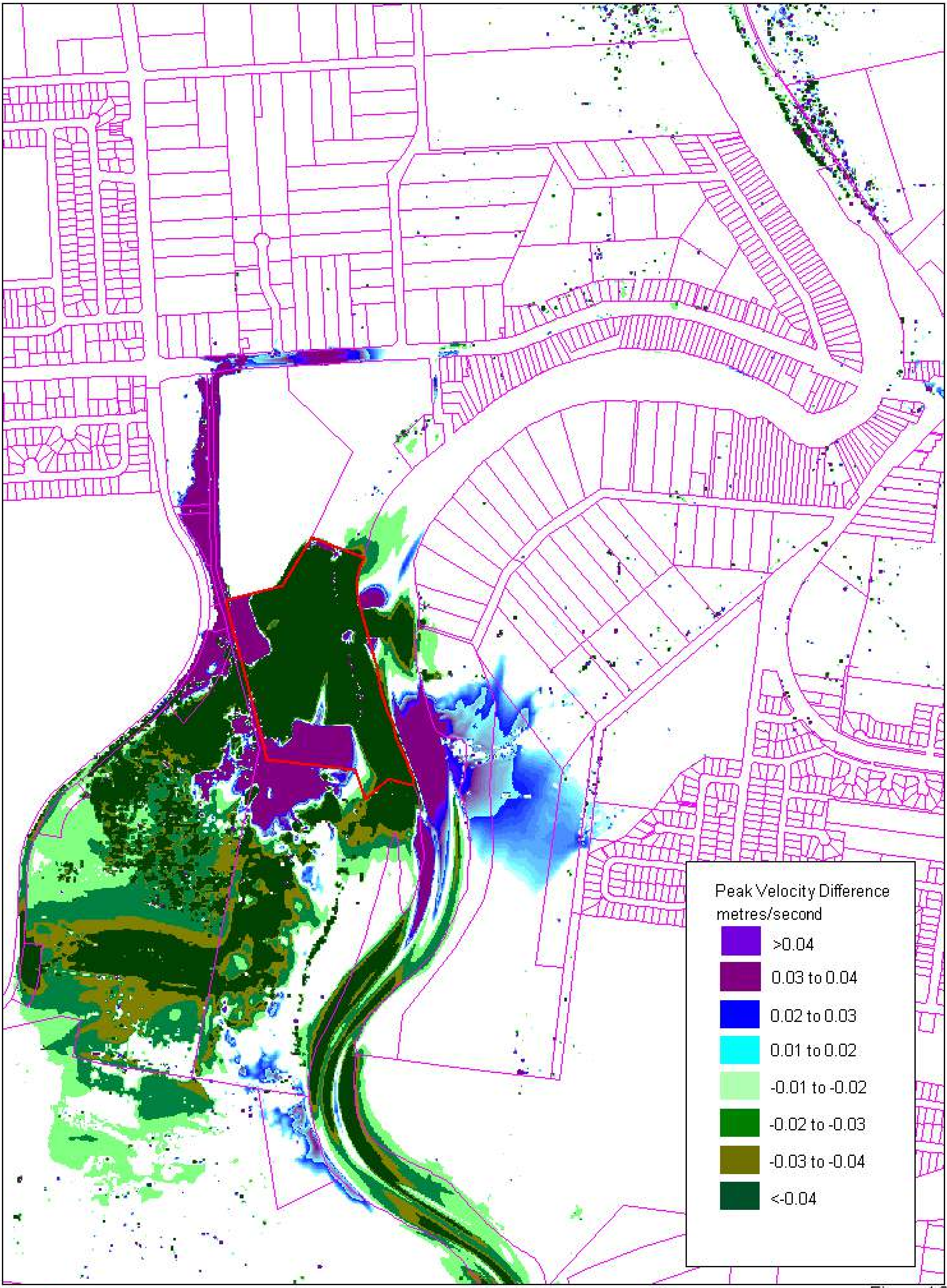
Figure 4-7
20 year ARI
Peak Water Velocity
Post-development-Blockage Less Predevelopment



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Flood Impact Assessment

Figure 4-8
100 year ARI
Peak Water Velocity
Post-development-Blockage Less Predevelopment

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