Drainage Investigation Report Anna Bay North Structure Plan

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Port Stephens Council

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1. Introduction

1.1 Background

Parsons Brinckerhoff (PB) was commissioned by Port Stephens Council (PSC) to prepare a Structure Plan for Anna Bay North. This drainage investigation has been undertaken as part of the Structure Plan process, to identify the trunk drainage infrastructure required to mitigate potential impacts of future development on existing drainage and flooding in the study area.

1.2 Objectives

The objectives of this drainage investigation are to:

- determine the existing extent of flooding within the study area;
- determine site constraints in relation to flooding and drainage;
- determine the impact of future development within the study area on exiting flows and flooding;
- formulate a trunk drainage strategy to alleviate the impacts of future development on existing drainage and flooding; and
- size a floodway for the conveyance of the 100 year flow to the north of the Anna Bay township.

1.3 Available data

The following data was used for this investigation:

- Structure Plan of study area prepared by Parsons Brinckerhoff (identifying future development areas);
- 1:25,000 topographic maps and 1:4,000 orthophoto maps of the study area;
- aerial photograph, 2m contours and cadastre of the study area (in MAPINFO format) provided by PSC;
- details of the existing channel connecting Catchment 3 to Fern Tree Drain, including details of lateral pipes to drain flood storage areas, provided by PSC;
- details of existing infrastructure within Anna Bay, including detention basins, culverts and open channels, provided by PSC;
- cross sections of Fern Tree Drain and the existing floodplain to the north of Anna Bay provided by Monteath & Powes;

- detailed survey of the low lying area within Catchment 4 provided by Monteath & Powes; and
- the *Anna Bay Catchment Drainage/Flood Study Masterplan* (Sinclair Knight, 1994).

Numerous site inspections were undertaken across the study area for familiarisation purposes and to allow identification of potential critical areas.

1.4 Site characteristics

The study area is located on the Tomaree Peninsula, Port Stephens. The location of the study area is shown in Figure 1-1.

The study area catchment has an area of approximately 259 hectares. The catchment area includes a large portion of the town of Anna Bay, as well as development along Gan Gan Road to the east and west of the town.

The study catchment has been broken down into four reduced catchments, referred to as catchments 2, 3, 4 and 10. A catchment plan is given in Figure 1-2.

Elevations in the catchment range from approximately RL 70m AHD on the hills to the east of Anna Bay, to less than RL 0m AHD within drainage lines on the floodplain to the north of Anna Bay.

Slopes range from flat along the floodplain north of Anna Bay, to greater than 20 percent on the hills to the east of Anna Bay.

The catchment is generally characterised by sandy soils, however, some areas of indurated sands exist and the upper hillslopes comprise residual clayey soils over rock. Permeability is generally estimated to be medium to high, however, the areas of indurated sand and clayey soils over rock are of low permeability.

1.5 Existing drainage

The major catchment within the study area is drained by an unlined man-made channel, referred to as Fern Tree Drain. Fern Tree Drain flows north from the town of Anna Bay, then northwest, where it discharges to Anna Bay Main Drain approximately 150m downstream of Nelson Bay Road.

Anna Bay Main Drain flows west, then north, to Wallis Creek. Anna Bay Main Drain also collects runoff from Back Drain, which is located north of the study catchment. Wallis Creek discharges to Tilligerry Creek, which discharges to Port Stephens near Taylor's Beach.

A flood gate structure is located on Anna Bay Main Drain, between Wallis Creek and Port Stephens Drive. The floodgate prevents the entry of high tides and salt water into the upstream area.

Parts of the township of Anna Bay (catchments 2, 3 and 4) and the low lying areas to the north of Anna Bay (Catchment 10) are prone to flooding during long-duration storm events.

Catchment 2 comprises mainly developed areas of the Anna Bay township, but also comprises undeveloped dunal areas in the northwest and southwest. The majority of Catchment 2 drains to McKinley's Swamp, which is a natural detention basin located on the southern side of Gan Gan Road. McKinley's Swamp discharges north to Fern Tree Drain, through a number of culverts under Gan Gan Road and Old Main Road.

Catchment 3 comprises urban development to the north and south of Gan Gan Road, as well as parts of Tomaree National Park in the south. There are two detention basins located to the south of Gan Gan Road within Catchment 3. The first is a dry detention basin located to the rear of Anglers Drive (referred to as Basin 1) and the second is a wet detention basin located opposite Essington Way (referred to as Basin 2). Discharge from the basins, as well as discharge from other parts of Catchment 3, is to either Catchment 2 or Catchment 10. Low flows are piped west to McKinley's Swamp in Catchment 2, before discharging north to Fern Tree Drain in Catchment 10. High flows are piped north under the sand ridge to a man-made open channel. The channel flows north, then west, and discharges to Fern Tree Drain. The high flow pipe and channel system was constructed recently and was a recommendation of the previous flood study of Anna Bay undertaken for PSC by Sinclair Knight in 1994. There is no overland flow path out of Catchment 3.

Catchment 10 comprises mainly bushland and agricultural land, except for scattered residences along Gan Gan Road. Catchment 10 comprises the low lying floodplain to the north of the Anna Bay township, and drains directly to Fern Tree Drain.

Catchment 4 is largely undeveloped, with some rural-residential development to the north and south of Gan Gan Road. There is no defined drainage path from the catchment, with all runoff infiltrating into the underlying groundwater system. PSC has installed a groundwater pump to alleviate existing flooding problems during periods of prolonged rainfall. The groundwater pump discharges west to Catchment 3. The previous Anna Bay flood study (Sinclair Knight, 1994) recommended construction of a defined drainage system out of Catchment 4, north from Gan Gan Road to Anna Bay Main Drain. The main components of the recommended system are a pipe through the sand ridge and an open channel.

2. Design Criteria

The following design criteria were adopted for the drainage investigation:

- the trunk drainage system is capable of conveying the 100 year average recurrence interval (ARI) storm event from the site;
- a 200 millimetre freeboard is to be maintained between the estimated 100 year flood level and the top of bank for any trunk drainage structures;
- open channels are to have internal batter slopes no steeper than 1V:4H;
- detention basins within Catchment 4 are to have internal batter slopes no steeper than 1V:8H;
- allowance is to be made for an appropriate blockage factor for closed conduit detention basin outlets by taking into account a) the risk of blockage b) design features to mitigate that risk; and c) additional storage effects; and
- the extent of existing flooding within the township of Anna Bay is not to be exacerbated.

3. Methodology

3.1 Choice of computer model

The hydrodynamic model XP-SWMM (version 9.00) was used to assess the impacts of future development and the proposed trunk drainage strategy on existing flood levels in Anna Bay.

XP-SWMM is a graphics based wastewater and stormwater computer model that utilises a proprietary self-modifying dynamic wave solution algorithm to adjust time steps when appropriate. The RUNOFF mode of the model allows for hydrology generation and the HYDRAULICS mode allows hydraulic simulation of open and closed conduits, including detention basins.

The model allows adoption of 'Laurenson's Non-Linear Runoff Routing' method as used by XP-RAFTS, which was used for hydrologic modelling in the previous flood study (Sinclair Knight, 1994).

3.2 Hydrology

Runoff hydrographs for the 10 and 100 year ARI storm events were estimated for the catchment using the RUNOFF mode in XP-SWMM. In the absence of long-term stream flow data, the XP-SWMM model was compared to peak flows from the previous flood study (Sinclair Knight, 1994) and the Probabilistic Rational Method as described in Australian Rainfall & Runoff (AR&R) (1987).

3.3 Hydraulics

The HYDRAULICS mode in XP-SWMM was used to route the runoff hydrographs through the existing and proposed drainage system to allow estimation of flood levels. Downstream boundary conditions were based on the estimated flood levels documented in the previous flood study (Sinclair Knight, 1994).

Preliminary sizing was undertaken for key trunk drainage components, including detention basins and floodways.

3.4 Development scenarios modelled

The model was run for the study catchment in the existing situation.

The model was also run for the following two development scenarios:

- Scenario 1 – Included management measures within individual allotments, provision of detention basins within Catchment 4 and provision of a defined drainage path out of Catchment 4. The existing floodplain and channel geometry to the north of the Anna Bay township was maintained.

- Scenario 2 - Included management measures within individual allotments, provision of detention basins within Catchment 4 and provision of a defined drainage path out of Catchment 4. An engineered floodway was provided to the north of Anna Bay to convey 100 year flows from the study catchment.

4. Existing modelling

This section describes calibration of the XP-SWMM model, and modelling of the existing situation.

4.1 Catchment plan

A catchment plan was prepared for the study area using 1:4000 orthophoto maps and details of existing drainage infrastructure in Anna Bay provided by PSC. The catchment plan is generally consistent with the plan used in the previous flood study (Sinclair Knight, 1994).

The study catchment was broken down into four reduced catchments, referred to as catchments 2, 3, 4 and 10. These catchments were further broken down into subcatchments to improve the spatial resolution of modelled catchment parameters, and to enable modelling of individual detention basins.

The total catchment area is approximately 259 hectares. Areas for catchments 2, 3, 4 and 10 are given in Table 4-1.

Table 4-1 Summary of catchment areas

Catchment	Area (ha)
2	41.0
3	72.0
4	68.0
10	78.0

A catchment plan for the existing situation is given in Figure 1-2.

4.2 Inflows to hydrodynamic model

The RUNOFF mode of XP-SWMM was used to estimate existing hydrographs at each node. The Laurenson routing method was used for the routing of excess rainfall to each subcatchment outlet.

4.2.1 RUNOFF model parameters

A summary of existing subcatchment data used in the RUNOFF mode of XP-SWMM is given in Appendix A. Data includes the area, existing percentage imperviousness and average slope for each subcatchment.

The existing percentage imperviousness was estimated from 1:4000 orthophoto maps, aerial photography of the study area and site inspections. Existing residential development was assumed to be 40 percent impervious, which is in accordance with the previous flood study (Sinclair Knight, 1994).

For the existing situation, two different loss models were used to represent the various land types within the study area. These were the 'pervious loss model' and 'impervious loss model'. The initial loss, continuing loss rate and overland roughness for each loss model is summarised in Table 4-2. Initial loss and continuing loss rates for the 'pervious loss model' were taken from the previous flood study (Sinclair Knight, 1994) and are considered appropriate for predominantly sandy soils. The sensitivity of the model to various antecedent moisture conditions has not been examined.

Table 4-2 Summary of loss models in the RUNOFF mode

Loss model	Initial loss (mm)	Continuing loss rate (mm/hr)	Overland Mannings 'n'
Pervious	20	10	0.080
Impervious	1.5	0	0.015

4.2.2 Design rainfall

Design rainfall pluviographs for a range of storm durations and recurrence intervals were generated by XP-SWMM using average rainfall intensity data for Anna Bay and rainfall temporal patterns from AR&R (1987).

4.2.3 RUNOFF model calibration

In the absence of recorded stream flow data from the site, peak flow estimates from the RUNOFF mode of XP-SWMM were compared against estimates from the Probabilistic Rational Method for a number of selected subcatchments. The comparison was done for local runoff from each subcatchment, and inflows from upstream subcatchments were not considered.

Probabilistic Rational Method calculations are given in Appendix B. Comparisons of Rational Method and XP-SWMM existing peak flows estimates for the 10 and 100 year ARI storm events are also given in Appendix B.

The peak flows predicted by XP-SWMM generally compare well with those predicted by the Rational Method. However, for some low-lying and flat subcatchments the Rational Method peak flows are higher than the XP-SWMM peak flows. This is because the Rational Method calculations did not account for the extremely flat grade or high initial losses and continuing loss rates.

A rough comparison can also be made between the peak flows predicted by the XP-SWMM model and those documented in the previous flood study (Sinclair Knight, 1994). The comparison is a guide only, as significant drainage improvements and some development has occurred in the study area since the Sinclair Knight 'existing' situation. However, not all of the drainage improvements or development accounted for in the Sinclair Knight 'future' situation has occurred. Drainage improvements within the study catchment since 1994 include:

- provision of an open channel to conveys flows from Catchment 3 to Fern Bay Drain;
- provision of three additional culverts on Fern Tree Drain under Nelson Bay Road;

- provision of a second detention basin in Catchment 3;
- pumping of groundwater flows from Catchment 4 west to Catchment 3; and
- improvements to the outlet of McKinley's Swamp in Catchment 2.

A comparison of peak flows in Fern Tree Drain at Nelson Bay Road is given in Table 4-3. The peak flows account for inflows from upstream catchments.

Table 4-3 Comparison of predicted peak flows from XP-SWMM and the previous flood study (Sinclair Knight, 1994)

	Peak Flow (m^3/s)					
	XP-SWMM existing		Sinclair Knight existing		Sinclair Knight developed (with culvert improvements)	
Location	10yr ARI	100yr ARI	10yr ARI	100yr ARI	10yr ARI	100yr ARI
Fern Tree Drain at Nelson Bay Rd	4.7	7.5	3.0	3.3	6.0	7.8

From Table 4-3 it can be seen that peak flows from the previous flood study (Sinclair Knight, 1994) and the XP-SWMM model compare well. The predicted XP-SWMM 'existing' flow is greater than the Sinclair Knight 'existing' flow due to development and drainage improvements being undertaken in the study area since 1994, including works to drain high flows from Catchment 3 north to Fern Tree Drain. The predicted XP-SWMM 'existing' flow is less than the Sinclair Knight 'future' flow due to not all of the development or improvement works accounted for in the Sinclair Knight 'future' situation having occurred to date.

4.3 Hydrodynamic model inputs

The HYDRAULICS mode of XP-SWMM was used to estimate existing flood levels within the trunk drainage system. In this mode, XP-SWMM also models the storage effects of floodplains, retention basins and open channels.

4.3.1 Hydrograph input

Runoff hydrographs for the 10 and 100 year ARI storm events were estimated at node locations using the RUNOFF mode in XP-SWMM. Details for hydrograph estimation are provided above in Section 4.2.

4.3.2 Downstream boundary conditions

Fern Tree Drain at Nelson Bay Road represents the downstream boundary of the study area.

The boundary condition for the XP-SWMM model comprised constant tailwater levels of 1.21m and 1.52m for the 10 and 100 year ARI storm events, respectively. These levels are the maximum water levels documented in the previous flood study (Sinclair Knight, 1994) for the 'existing' situation. It should be noted that the documented water levels were higher for the Sinclair Knight 'existing' situation rather than the 'future' situation, and were adopted for a conservative approach.

4.3.3 Flooding scenarios

There are many combinations of weather patterns that can cause flooding in low lying coastal areas. Flooding within the study area could be caused by:

- runoff generated from rainfall on site;
- high water levels in Port Stephens; or
- a combination of the above.

The previous flood study (Sinclair Knight, 1994) considered rainfall on site coinciding with maximum storm surge in Port Stephens (1.7 m AHD). This represents low tailwater conditions, with no coincident flooding in Tilligerry Creek or Port Stephens and all outflows from the study area through the floodgate structure on Anna Bay Main Drain. Thus it depicts minimum floodplain storage which gives the maximum discharges from the catchment and therefore the maximum impacts of development in a local storm event.

As mentioned above, the flood levels in Fern Tree Drain at Nelson Bay Road documented in the previous flood study were adopted as tailwater conditions for the XP-SWMM model.

It should be noted that the low tailwater condition does not give the worst case flood levels. Maximum flood levels would occur for the high tailwater condition (2.2 m AHD) with backup from regional flooding in Port Stephens and Tilligerry Creek (Sinclair Knight, 1994). The level of 2.2m AHD should apply as the minimum flood level.

4.3.4 Channel and conduit geometry

The pit and pipe network (referred to as pits 1 to 6) along Gan Gan Road to the east of the Morna Point intersection was represented in the HYDRAULICS mode of XP-SWMM as a series of conduits and weirs. It was necessary to model this network in XP-SWMM to reproduce the low and high flow bypasses from Catchment 3 to Catchments 2 and 10, respectively. Engineering drawings of this network, including low flow and high flow pipe long sections and pit details, were provided by PSC (reference: 'Main Drainage Relief Scheme – Anna Bay' file no. E5105-06, plan no. D-102).

The high flow bypass, connecting Catchment 3 to Fern Tree Drain, was represented in the HYDRAULICS mode as a series of conduits and trapezoidal channels. The engineering drawings provided by PSC included long sections and cross sections of the pipe and channel.

There was limited information available on the existing trunk drainage system within other areas of the Anna Bay township. Connectivity between subcatchments was therefore generally modelled in the HYDRAULICS mode as trapezoidal channels with 1V:2H side slopes and a variable base width and longitudinal grade.

Natural cross-sections of Fern Tree Drain and the floodplain north of Anna Bay have been derived from detailed survey information, 1:4000 orthophoto maps and field observations.

Connectivity between all subcatchments draining to the trunk drainage system was modelled as trapezoidal channels with 1V:2H side slopes and a variable base width and longitudinal grade.

An output file from the existing XP-SWMM model is given in Appendix C. The output file includes details of modelled control structures and open channels.

4.3.5 Storage nodes

Storage characteristics of McKinley's Swamp were estimated from 1:4000 orthophoto maps. Details of the basin outlet configuration were obtained from PSC.

Storage characteristics of 'Basin 1' (dry basin at rear of Angler's Drive) and 'Basin 2' (wet basin opposite Essington Way) within Catchment 3 were estimated from detailed survey. Details of the basin outlet configurations were obtained from PSC.

Storage characteristics of the low lying area within Catchment 4 were estimated from detailed survey information. The HYDRAULICS mode includes a pump (maximum flow rate $0.027m^3/s$) from the low lying area of Catchment 4 to Catchment 3. This represents the groundwater pump at 192 Gan Gan Road, which is used to alleviate existing flooding problems.

Storage characteristics of the floodplain to the north of Anna Bay were estimated from detailed survey information (cross sections). Storage characteristics of the floodplain to the northwest of Anna Bay (downstream of the sand ridge crossing of Fern Tree Drain) were estimated from 1:4000 orthophoto maps.

An output file from the existing XP-SWMM model is given in Appendix C. The output file includes details of modelled storage nodes.

4.3.6 Channel and conduit roughness

Closed conduits were assumed to have a Mannings 'n' of 0.013 to 0.014. Open channels were assumed to have a Mannings 'n' of 0.033, which is in accordance with the previous flood study (Sinclair Knight, 1994).

Mannings 'n' values adopted for overbank areas of the floodplain north of town ranged from 0.060 (for cleared areas) to 0.120 (for heavily vegetated areas). These values were chosen based on site inspections and examination of aerial photography.

An output file from the existing XP-SWMM model is given in Appendix C. The output file includes details of adopted channel and conduit roughness's.

4.4 Hydrodynamic model output

Estimated 10 and 100 year ARI peak flows and critical storm durations are provided in Table 4-4 for existing conditions. The peak flows are for low tailwater conditions. Peak flows are given for all subcatchments within Catchment 10, as well as for detention storages within catchments 2, 3 and 4.

Table 4-4 Estimated 10 and 100 year ARI peak flows and critical storm durations for existing situation

Node	10 yr ARI		100 yr ARI	
	Peak flow (m^3/s)	Critical storm duration (min)	Peak flow (m^3/s)	Critical storm duration (min)
10a	0.75	120	1.57	120
10b	1.56	120	2.52	120
10c	1.70	90	2.30	120
10d	3.29	120	4.74	120
10e	3.51	120	5.13	120
10f	3.90	120	5.77	120
10g (Nelson Bay Road)	4.70	540	7.51	120
2b (McKinley's Swamp)	0.94	540	1.83	540
3d (Basin 1)	0.19	540	0.77	540
3b (Basin 2)	0.19	720	3.67	540
4c (groundwater pump)	0.027	NA	0.027	NA

From Table 4-4 it can be seen that the peak flows in Fern Tree Drain at Nelson Bay Road are 4.70 and 7.51 m^3/s for the 10 and 100 year ARI storm events, respectively. The critical storm duration is generally 120 minutes for subcatchments 10a to 10f, located on the low lying area to the north of the Anna Bay township. The critical storm duration for the detention basins within the town of Anna Bay is generally 540 minutes.

Estimated 10 and 100 year ARI peak water levels at critical locations within the study catchment are provided in Table 4-5 for existing conditions. The peak water levels are for low tailwater conditions.

Table 4-5 Estimated 10 and 100 year ARI peak water levels for existing situation

Node	10 yr ARI		100 yr ARI	
	Peak water level (m AHD)	Critical storm duration (min)	Peak water level (m AHD)	Critical storm duration (min)
10a	4.12	120	4.18	120
10b	1.90	120	2.11	120
10c	1.79	120	1.93	120
10d	1.74	120	1.88	120
10e	1.50	120	1.78	120
10f	1.33	120	1.65	120
10g (Nelson Bay Road)	1.21	120	1.52	120

Node	10 yr ARI		100 yr ARI	
	Peak water level (m AHD)	Critical storm duration (min)	Peak water level (m AHD)	Critical storm duration (min)
2b (McKinley's Swamp)	2.42	540	3.05	540
3d (Basin 1)	3.40	540	3.64	540
3b (Basin 2)	4.21	720	4.52	540
4c (groundwater pump)	5.14	720	5.78	720

From Table 4-5 it can be seen that the peak water levels in Fern Tree Drain at Nelson Bay Road are the same as the low tailwater conditions set for the model.

5. Trunk drainage strategy

This section describes the key trunk drainage components proposed for managing developed runoff from the study area. The effectiveness of the trunk drainage components is quantified through hydrologic and hydraulic modelling. Further detailed modelling is recommended as part of a development application for proposed development within the study area.

5.1 Private infrastructure within allotments

Rainwater tanks

Rainwater tanks would be provided on each allotment for the collection of roof runoff. Water collected in the tanks would be used for non-potable applications such as landscape watering, toilet flushing and laundry purposes. The use of rainwater tanks can substantially reduce both the peak flow and annual discharge volume for minor storm events. The tanks could be designed to include an upper temporary storage volume for peak flow attenuation in addition to a lower permanent storage volume for water reuse.

Rainwater tanks should be fitted with a first flush bypass device to intercept the first volume of runoff and prevent it entering the tank. Water captured by the bypass device should be discharged to surrounding landscape areas.

Overflows from rainwater tanks should be directed to an onsite infiltration trench or soakage area where practicable. Where it is not practicable to direct runoff to an infiltration trench or soakage area (e.g. unsuitable soil types, high groundwater table) overflows should be connected to the street or interallotment drainage system.

Rainwater tanks would help new residences achieve compliance with the Building Sustainability Index (BASIX), which was introduced into the NSW development approval process on 1 July 2004. The introduction of BASIX means that people who wish to build new homes need to include a BASIX Certificate with their development proposals. BASIX focuses on reducing the amount of mains supply water used in, and greenhouse gases emitted from, residential development.

Infiltration trenches or soakage areas

Infiltration trenches or soakage areas would be provided on each allotment for the collection and infiltration of rainwater tank overflows. In addition to rainwater tanks overflows, runoff from some other allotment areas would also be conveyed to infiltration trenches or soakage areas.

Infiltration trenches would be designed to increase soil infiltration and maintain existing groundwater levels in developed areas. The presence of indurated sand within catchments 3 and 10 has the potential to limit infiltration and consideration should be given to siting structures in locations free of indurated sand or where it may be easily excavated and replaced with free draining material. Elevated groundwater levels may also limit the potential for infiltration (as soil salinity problems may occur) and should be considered during the development application stage.

Infiltration trenches would be capable of capturing the one year ARI one hour storm.

5.2 Defined drainage path from Catchment 4

A constructed drainage system would be provided to connect Catchment 4 to Anna Bay Main Drain. The major components of the system would be pipe drainage (through the sand ridge) and approximately 590m of open channel.

The previous flood study (Sinclair Knight, 1994) proposed a 1.2m diameter pipe at grade 0.5%, with a maximum flow rate of $3m^3/s$, from Catchment 4. The maximum flow rate from Catchment 4 to Anna Bay Main Drain was therefore limited to $3m^3/s$ for this drainage investigation.

5.3 Detention basins

Two dry detention basins will be provided within Catchment 4 to capture and attenuate stormwater runoff. These basins have been sized to contain the 100 year ARI storm event in Catchment 4, with the outflow from the catchment limited to $3m^3/s$.

The basins are located on either side of Gan Gan Road, and are hydraulically connected by triple 900mm diameter culverts. Two basins were provided to allow ease of drainage of future development, and to reduce filling requirements. The basin to the north of Gan Gan Road (‘northern’ basin) has an invert of 3.5m AHD, and the basin to the south (‘southern’ basin) has an invert of 3.7m AHD. Both basins would be excavated into the natural surface to reduce the surface area required (natural invert approximately 4.5m AHD). The previous flood study (Sinclair Knight, 1994) indicates that the groundwater level measured at the location of the basin is less than 2.5m AHD, however, this should be confirmed by way of long term monitoring to ensure that basin inverts are above the groundwater table. In addition, the presence of indurated sand should also be investigated to ensure that the base of each basin is free draining.

The top of bank of each basin was set to 5.3m AHD, which is the lowest elevation of Gan Gan Road in this area. Side slopes of 1V:8H were provided within each basin to ensure public safety.

A plan showing the location and extent of the detention basins is given in Figure 5-1.

The outlet from the northern basin comprises approximately 40m of 1.8m wide x 0.6m high box culvert, followed by approximately 150m of 1.2m diameter pipe constructed through the sand ridge. The box culvert was required at the basin outlet to achieve minimum cover beneath potential development areas to the north of the northern basin.

The previous flood study (Sinclair Knight, 1994) predicted the peak 100 year ARI water level in Anna Bay Main Drain at Murrumburrimbah Swamp to be 1.48m AHD. As this level is significantly lower than the northern basin invert of 3.5m AHD, it is unlikely that tailwater conditions in Anna Bay Main Drain would control discharge from the basin.

For the purpose of basin sizing, the soil profile beneath each basin was assumed to be saturated due to elevated groundwater levels and infiltration was limited. The basin was sized to maintain a freeboard of 200mm between the peak water level in the basins and the Gan Gan Road pavement during a 100 year ARI storm event. A check was undertaken to ensure that Gan Gan Road would not be overtopped during a 100 year ARI storm event if the outlet of the northern basin was 50% blocked. Given the absence of a formal overflow path from Catchment 4, the recommended finished floor levels should be set to a minimum of 500mm above the estimated 50% blocked flood level.

The detention basin design presented above is indicative only and should be further refined and incorporated into the future development layout for the area. It may be possible to reduce the combined volume of the basins by increasing the discharge above $3m^3/s$. However, this would require further examination of the Anna Bay Main Drain to ensure that the increased discharge has no impact on existing flow rates and water levels.

Alternatively, a basin could be provided in the upper reaches of Catchment 4 to reduce the size of the basins located adjacent to Gan Gan Road.

5.4 Floodway

Two scenarios for the floodway to the north of the Anna Bay township have been considered in this investigation. These are:

- Scenario 1 - The existing floodplain and channel geometry to the north of the Anna Bay township was maintained.
- Scenario 2 - An engineered floodway was provided to the north of Anna Bay to convey 100 year flows from the study catchment. The floodway would be at a constant grade between Nodes 10b and 10f. The floodway would be unlined to allow infiltration, with overbank areas planted with native tree species.

The location of the engineered floodway for Scenario 2 is shown in Figure 5-2.

Both scenarios included management measures within individual allotments, provision of detention basins within Catchment 4 and provision of a defined drainage path out of Catchment 4.

5.5 Pit and pipe network

Localised runoff from roadways and other developed areas should be captured and conveyed in a pit and pipe network sized to convey the 10 year ARI storm event. Flows in excess should be conveyed along roadways and other designated overland flow paths at safe velocities.

Outlet headwalls or surcharge pits with suitable scour protection should be provided on all pipe outlets discharging to channels to minimise erosion at these locations.

5.6 Vegetated swales and channels

Vegetated swales should be incorporated into the urban design for the study area and situated along streets and parklands. They would be used to convey stormwater in lieu of concrete pipes and when constructed on mild slopes would convey stormwater slowly downstream. This has the benefit of increasing catchment travel times and reducing peak flow rates. Given the predominantly sandy soils in the study catchment, it is likely that high infiltration rates would occur through the base of these structures. These structures should be considered in combination with pit and pipe networks throughout the development.

Unlined channels will be retained to encourage infiltration and transpiration and will also provide a mechanism by which runoff rates may be reduced due to interaction with vegetation. Existing areas of channel erosion should be stabilised with native vegetation and rock protection if required.

6. Developed modelling

This section describes XP-SWMM modelling for the developed situation and examines the effectiveness of the proposed trunk drainage strategy on mitigating the impacts of future development on existing drainage conditions.

6.1 Parameters

Estimates for the developed situation were based on the suitable development areas identified in the Anna Bay Structure Plan. Subcatchment data for the developed situation is given in Appendix A.

Three types of residential development are included in the Structure Plan. These are general, medium lot and large lot residential development. Development densities are provided in Table 6-1.

For the purposes of hydrologic and hydraulic modelling, assumptions were made with regards to the breakdown of future development areas. The assumed road area per allotment and number of allotments per hectare are provided in Table 6-1. The assumed breakdown of each allotment is provided in Table 6-2.

Table 6-1 Assumed number of allotments per hectare of future development

Residential development type	Average lot area (m^2)	Assumed road area per lot (m^2)	Lots per ha
General	600	150	13.3
Medium Lot	400	115	19.4
Large Lot	5000	375	1.9

Table 6-2 Assumed breakdown of allotment areas for future development

Residential development type	Roof area (m^2)	Impervious area to infiltration trench (m^2)*	Pervious area to infiltration trench (m^2)	Impervious area to street (m^2)*	Pervious area to street (m^2)
General	300	45	105	45	105
Medium Lot	300	15	35	15	35
Large Lot	400	45	105	155	4295

* Does not include roof area

Loss models

The pervious and impervious loss models used for the existing situation were used for the developed situation. In addition to these loss models, where it is proposed to incorporate rainwater tanks into residential development, a 'roof loss model' was also used. Details of the 'roof loss model' are given in Table 6-3.

Table 6-3 Details of the 'roof loss model' used for the developed situation

Loss model	Initial loss (mm)	Continuing loss rate (mm/hr)	Overland Mannings 'n'
Roof	0.5	0	0.015

Rainwater tanks

It was assumed that each allotment contains a 5m^3 rainwater tank for the capture of roof runoff. The tanks were represented as storage nodes in the HYDRAULICS mode of XP-SWMM, with all tanks for each subcatchment being grouped together. All storages were assumed to have a height of 1m, and the surface area was varied to achieve the required volume. The storages were modelled with a weir at depth 1m to simulate tank overtopping.

Tanks have been assumed half full at the start of each simulation. The sensitivity of varying tank capacities or starting volumes has not been examined.

Infiltration trenches or soakage areas

It was assumed that each allotment contains an infiltration trench or soakage area for the collection and infiltration of rainwater tank overflows. In addition to rainwater tanks overflows, runoff from some other allotment areas would also be conveyed to the infiltration trenches or soakage areas. Details of these areas are given above in Table 6-2.

Infiltration trenches were sized to capture the one year ARI one hour storm at Anna Bay (rainfall depth of 28.9mm). Runoff coefficients of 1.0 and 0.5 were adopted for impervious and pervious surfaces, respectively. The infiltration trench volumes per lot for the future development types are provided below in Table 6-4.

Table 6-4 Assumed infiltration trench or soakage volume per lot for future development types

Residential development type	Infiltration trench or soakage volume per lot (m^3)
General	6.5
Medium Lot	4.6
Large Lot	9.4

Infiltration trenches were represented as storage nodes in the HYDRAULICS mode of XP-SWMM, with all basins for each subcatchment being grouped together. All storages were assumed to have a height of 0.5m, and the surface area was varied to achieve the required volume. The storages were modelled with a weir at depth 0.5m to simulate basin overtopping.

It has been assumed that infiltration trenches are empty at the start of each simulation. The sensitivity of varying basin capacities or starting volumes has not been examined.

Defined drainage path from Catchment 4

A drainage system connecting the detention basins in Catchment 4 to Anna Bay Main Drain was modelled for the developed situation. The drainage system included approximately 40m of 1.8m wide x 0.6m high box culvert, followed by 150m of 1.2m diameter pipe and 590m of open channel. The open channel was represented in XP-SWMM as a trapezoidal channel, with 1m wide base and 1V:2H side slopes.

As the study catchment did not include Anna Bay Main Drain, only the outflows from Catchment 4 were modelled. The downstream boundary condition for the Catchment 4 outlet comprised constant tailwater levels of 1.27m and 1.48m for the 10 and 100 year ARI storm events, respectively. These levels are the maximum water levels documented for ‘future’ conditions in the previous flood study (Sinclair Knight, 1994) for Anna Bay Main Drain at Murrumburrimbah Swamp.

It should be noted that for the developed situation separate boundary conditions, and outlets, were modelled for catchments 4 and 10. This was achieved by developing two separate XP-SWMM models, the first model comprising catchments 2, 3 and 10 and the second model comprising Catchment 4.

Detention basins

Two dry detention basins located on either side of Gan Gan Road have been modelled within Catchment 4 for the developed situation. The basin to the south of Gan Gan Road (southern basin) discharges to the basin to the north of Gan Gan Road (northern basin) through triple 900mm diameter pipes. The northern basin discharges to Anna Bay Main Drain through approximately 40m of 1.8m wide x 0.6m high box culvert, followed by pipe and open channel (refer above).

The northern basin has an invert of 3.5m AHD, and the southern basin has an invert of 3.7m AHD. The top of bank of each basin was set to 5.3m AHD, with 1V:8H side slope.

No infiltration was modelled from the detention basins.

An extract of the output file from the developed XP-SWMM model is given in Appendix C. The extract includes height-storage relationships for the ‘northern’ and ‘southern’ basins.

Engineered floodway

An engineered floodway to the north of the Anna Bay township was modelled for Scenario 2. A Mannings ‘n’ of 0.033 was assumed for the main channel and 0.060 for overbank areas.

Groundwater pump

The groundwater pump within Catchment 4 has been removed from the developed XP-SWMM model.

Pit and pipe network

Pit and pipe networks have not been modelled for future development due to the preliminary nature of this investigation.

Vegetated swales and other drainage lines

Vegetated swales have not been modelled for future development due to the preliminary nature of this investigation. As development layouts are developed, the benefits of vegetated swales should be assessed.

6.2 Performance

6.2.1 Catchments 2, 3 and 10

Scenario 1

Estimated 10 and 100 year ARI developed peak flows and critical storm durations within catchments 2, 3 and 10 are provided in Table 6-5 for Scenario 1. The percentage change in peak flows from the existing situation is also tabulated. The peak flows are combined with low tailwater conditions as previously discussed.

Table 6-5 Estimated 10 and 100 year ARI developed peak flows and critical storm durations in Catchments 2, 3 and 10 for Scenario 1

	10 yr ARI			100 yr ARI		
Node	Developed peak flow (m^3/s)	Critical storm duration (min)	% Peak flow difference	Developed peak flow (m^3/s)	Critical storm duration (min)	% Peak flow difference
10a	0.70	90	-6%	1.25	90	-20%
10b	1.46	180	-6%	2.09	120	-17%
10c	1.76	120	+4%	2.13	120	-7%
10d	3.13	120	-5%	4.58	120	-3%
10e	3.42	120	-3%	5.00	120	-3%
10f	3.89	120	0%	5.70	120	-1%
10g (Nelson Bay Rd)	4.67	120	-1%	7.46	120	-1%
2b (McKinley's Swamp)	0.94	540	0%	1.33	540	-27%
3d (Basin 1)	0.19	540	-1%	0.73	540	-5%
3b (Basin 2)	0.20	720	+2%	3.08	540	-16%

From Table 6-5 it can be seen that the critical storm durations for developed Scenario 1 are generally the same as those for the existing situation.

Peak flows are generally similar between the existing and developed situation, with differences ranging up to -6% for the 10 year ARI storm event and up to -27% for the 100 year ARI storm event. Decreases in peak flows may be attributed to rainwater tanks storing runoff from roof areas, and runoff from impervious areas associated with future development passing through the catchment before runoff from pervious areas, which is considerably lagged due to low catchment slopes.

Estimated 10 and 100 year ARI peak water levels at critical locations within catchments 2, 3 and 10 are provided in Table 6-6 for developed Scenario 1. The difference in water level between the existing and developed situation is also tabulated. The peak water levels are for low tailwater conditions.

Table 6-6 Estimated 10 and 100 year ARI developed peak water levels in Catchments 2, 3 and 10 for Scenario 1

	10 yr ARI			100 yr ARI		
Node	Developed peak water level (m AHD)	Critical storm duration (min)	Peak water level difference (m)	Developed peak water level (m AHD)	Critical storm duration (min)	Peak water level difference (m)
10a	4.11	90	-0.004	4.16	90	-0.022
10b	1.86	120	-0.043	2.08	120	-0.025
10c	1.77	120	-0.017	1.92	120	-0.010
10d	1.73	120	-0.006	1.87	120	-0.007
10e	1.49	120	-0.011	1.78	120	-0.005
10f	1.33	120	0.000	1.65	120	-0.002
10g (Nelson Bay Rd)	1.21	60	+0.001	1.52	120	0.000
2b (McKinley's Swamp)	2.42	540	-0.005	3.01	540	-0.037
3d (Basin 1)	3.40	540	-0.003	3.63	540	-0.002
3b (Basin 2)	4.25	720	+0.031	4.50	540	-0.018

From Table 6-6 it can be seen that developed peak water levels generally show negligible difference between the existing and developed situation.

Scenario 2

Estimated 10 and 100 year ARI developed peak flows and critical storm durations within catchments 2, 3 and 10 are provided in Table 6-7 for Scenario 2. The percentage change in developed peak flows from the existing situation is also tabulated.

Table 6-7 Estimated 10 and 100 year ARI developed peak flows and critical storm durations in Catchments 2, 3 and 10 for Scenario 2

	10 yr ARI			100 yr ARI		
Node	Developed peak flow (m^3/s)	Critical storm duration (min)	% Peak flow difference	Developed peak flow (m^3/s)	Critical storm duration (min)	% Peak flow difference
10a	0.71	90	-5%	1.16	120	-26%
10b	1.52	120	-2%	2.80	120	+11%
10c	1.98	120	+17%	3.58	120	+56%
10d	4.16	120	+26%	7.16	120	+51%
10e	4.68	120	+33%	7.62	120	+48%

Node	10 yr ARI			100 yr ARI		
	Developed peak flow (m^3/s)	Critical storm duration (min)	% Peak flow difference	Developed peak flow (m^3/s)	Critical storm duration (min)	% Peak flow difference
10f	5.32	120	+36%	7.93	360	+37%
10g (Nelson Bay Rd)	6.00	120	+28%	9.26	360	+23%
2b (McKinley's Swamp)	0.86	540	-9%	1.24	540	-32%
3d (Basin 1)	0.19	540	-1%	0.72	540	-6%
3b (Basin 2)	0.20	720	+2%	3.09	540	-16%

Peak flows in Catchment 10 generally increase between the existing and developed situation, with increases of up to +56%. This increase may be attributed to the floodway allowing floodwaters to be conveyed from the catchment quicker than for the existing situation.

Peak flows in catchments 2 and 3 generally decrease between the existing and developed situation. This can be attributed to the floodway resulting in lower water levels downstream of the detention basins in these catchments.

Estimated 10 and 100 year ARI developed peak water levels at critical locations within catchments 2, 3 and 10 are provided in Table 6-8 for Scenario 2. The difference in water level between the existing and developed situation is also tabulated. The peak water levels are for low tailwater conditions.

Table 6-8 Estimated 10 and 100 year ARI developed peak water levels in Catchments 2, 3 and 10 for Scenario 2

Node	10 yr ARI			100 yr ARI		
	Developed peak water level (m AHD)	Critical storm duration (min)	Peak water level difference (m)	Developed peak water level (m AHD)	Critical storm duration (min)	Peak water level difference (m)
10a	4.11	90	-0.029	4.15	120	-0.004
10b	1.49	120	-0.349	1.76	60	-0.409
10c	1.38	120	-0.219	1.71	120	-0.411
10d	1.33	120	-0.192	1.69	120	-0.410
10e	1.30	120	-0.125	1.66	120	-0.206
10f	1.28	120	-0.004	1.65	120	-0.049
10g (Nelson Bay Rd)	1.23	120	+0.021	1.54	360	+0.018
2b (McKinley's Swamp)	2.44	540	-0.068	2.98	540	+0.014
3d (Basin 1)	3.40	540	-0.002	3.63	540	-0.003
3b (Basin 2)	4.25	720	-0.018	4.50	540	+0.035

From Table 6-6 it can be seen that peak water levels generally decrease between the existing and developed situation. Peak water level decreases are highest in Catchment 10, with decreases of up to -0.411m. This decrease may be attributed to more efficient conveyance of floodwaters from the catchment.

The engineered floodway to the north of the Anna Bay township was sized to convey the predicted 100 year flows. The floodway comprises two typical sections, which are shown in Figure 6-1. Section ‘A’ is located from Nodes 10b to 10d and from the outlet of McKinley’s Swamp to Node 10d. Section ‘B’ is larger, and is located from Node 10d to Node 10f. The channel has a constant grade of 0.12% between Nodes 10b and Node 10f, and a constant grade of 0.23% between the outlet of McKinley’s Swamp and Node 10d. The existing floodplain and channel geometry have been modelled downstream of Node 10f (i.e. where the sand ridge crosses Fern Tree Drain).

As discussed earlier, it should be noted that predicted flood levels given above are based on low tailwater conditions. The top of bank of the floodway would therefore be overtopped for high tailwater conditions of 2.2m AHD, which occur with back-up from flooding in Port Stephens and Tilligerry Creek. Consideration should be given to raising the top of bank to contain the high tailwater condition, however, this would require additional land-take.

6.2.2 Catchment 4

Estimated 10 and 100 year ARI peak discharges from the northern and southern detention basins within Catchment 4 are provided in Table 6-9. Peak water levels and storage volumes are given in Table 6-10. The peak flows and water levels are for low tailwater conditions.

Table 6-9 Estimated 10 and 100 year ARI developed peak flows and critical storm durations in Catchment 4

	10 yr ARI		100 yr ARI	
Node	Developed peak flow (m^3/s)	Critical storm duration (min)	Developed peak flow (m^3/s)	Critical storm duration (min)
Northern basin (discharges to Anna Bay Main Drain)	1.83	540	2.32	540
Southern basin (discharges to northern basin)	2.23	540	4.17	120

Table 6-10 Estimated 10 and 100 year ARI developed peak water levels and volumes in Catchment 4

Node	10 yr ARI		100 yr ARI		
	Developed peak water level (m AHD)	Critical storm duration (min)	Developed peak water level (m AHD)	Critical storm duration (min)	Storage volume (m3)
Northern basin (discharges to Anna Bay Main Drain)	4.44	540	5.05	540	15,145
Southern basin (discharges to northern basin)	4.47	540	5.08	540	16,240

From Table 6-9 it can be seen that the predicted developed 100 year ARI peak discharge from Catchment 4 to the Anna Bay Main Drain is 2.32m^3/s (for low tailwater conditions). This is less than 3m^3/s, which was proposed in the previous flood study (Sinclair Knight, 1994).

From Table 6-10 it can be seen that there would be over 200mm freeboard between the developed peak water level in the basins and the Gan Gan Road pavement during a 100 year ARI storm event.

Storage volumes predicted in Table 6-10 represent an upper limit and may be reduced by:

- increasing the rate of discharge from the northern basin to Anna Bay Main Drain. This would require examination of the potential impact of this increased flow on existing flooding downstream;
- provision of storage higher up in the catchment; and
- allowance for infiltration in basins. Long term groundwater monitoring should be undertaken to determine maximum groundwater levels and a detailed geotechnical investigation to determine soil profiles and existing infiltration rates.

50% Blockage Scenario

The XP-SWMM model was also run for 50% blockage of the northern basin outlet. Estimated 10 and 100 year ARI developed peak discharges, water levels and storage volumes for the 50% blockage scenario are provided in Table 6-11 and Table 6-12. Peak discharges, water levels and storage volumes are for low tailwater conditions.

Table 6-11 Estimated 10 and 100 year ARI developed peak flows and critical storm durations in Catchment 4 for 50% blockage

Node	10 yr ARI		100 yr ARI	
	Developed peak flow (m^3/s)	Critical storm duration (min)	Developed peak flow (m^3/s)	Critical storm duration (min)
Northern basin (discharges to Anna Bay Main Drain)	1.19	540	1.72	540
Southern basin (discharges to northern basin)	2.13	540	2.89	720

Table 6-12 Estimated 10 and 100 year ARI developed peak water levels and volumes in Catchment 4 for 50% blockage

Node	10 yr ARI		100 yr ARI		
	Developed peak water level (m AHD)	Critical storm duration (min)	Developed peak water level (m AHD)	Critical storm duration (min)	Storage volume (m3)
Northern basin (discharges to Anna Bay Main Drain)	4.60	540	5.27	540	18,085
Southern basin (discharges to northern basin)	4.61	540	5.29	540	19,200

From Table 6-12 it can be seen that for 50% blockage of the northern basin outlet, Gan Gan Road would not be overtopped during a 100 year ARI storm event.

Due to the absence of a formal overflow path from Catchment 4, it is recommended that finished floor levels be set a minimum of 500mm above the estimated peak water level for 50% blockage of 5.3m AHD (i.e. 5.8m AHD).

7. Conclusions and recommendations

Recommendations for the study area include provision of:

- rainwater tanks on all new allotments for the collection and re-use of roof runoff;
- infiltration trenches or soakage areas on all new allotments for the collection and infiltration of rainwater tank overflows and runoff from other allotment areas;
- a constructed drainage system from Catchment 4 to Anna Bay Main Drain, with maximum 100 year flow limited to 2.32m^3/s;
- detention storage within Catchment 4 to capture and attenuate stormwater runoff;
- a pit and pipe network to capture and convey localised runoff from all future developed areas;
- vegetated swales and unlined channels to convey runoff; and
- an engineered floodway to the north of Anna Bay to convey 100 year flows from the study catchment.

Peak water levels in catchments 2, 3 and 10 decrease following implementation of the proposed management measures. Peak water level decreases are highest in Catchment 10, with decreases of up to -0.411m. This decrease may be attributed to more efficient conveyance of floodwaters from the catchment. Peak flows in Catchment 10 generally increase as a result of development. This increase is due to the floodway allowing floodwaters to be conveyed from the catchment quicker than for the existing situation. It may be necessary to provide offline detention storage within Catchment 10 to attenuate peak flows to existing levels and not impact downstream flood levels.

Developed modelling undertaken for this investigation is based on the development areas identified in the Structure Plan. Further detailed modelling will need to be undertaken by each developer to quantify trunk drainage requirements once development layouts are formulated.

8. References

Institute of Engineers Australia, Reprinted 2001. *Australian Rainfall and Runoff – A Guide to Flood Estimation*.

Port Stephens Council, 2002. *Main Drainage Relief Scheme – Anna Bay* engineering drawings (file no. E5105-06, plan no. D-102).

Sinclair Knight and Partners Pty Ltd, 1994. *Anna Bay Catchment Drainage/Flood Study Masterplan* (for Port Stephens Council).

Figures

Client: Port Stephens Council
Project: Drainage Investigation Report
Location: Anna Bay North
Structure Plan

PARSONS BRINCKERHOFF

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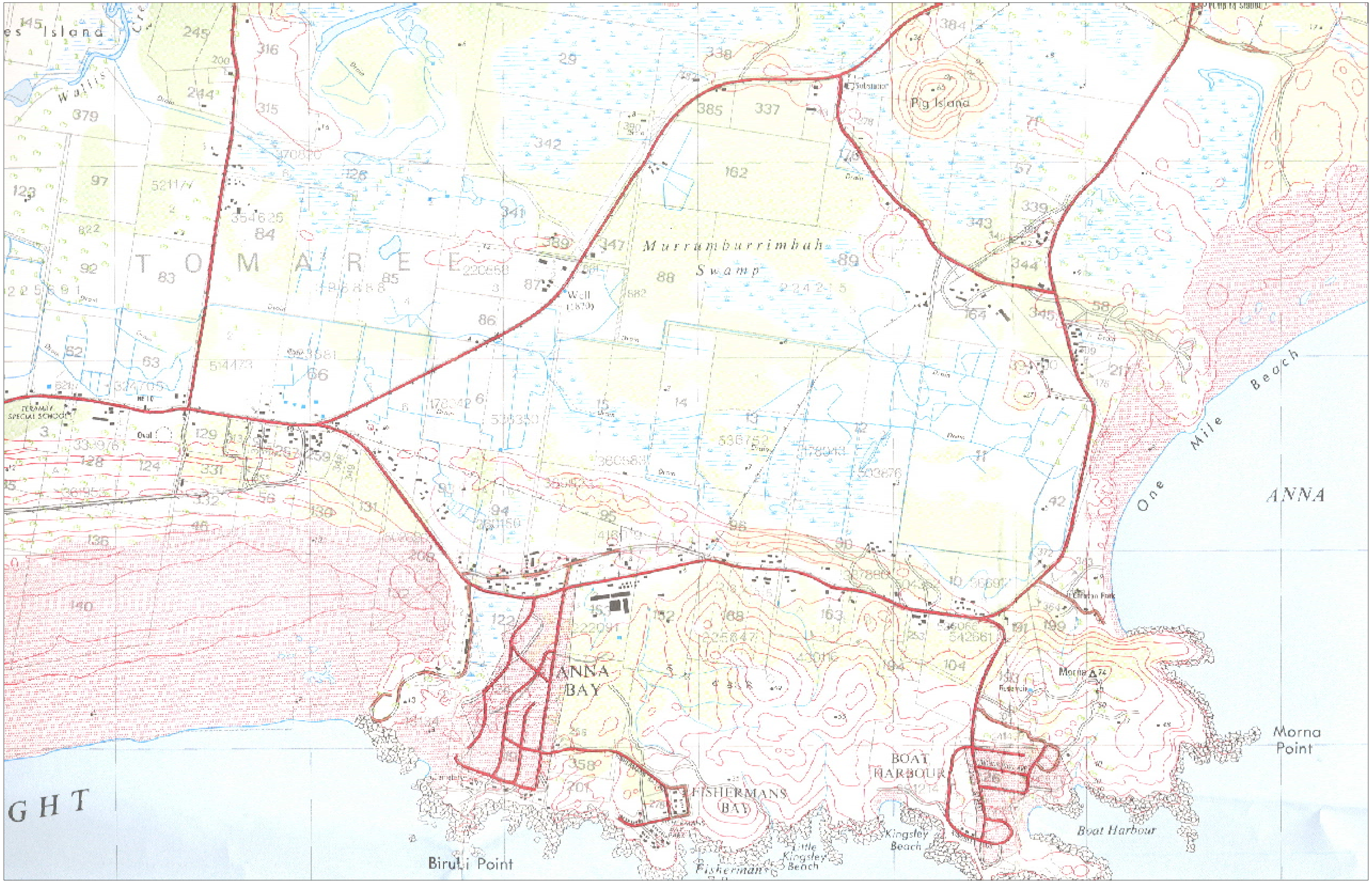
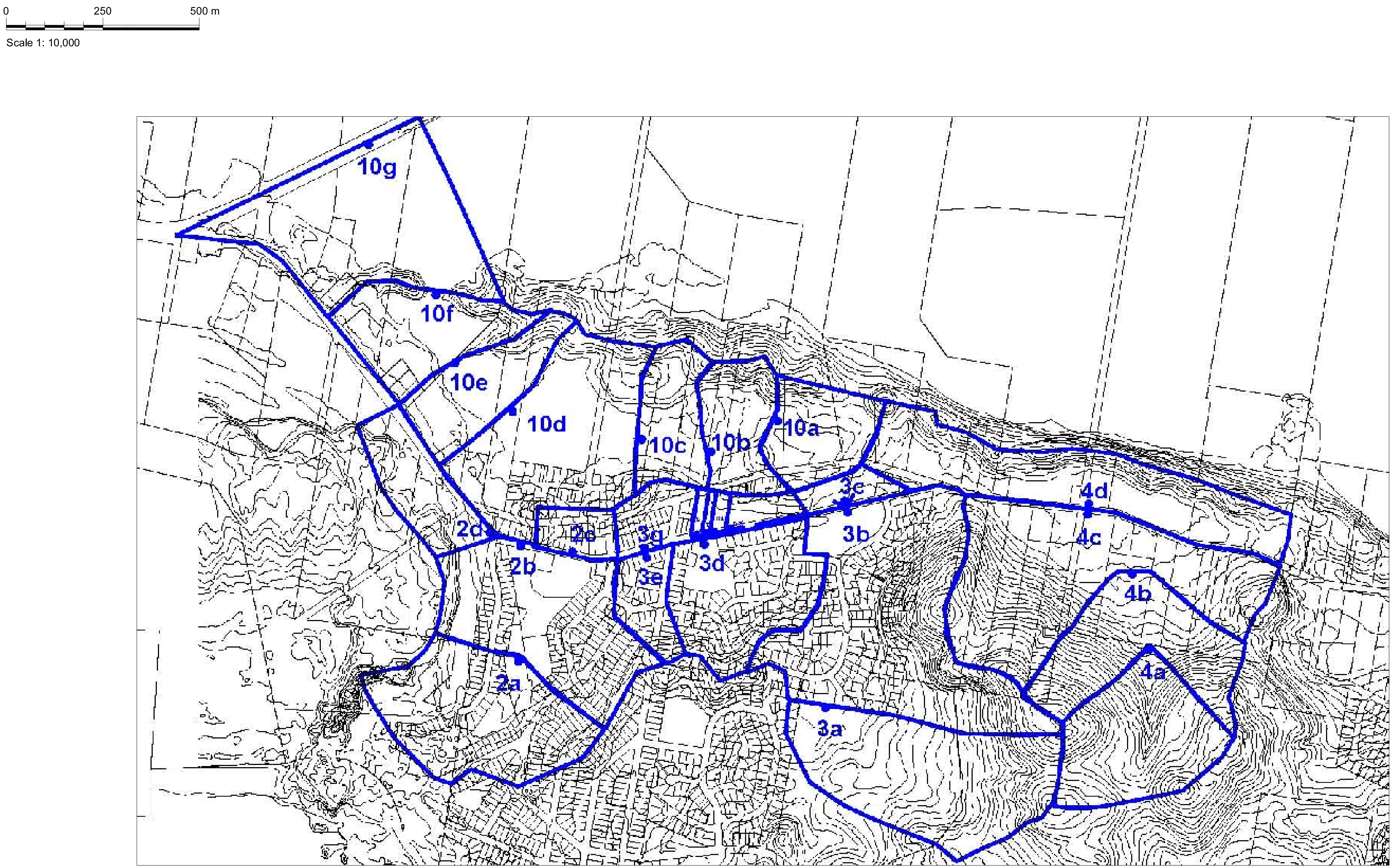
Scale 1: 25,000



Locality Map
Figure 1.1

REPORT: 2122425A\PR_0894
CAD REFERENCE: E:\2122425A\CAD\Pr_0894\FIGURE1.1.DWG

Client: Port Stephens Council
Project: Drainage Investigation Report
Location: Anna Bay North
Structure Plan



PARSONS BRINCKERHOFF

Existing Catchment Plan
Figure 1.2

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Client: Port Stephens Council
Project: Drainage Investigation Report
Location: Anna Bay North Structure Plan

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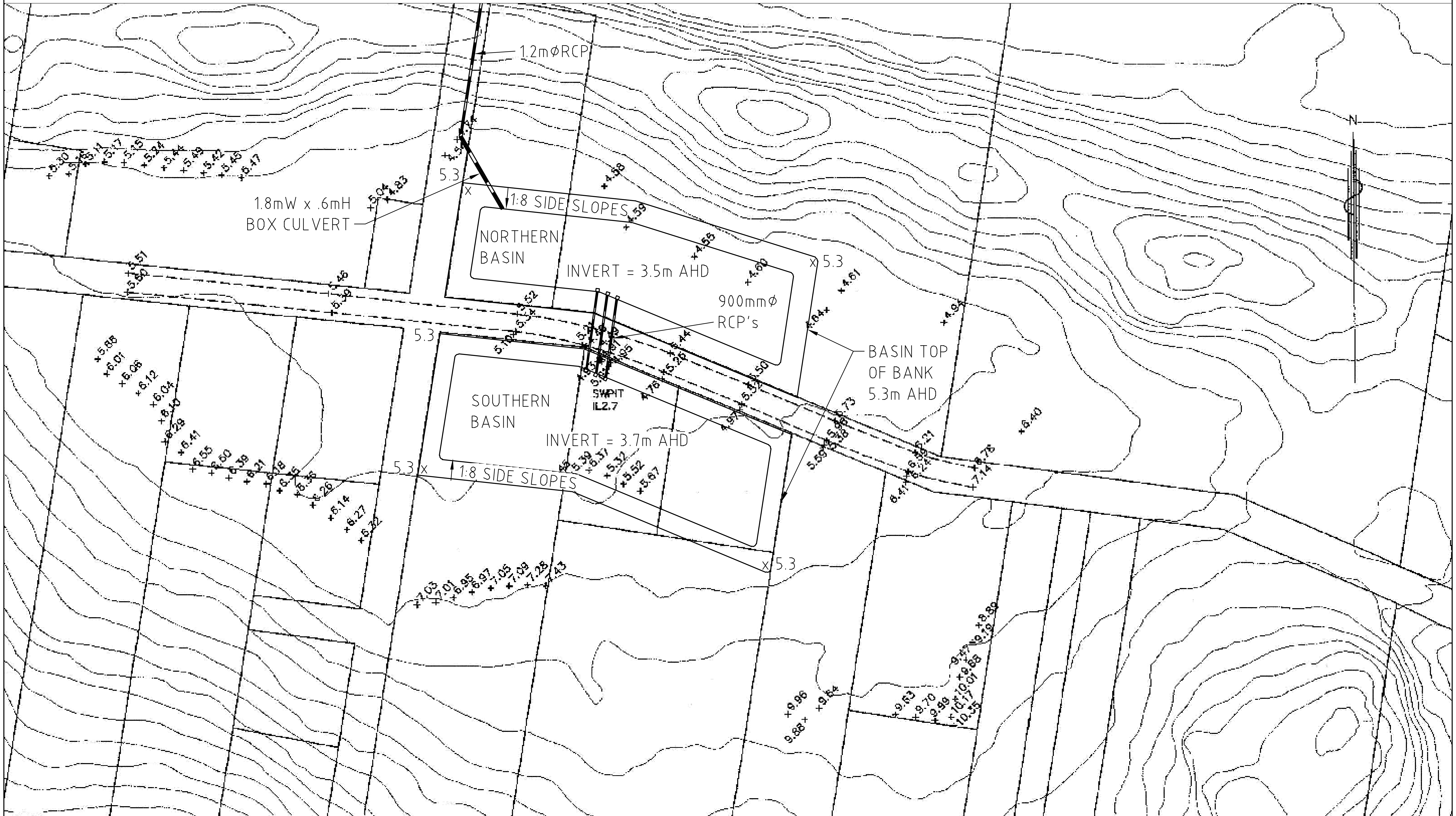

Extent of proposed detention basins within Catchment 4

Figure 5.1

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Client: Port Stephens Council
Project: Drainage Investigation Report
Location: Anna Bay North Structure Plan

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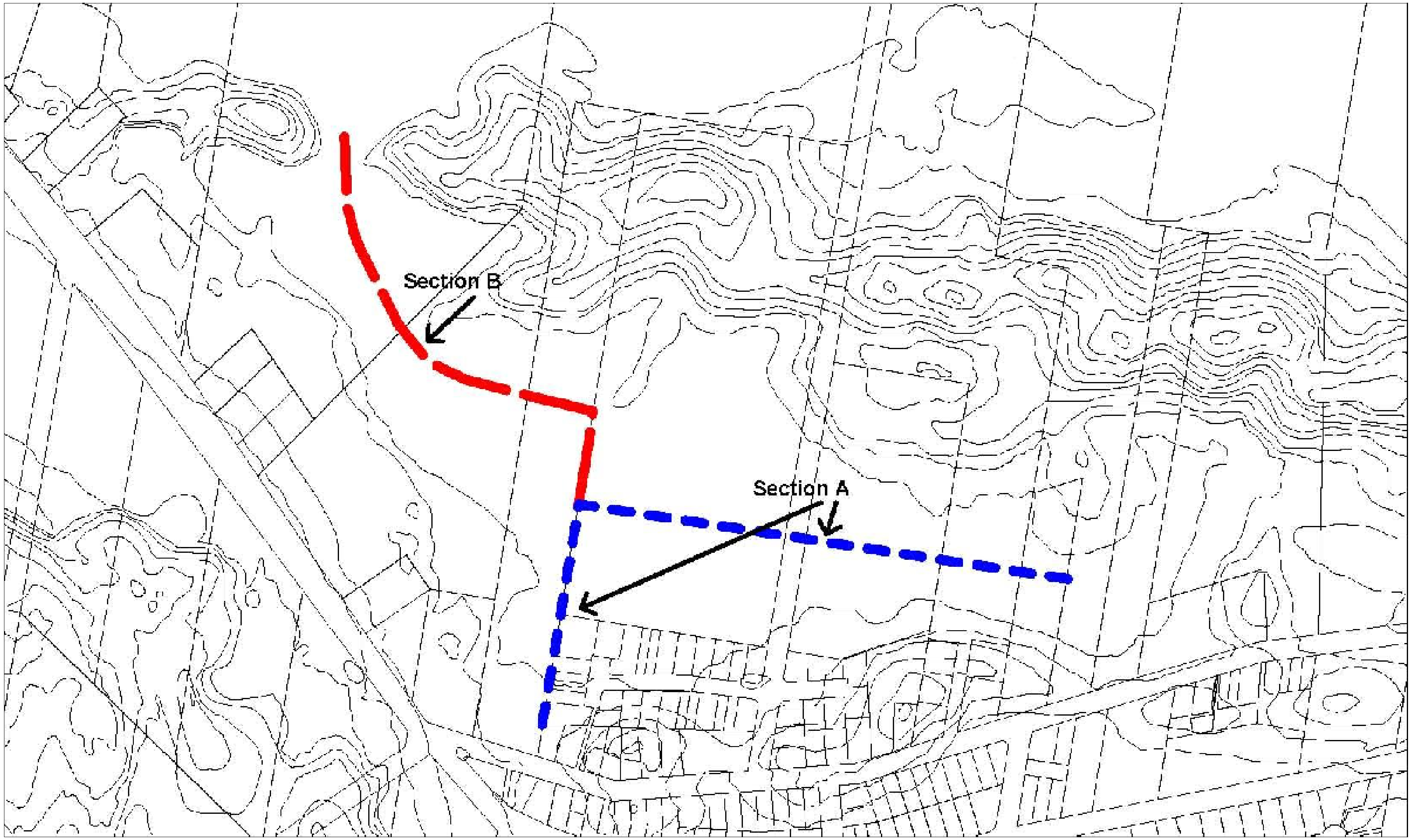

Scenario 2
Figure 5.2

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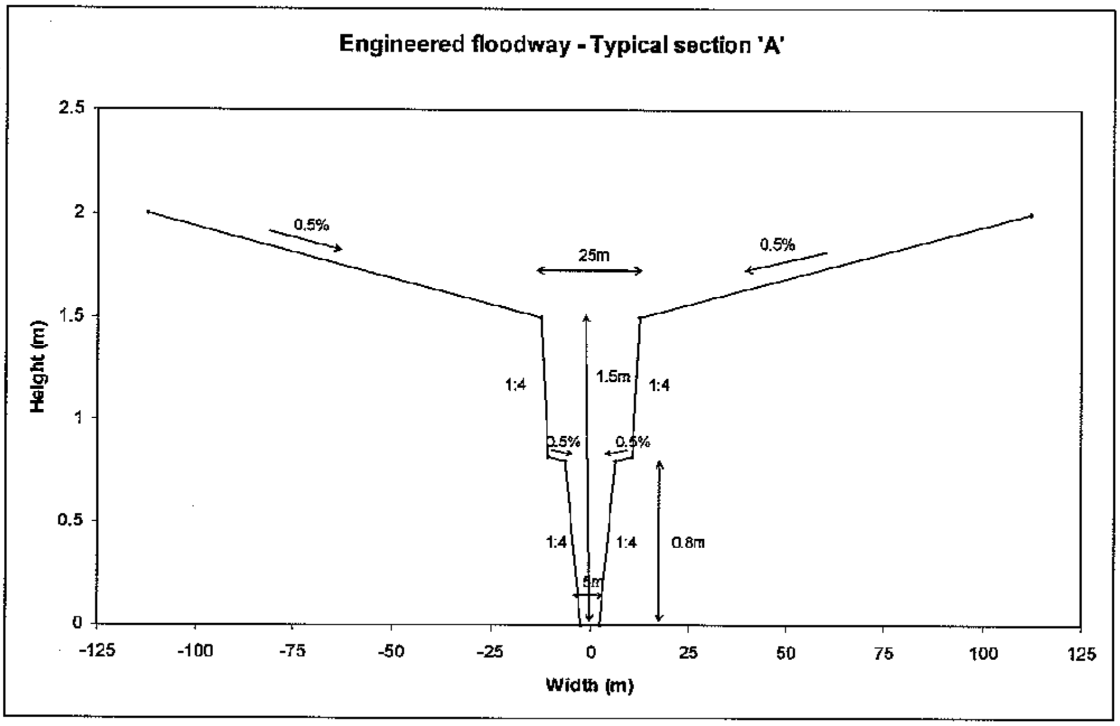
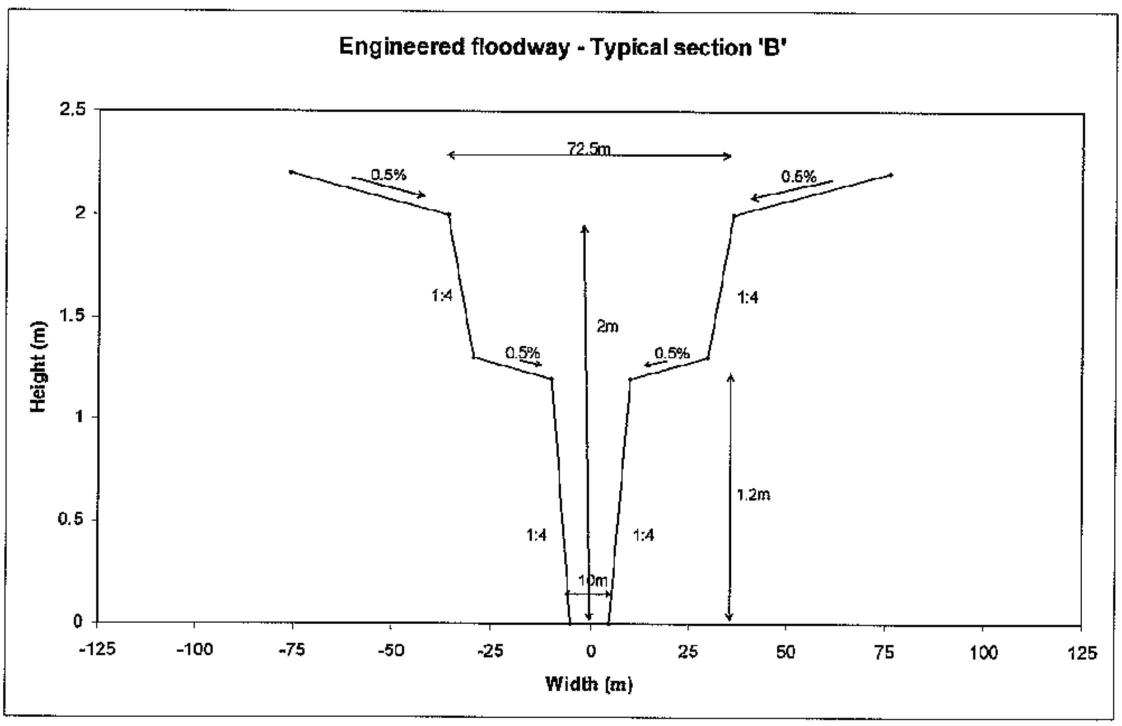
Project: Drainage Investigation Report
Location: Anna Bay North
Structure Plan

PARSONS BRINCKERHOFF





Typical sections of Scenario 2 floodway
Figure 6.1