STORMWATER CATCHMENT MANAGEMENT PLAN

PORT WAIKATO TOWNSHIP

FINAL REPORT

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EXECUTIVE SUMMARY

This Catchment Management Plan has been prepared for the Port Waikato Township Catchment within the Franklin District Council. It recommends options for improving and managing stormwater flooding, water quality and wastewater treatment and disposal.

The questionnaire survey indicates that the majority of responses reported no flooding problems (69%), or attributed flooding to runoff from roads as well as from neighbouring properties.

A stormwater hydrological and hydraulic model of the Main Drain drainage system covering a catchment area of 63ha was developed using the MOUSE modelling package (TP108 module) to identify flood hazards for the existing drainage system under Existing Development and the Maximum Probable Development landuse scenarios.

Flood hazard maps show that for future development about 27% of the catchment would be flooded during a 100 year ARI storm event. Also, due to its flat nature a large portion of the central low lying floodplain would be flooded even in 2 and 5 year ARI storm events.

Analysis of Existing and Maximum Probable Development landuse scenarios shows that an increase in stormwater flow due to proposed development (as per Franklin District Plan) in the area would not make a significant difference to the extent of the flood hazard areas.

Sensitivity analysis of various pumping rates indicates that they have a minor impact on the flood levels as this is determined by the relationship between the runoff volume and the storage characteristics of the large central low-lying flat area.

Model results indicates that due to low conveyance capacity of the culverts a number of properties are flooded frequently along Maunsell Road, Mission Road, Centreway Road and Cordyline Road.

A number of stormwater flooding and water quality problems have been identified within the study area, mostly in terms of inundation caused by poor reticulation capacity and the low standing ground levels.

Several remedial options including structural and non-structural measures are recommended to mitigate the existing stormwater flooding and water quality problems and also for the future development within the study area. However implementation of the preferred option would require further investigation and evaluation.

This Catchment Management Plan is expected to allow meaningful consultation with stakeholders including iwi, developers, potentially affected landowners and the public generally.

1. INTRODUCTION

1.1 Background

Franklin District, split between the Auckland Region and the Waikato Region, is one of the fastest growing districts in New Zealand. Pressure for subdivision is occurring in rural and coastal areas. The subsequent change of land use resulting from subdivision has the potential to have adverse effects on the environment.

In recognition of these growth pressures, a plan change to better guide future development was publicly notified in May 2004 by Franklin District Council (FDC) (District Plan change 14-Rural). The purpose of the Plan Change was to provide a regime of strategies, objectives and rules that better address the current issues facing the District, particularly those relating to the management of growth, occurring in rural and coastal areas of Franklin. The Plan Change background material states "the extension and consolidation of existing villages and infrastructure could more effectively support existing services and local economies to provide better economies of scale for new or additional infrastructure development".

Although the Port Waikato area has not been identified as an area where growth has been specifically allowed for, Port Waikato Township is likely to come under growth pressure in the future. It has been recognised that the settlement is moving from providing holiday accommodation to the provision of more permanent accommodation resulting in different pressures on infrastructure and a different expectation of service. It is important to understand the constraints of infrastructure service so that development can be guided in a sustainable matter.

Franklin District Council commissioned City Design Limited (CDL) in May 2004 to develop a Catchment Management Plan (CMP) for the Port Waikato Township. Port Waikato settlement lies predominantly on low-lying ground at the base of the substantial Waikato Heads sandspit (refer to Figure 1). Many of the low-lying properties are prone to flooding which results in flooding of septic tanks and soakage trenches. This can result in contaminant discharge into the receiving environments and affects stream habitat.

1.2 Scope of Works

The main objective of this study is to address the flooding and stormwater quality issues and develop long term solutions for flood and stormwater management in the Port Waikato Township area.

The scope of works for this project is to prepare a Catchment Management Plan in sufficient detail to:

- Guide future development,
- Develop workable flood protection, water quality and other infrastructure work concepts including indicative costs,
- Allow meaningful consultation with stakeholders including iwi, developers, potentially affected landowners and the public generally.

This CMP has been developed taking into account FDC stormwater policies and objectives and Franklin District Code of Practice for Subdivision and Development (1).

1.3 FDC Objectives and Policies Relating to Stormwater

There are no specific objectives and policies that relate to stormwater in coastal village settlements such as Port Waikato. However, the following objectives contained in Franklin District Plan Change 14 (Rural Plan Change) are broadly relevant in that they seek to guide the management of growth and resources in a sustainable manner.

17.1 Main Objectives applying to all rural and coastal parts:

3. Provide for the sustainable growth of the District at appropriate rural and coastal village settlements through zoning, structure planning and appropriate subdivision and land use controls.

6. Maintain and enhance the quality and quantity of water resources.

7. Preserve and enhance remaining indigenous ecological resources and enhance their contribution to biodiversity, landscape and amenity values.

9. Recognise and provide for enhancement of those landscape values that contribute to the sense of enjoyment and appreciation of living in rural and coastal areas.

11. Protect and preserve the taonga of Tangata Whenua.

17.3.3 outlines Village growth Policies, one of which is to

3. Ensure that private and public services are provided and maintained in a sustainable way.

1.4 Level of Service

Any new stormwater improvements for the area are currently required to be designed in accordance with the Franklin District Code of Practice for Subdivision and Development (1).

In particular the following specifications are relevant to this CMP:

- The soakage system shall be capable of containing a 5% AEP (20 year ARI) 10 minute storm without overflowing and completely soak away within 24 hours.
- Initial time of concentration for design purposes shall be 10 minutes (0.037 mm/sec).
- Secondary flow paths shall be provided for 1% AEP (100 year ARI) flows in which case the primary design flow shall be based on the following minimum storm frequencies: Residential, Commercial & Industrial areas – 20% AEP (5 year ARI). Auckland Regional Council (ARC) Technical Publication No. 108 "Guideline for the estimation of flood flows in the Auckland Region" (2) shall be used for calculating peak flows in rural catchments.
- Major watercourses should be retained and be located together with secondary 1% AEP (100 year ARI) overload flow paths in public reserves.
- A minimum velocity of 0.7m/s for pipes when flowing full is desirable.
- In potentially unstable ground sewer pipes should be specifically designed.

2. Catchment Description

2.1 Location

Port Waikato Township is located at the mouth of the Waikato River on the southern margin of a spit which comprises a river delta-sand dune complex (refer to Figure 1). The higher ground to the south of the catchment is steep whereas the low-lying central reserve area is flat and below tide levels.

Topography varies from relatively steep tributary valleys at a maximum elevation of 135 metres above mean sea level in the southern upper catchment areas to less than 2 metres in the low-lying central reserve floodplain areas.

2.2 Geology

2.2.1 Geological Structure

The Port Waikato area is underlain by consolidated siltstones and sandstone's of the Apotu Group which represent an old sequence of rocks (Upper Jurassic Age: 142 million years) faulted to the surface by regional tectonics.

Overlying the siltstones in the southern portion of the study area, in an unconformable manner, are partially consolidated/cemented dunes sequences of the Awhitu Group (1.8 million years) which form distinct hilly areas. This material is an extension of the dune sands exposed along the Awhitu Peninsular. In the north of the area unconsolidated black iron sands of recent origin (< 800 years old) form a low-lying topography and appear to lie directly on the older siltstones.

Bore logs from across the area have been gained from both Environment Waikato and FDC. These logs illustrate that across most of the low-lying area it is expected that unconsolidated iron sands will be in the order of 3-5 meters thick. Directly below sands are expected to be the consolidated siltstones of the Apoutu Group forming a generally horizontal surface. No additional bore logs were considered essential for this study.

The underlying Apoutu Group appears to represent a shore platform erosion period that is now covered with the recent iron sands, deposited in either an inter-tidal setting or directly onshore. The upstanding Awhitu sands at the rear of the study area are an older sequence of deposits and lie along a major fault trace in the underlying basement mudstones. This position appears to be the most inland extent of coastal erosion.

Groundwater movement within the iron sands can be expected to be dominantly within the unconsolidated black sands due to the low permeability of the underlying sediments. Also the limited thickness of the sands will mean that saturation in extreme rain events (intensity or duration) will occur.

2.2.2 Geological Hazard Profile

Based on the geology it is clear that the Port Waikato coastal village has been developed over an area where significant coastal erosion has occurred in the past.

While there is little evidence that the fault traces evident on the Apotu Group rocks are likely to be active (movement in the last 100 years) it is however probable that current coastal processes are dynamic. Accretion seems to be occurring at this time however this may not be prevalent in the long-term. Erosion is just as likely to occur particularly in extreme weather conditions and with the likely scenario of sea level rise even more so.

In-line with the existence of major faulting in the area and the low-lying nature of the development in an area of dynamic coastal activity. Investment in either stormwater or wastewater management infrastructure through the area needs to be considered in terms of location. Large engineering structures are likely to be exposed to high risk in coastal erosion times. Low technology engineering would best suit the unconsolidated and porous sands.

2.2.3 Geological Issues Related to Flooding Potential

Early geomorphic maps of the area also indicate that the area was extremely swampy with a lake system existing within the central area now forming the current wetland and drainage channel. The standing water level appears to lie within the thinnest areas of unconsolidated iron sands and reflect the impermeable lower siltstones.

Flooding of the area during extreme rainfall conditions would therefore be expected to occur in those zones where the previous lake feature existed. Development should therefore avoid this zone.

2.2.4 Geological and Soil Hydrological Characteristics.

Both the flat zone (iron sands) and hilly areas (Awhitu sands) at the rear of the catchment are likely to have high infiltration potential. This is due to the high percentage of sand and porosity. Those limited zones where the underlying siltstones are exposed will have high runoff and limited infiltration potential. Assumptions on infiltration rates for stormwater modelling purposes are provided in Section 3.3.

2.3 Landuse and Development

2.3.1 Current Development

The Port Waikato Catchment is predominantly a coastal village containing a mixture of low to medium density residential developments.

2.3.2 Future Development

Rural Plan Change 14 (Rural) recognises Port Waikato as having limited future development potential.

> *"This area of predominantly holiday homes is located in an area subject to the constraints from three sides:coastal erosion, river erosion and movement, and adjacent hills. It has more or less reached its full development potential on the existing sites, and for this reason, expansion has not been provided for in the Proposed Plan Change 14." Pg 100.*

The land use zonings as detailed in the Franklin District Plan (Plan Change 14) are shown in Figure 2 and summarised in the following table.

Table 2.1: Landuse Zones in the Port Waikato Catchment

Description	% Zoned Area in Modelled Catchment Area
Coastal Village	34.7
Coastal Zone	9.5
Recreation Zone	12.5
Rural Zone	34.5
Roads	8.8

The majority of lots zoned Coastal Village are at their maximum development except for an area up the hill from the beach off Maunsell Road and a group off the end of Westside Road adjacent to the sand dunes.

2.4 Catchment Hydrology

The overall three-water structure for the Port Waikato Township consists of rainwater tanks for potable supply, septic tanks and ground soakage trenches for wastewater disposal and a combination of ground soakage and drainage reticulation networks for stormwater. There are a number of groundwater bores that are utilised for emergency potable and irrigation supplies. There is a reticulated potable water supply with limited capacity. A number of sewage disposal systems are vulnerable to flooding, while others have "failed" as result of being located in areas with a high ground water table (3).

The Port Waikato Catchment maintains a northerly aspect. It is comprised of three valley systems, which slope steeply down from its southern boundary to flat areas, which in turn gently slope towards the receiving environment. Two main watercourses, Maraetai Stream from the south and the Main Drain running eastwards, discharge into the Waikato River at Maraetai Bay (refer to Figure 3).

The main stormwater drainage system is generally comprised of ephemeral streams within the valleys in the upper catchment leading to engineered watercourses and pipe networks in the flat parts of the catchment. The low-lying flat areas of the catchment have high water tables and poor drainage. This has resulted in the engineered watercourses being constructed to assist in drainage.

The engineered watercourses can be characterised as straight unlined trapezoidal shaped channels with, flat soft sedimented bottoms, slow flow and no riparian vegetation. The drainage capacity of the watercourse is generally well maintained. During the site visit there was evidence of a proliferation of certain aquatic weeds and algae.

The lateral systems are comprised of small v-drains and pipes. There are several ponding areas located within the network. Pipe systems within the network generally consist of concrete pipes ranging from 225mm to 600mm in diameter. Pipes function to connect sections of open channels and act as culvert units to allow traffic access across the roadside v-drain/channel system.

V-drains are commonly shallow and grass covered and often located within the road reserve. Some roads have kerbs and channelling to direct flow to the drainage system and these are generally located on main roads or in newly developed areas within the catchment.

A low lying swampy wetland area is situated to the west of Maunsell Road and extends east towards Ashwell Drive. Groundwater levels are at or very close to the ground surface and it is likely that significant organic deposits are present.

The wetland area consists of a large area of raupo and more open areas containing a mixture of gasses, sedges and muehlenbeckia. Weeds such as ginger, blackberry, pampas, and gorse present.

Within the wetland area is a watercourse which flows south west to meet the main channel. The main watercourse then flows west of Ashwell Drive at the base of the rear of the sand dunes towards Maraetai Bay.

The Main Drain drainage system leads to a Pump Station. The Pump Station has two pumps discharging to an outlet channel which flows a short distance to the Waikato River. The Pump Station is at a level above the general range of Waikato River tidal and flood levels.

2.5 Stormwater Catchments

A questionnaire survey was undertaken in June 2004, which indicates that the Maraetai Stream Catchment has some localised flooding problems near its mouth in the vicinity of Stack Road and Maraetai Place. This is of relatively minor concern and therefore Maraetai Stream catchment was not analysed in detail. Most of the existing problems with the stormwater system appear to be associated with the catchment of the Main Drain drainage system.

Contour, property boundaries, channel and pipe drainage asset data in GIS format provided by FDC was used to undertake delineation of sub-catchments of the Main Drain drainage system.

The total catchment area of the Main Drain system is approximately 63ha which comprises 27 sub-catchments. Figure 3 shows the layout plan of the sub-catchments.

2.6 The Receiving Environment

Port Waikato is part of the Meremere Ecological District bordering the lower Waikato River. This district is characterised by alluvial flats, swamps, shallow lakes, (including Whangamarino and Mangatawhiri wetlands) containing mostly river and swamp deposits. The ecological district has undergone early vegetation clearance by Polynesians. More recent activities include open cast coal mining, farming, orchards.

The southern most waterway in the Port Waikato Township discharges directly to the West Coast at Sunset Beach. Water samples collected at the beach by Environment Waikato met relevant guidelines (4). Sunset beach is typical of many West Coast beaches in the Waikato/Auckland Region. The beach is a high energy system with the coast being exposed to the prevailing weather patterns. A beach such as this has good flushing characteristics.

The main watercourse is pumped to the base of the Waikato River and discharges at the west end of Phillips Reserve at the western extent of Maretai Bay. The Maraetai Stream discharges to the east of the Bay.

Maraetai Bay is part of the Port Waikato Estuary. This tidal area at the mouth of the Waikato River is sheltered in the west by the large expansive sand dune system creating the feeling of a beach like bay.

The area around Maraetai Bay and the Port Waikato estuary is an important feeding and breeding area for birds. During the site visit the following birds were observed: kingfishers herons, stilts, and shags. The nearby sand island is prime breeding habitat for regionally important bird species such as the Caspian Tern and NZ Dotterels.

In addition to bird life the estuary area around Port Waikato provides important habitat to a variety of native fish such as kahawai, yellow-eyed mullet, black flounder and the longfinned eel.

Samples collected at Maraetai Bay by Environment Waikato (4) have exceeded their guideline of 280cfu (colony-forming unit) per 100ml for Enterococci. The water quality of the Bay is likely to reflect the quality of the Waikato River as well as local inputs from the Port Waikato Township.

3. MODEL DEVELOPMENT

A stormwater hydrological and hydraulic model of the Main Drain drainage system was developed in order to assess catchment design flows, define network capacity and identify flood hazards for the existing reticulation system. The catchment boundary for the model is shown in Figure 3. Model simulations were carried out for 2, 5, and 100 year ARI storm events for Existing Development (ED) and Maximum Probable Development (MPD) conditions. Details of this process are outlined below.

3.1 Review of Existing Stormwater Asset Data

Asset data was provided by FDC in GIS Format. The data included a considerable amount of surveyed asset information. Additional channel cross-section details were collected by *CITY* DESIGN Ltd's Survey Team.

Figure 4 summarises the locations of the surveyed spot heights with associated data density zones. Internal Quality Assurance and Quality Control (QA/QC) of the data acquisition process was undertaken.

The Pump Station, located at the end of the Main Drain, has two pumps discharging to the Waikato River. No definite information was available regarding the pump operation and pump capacity. The following information regarding pump specification was provided by FDC and Mechanical NZ Limited (who are maintaining the Pump Station):

- One 3" submersible pump is currently working with a capacity of approximately 4.5 m^3/min (0.075 m^3/s).
- The second 9/10 axial pump with a capacity of approximately 0.15 m^3/s is situated at a higher level. This pump is not working effectively at present and FDC are planning to lower the pump intake.
- The submersible pump turns on when water depth within the pump station reaches a depth of approximately 900mm and it turns off when the water level drops by approximately 300mm. In terms of LINZ levelling datum the operating levels of the submersible pump are approximately 1.7m RL (on) and 1.4m RL (off).
- The axial pump intake is situated approximately 150mm above the level the submersible pump turns on. Considering that the axial pumps require generally about 400-500mm depth of water below and above the pump intake before they operate efficiently,

the existing axial pump turns off approximately 600mm above the level that the submersible pump turns on. In terms of LINZ levelling datum the operating levels of the axial pump are approximately 2.6m RL (on) and 2.3m RL (off).

- The operating levels of the axial pump are found to be above the 100-year ARI flood level (refer to Section 5) and thus the axial pump is assumed to be not working at present.
- The axial pump intake should be lowered by about 550mm, that is to 1.3m RL (above 500mm depth of water within the pump station). The new operating levels of the axial pump should be 1.9m RL (on) and 1.7m RL (off). Model simulations were carried out for ED and MPD landuse scenario with this proposed new operating levels assuming that the axial pump intake is lowered at 1.3m RL and working.

The details of the existing two pumps including the proposed new pump operating levels are provided in Table 3.1.

Table 3.1: Existing Pump Capacity and Operating Levels

Pumps	Capacity	Existing Operating levels		Proposed New Operating levels	
		Start Level	Stop Level	Start Level	Stop Level
3" Submersible Pump	0.075 m^3/s	1.7m RL (900mm)	1.4m RL (600mm)	Same as existing	Same as existing
9/10 Axial Pump	0.150 m^3/s	2.6m RL (1800mm)	2.3m RL (1500mm)	1.9m RL (1100mm)	1.7m RL (900mm)

Note: Values within the brackets are depth of water in the pump station.

Sensitivity analysis of the pumping rates was undertaken to assess their effects on the depth and duration of flooding within the catchment. Brief discussions are provided in Section 4.3.

3.2 Overview of Model

A combined hydrological and hydraulic model of the Port Waikato Catchment was constructed using the DHI MOUSE (Version 2003B) modelling software package. MOUSE is an advanced, powerful, and comprehensive surface runoff, open channel flow, pipe flow, water quality and sediment transport modelling package utilised for urban drainage systems, stormwater sewers and sanitary sewers.

3.3 Hydrological Model

3.3.1 Method Used

For this investigation runoff was modelled following the guidelines outlined in the ARC Technical Publication No. 108 document (2). The key features of this are:

- A standard 24 hour temporal rainfall pattern, having peak rainfall intensity at mid-duration. Shorter duration rainfall bursts with a range of durations from 10 minutes to 24 hours are nested within the 24 hour temporal pattern,
- Runoff depth calculated using SCS (Soil Conservation Services) rainfall-runoff curves, with curve numbers determined from the SCS guidelines according to classifications assigned to Auckland soil types,
- Runoff hydrograph calculated using the standard SCS synthetic unit hydrograph,
- Time of concentration estimated using an empirical lag equation derived from a regression analysis of data from the Auckland Region.

3.3.2 Catchments

The hydrological component of the MOUSE model represented the Port Waikato Catchment as 27 sub-catchments connected to nodes within the hydraulic model. Catchment slope was calculated using the Equal Area Method as outlined in ARC TP108. Figure 3 presents the sub-catchments boundary delineation and modelled stormwater network.

3.3.3 Hydrological Parameters

The TP108 methodology uses SCS Curve Numbers (CN) to represent runoff potential. Curve numbers consider both permeability of surface soil as well as land use. For this study the upper hilly sub-catchments (1 to 4) were considered to be Group B to C soils having low infiltration rates and comprising soils with higher percentage of silts. All other sub-catchments were considered to be Group B having moderate infiltration rates and comprising soils with higher percentage of sands.

The SCS curve numbers were lowered to account for infiltration through soakage pits. The SCS curve numbers were decreased such that a maximum 10% reduction in runoff volume occurred due to losses through soakage pits. This approach provides a uniform distribution of rainfall losses due to soakage pits across the area. The SCS curve numbers for the upper hilly sub-catchments (1 to 4) were not lowered, as there are no soakage pits within these sub-catchments. The SCS curve numbers associated with various land uses are given in Table 3.2.

Table 3.2: SCS Curve Numbers for Various Land Use Soils

Land Use	SCS Curve Number
Impervious Area	98
Bush (Group B to C) For sub-catchments 1 to 4	63
Lightly grazed pasture (Group B) For other sub-catchments	55

The Proposed District Plan indicates that no more than 35% of a site should be covered in buildings for MPD. It was assumed that 10% of the remaining area would be driveway and feeder roads would take up another 10% of residential areas. Therefore a total of 55% impervious surface area was considered appropriate for residential areas and used in analysis for the MPD scenario. For the ED scenario a maximum total of 35% impervious area was considered for residential areas including driveways and feeder roads based on visual assessment from the aerial photos.

The TP108 SCS Method allows one parameter to account for Initial Loss and this is the Initial Abstraction Parameter (Ia). For this study the Ia has been set between a range of 0 and 5 depending on the subcatchment characteristics. The value 0 is used for impervious, 5 for pervious and a weighting is applied depending on the percentage of impervious area.

3.4 Hydraulic Model

3.4.1 Method Used

The hydraulic model of the study area was developed incorporating the existing stormwater network, open channels, overland flow and off-channel storage. The computation is based on an implicit, finite difference numerical solution of basic 1-D free surface gradually varied unsteady flow equations (Saint Venant).

3.4.2 Nodes

MOUSE model nodes were utilised to represent the stormwater system attributes. Node inverts were obtained from contour and survey data. Node losses for manholes and culvert inlets were assigned as total head loss coefficient of 0.10 in accordance with the Culvert Manual (5). Channel cross-section nodes were assigned as the default 'No Cross-sectional Changes'.

3.4.3 Network

The modelled stormwater network was developed from surveyed asset data. As required, some information was interpolated between known levels. Streams and overland flowpaths were modelled as open channels based on surveyed cross-sections interpolated from both survey and contour data. The node and cross-section configuration was developed to replicate channel conveyance and storage.

There are many driveway culverts within the drainage system and generally it was considered that these do not impact on flood event flow characteristics. This is based on the logic that once the pipe capacity is exceeded, generally, the overland flow will overtop the pipe and re-enter the channel on the downstream side of the pipe.

A Manning's n value of 0.03 and 0.013 were used to account for channel and concrete pipe roughness respectively.

3.4.4 Pump Station

Pumps are assigned in the MOUSE model as head-discharge relationship. Two existing pumps were modelled and head-discharge relationships were developed based on the capacity of the pumps as specified in Table 3.1. The operation of the pumps, start and stop levels were assigned as given in Table 3.1

3.4.5 Ponding and Storage

There are significant ponding areas within the Port Waikato Catchment drainage network. Topographical contours, survey data and site inspection information were used to help identify extents of significant ponding areas. Ponding area and volume were represented in the model using representative nodal/cross-section and basin node configurations.

3.5 Boundary Data

3.5.1 Rainfall Data

The standard TP108 rainfall profile was adopted and utilised following the rainfall derivation procedure outlined within the TP108 guidelines. The daily design rainfall depth for Port Waikato was estimated from the Design Rainfall Maps prepared by Beca Carter

Hollings & Ferner Ltd (6). Rainfall profiles were developed for 2, 5, and 100 year ARI TP108 design rainstorms.

3.5.2 Tidal Data

A constant Mean High Water Spring (MHWS) tidal boundary condition was utilised for all simulations. However this boundary condition had no effect on the model simulation results as outflow is controlled by the pumps. The MHWS and Mean Sea Level (MSL) at the Waikato River Entrance are 3.7m and 2.1m Chart Datum respectively (from LINZ web site). Tide tables list the height of tide above Chart Datum, which is a water level so low that the tide will but seldom fall below it. As no information was found relating Chart Datum to LINZ levelling datum, the MSL is assumed as 0.0 m RL and the corresponding MHWS is 1.6 m RL. All levels discussed in subsequent sections relate to LINZ levelling datum.

3.6 Model Assumptions

Hydrological and hydraulic assumptions include:

- House rainwater tanks are full;
- Stormwater can freely enter the network. All runoff enters the reticulation system after initial losses are subtracted. Catchpits and inlets are not blocked;
- Stormwater can freely exit the network. Outlets are not blocked.
- There is no unmodelled storage required in the upper catchment.
- Formal soakage is not included in the model.

3.7 Model Validation

Explicit calibration of the Port Waikato Catchment model was not undertaken as part of this study. Flow or stream gauging data was not monitored for this project.

The results of the public questionnaire survey were analysed to identify stormwater flooding issues within the catchments and correlated with model performance (refer to Section 4.1).

3.8 Quality Assurance

The surveying and asset data collection phases were internally Quality Assured and Quality Control audited as part of the *CITY* DESIGN Survey Team standard processes.

The Port Waikato Catchment model has been internally quality assured and checked in accordance with the *CITY* DESIGN QA/QC Methodology.

It is considered that the Port Waikato Catchment model is 'fit for purpose' given acceptable modelling uncertainty and reliability.

4. ASSESSMENT OF EXISTING SYSTEM

4.1 Community Questionnaire

In June 2004, approximately 700 questionnaires were delivered to all households and businesses in the Port Waikato area. A copy of the questionnaire delivered is included in Appendix A.

A total of 135 (19% of total) questionnaires were returned completed. This is considered to be a normal response rate for this type of study.

A full list of responses and the extent and source of flooding problems are shown Figure 5. Table 4.1 summarises the responses received for the Port Waikato Township.

Table 4.1: Survey Questionnaire Responses

Survey Response	Number	Percentage of Responses	Percentage of Catchment
Total Number of Responses	**135**		**19**
Flooding Problems Reported			
Front of section	20	15	2.8
Rear of section	21	16	3.0
Street or footpath	13	10	1.8
Garage	4	3	0.6
Habitable basement	3	2	0.4
Non-habitable basement	2	1	0.3
Ground floor	1	1	0.1
Other	3	2	0.4
Unknown	0	0	0.0
No flooding problems reported	93	69	13.1
Source of Flooding			
Pipe system	2	1	0.3
Stream or open watercourse	3	2	0.4
Neighbouring property	14	10	2.0
Street	24	18	3.4
Other	11	8	1.5
Unknown	1	1	0.1
Frequency			
Every time it rains	11	8	1.5
3 - 6 months	6	4	0.8

Survey Response	Number	Percentage of Responses	Percentage of Catchment
1 year	10	7	1.4
2 years	2	1	0.3
5 years	1	1	0.1
Other	8	6	1.1
Unknown	3	2	0.4

As shown in Figure 4 the survey responses covered a wide area with an even distribution across the study area.

The majority of responses reported no flooding problems (69%) or attributed flooding to runoff from roads as well as from neighbouring properties. These questionnaire responses indicate that local or relatively minor stormwater deficiencies are the predominant concern for the residents within the catchment. This could be a reflection of the temporary residence time of the population and that a significant storm event has not occurred during the temporary period of residence.

4.2 Assessment of Existing Drainage System

The FDC drainage standard requires that the primary pipe system is capable of containing the 5 year ARI flow. Model simulations were carried out for ED and MPD landuse scenarios with the two existing pumps assuming that the second axial pump is lowered and working (refer to Section 3.1). Modelling results indicate that there are three significant areas within the study area where the pipe systems do not have sufficient capacity to convey the MPD landuse 5 year ARI storm event without surcharging above ground level. These are described below.

4.2.1 Maunsell Road

Model simulations indicate that the existing 300mm diameter pipe around 208 Maunsell Road does not have sufficient capacity to convey the ED and MPD landuse 2 and 5 year ARI storm events without surcharging above ground level. The upper hilly sub-catchments are steep with relatively impermeable soil which results in high stormwater flows to the Maunsell Road area. At present the only outlet from Maunsell Road west of Mission Road is a 300mm diameter pipe which drains to the Centreway roadside drain.

The stormwater runoff from the upper hilly areas is intercepted by roadside drains and discharges through the 300mm diameter pipe to the open drain that passes along the Centreway and Mission Road and finally discharge into the Main Drain through the 300mm diameter culvert under the Mission Road.

The flows through the 300mm diameter pipe from Maunsell Road are not affected by the downstream water level in the open drain, however this pipe and roadside drains have very low conveyance capacity, resulting in overflows and frequent flooding of adjacent properties. This matches reasonably well with the questionnaire responses as shown in Figure 5. The resulting depths of flooding in Maunsell Road are about 100mm and 180mm during 5 year and 100 year ARI storm events respectively.

4.2.2 Mission Road

The existing 300mm diameter culvert discharging into the Main Drain does not have sufficient capacity to convey the MPD landuse 2 and 5 year ARI storm events without surcharging above ground level. The stormwater flows through this culvert are affected by the downstream water level in the Main Drain. This culvert frequently overflows resulting in flooding of adjacent properties. The Main Drain downstream of this culvert overflows during the 5 year ARI storm event and resulting in flooding of low-lying properties along the Main Drain. This matches with the questionnaire responses.

4.2.3 Cordyline Road

The analysis of the ED and MPD scenarios indicates that the existing 300mm diameter pipe around the 15 Cordyline Road area has insufficient capacity to convey the ED and MPD landuse 2 and 5 year ARI storm events without surcharging above ground level. This pipe has very low conveying capacity, passes through the flat area and discharges into the Main Drain.

The stormwater flows through this pipe are affected by the downstream water level in the Main Drain. This pipe frequently overflows resulting in flooding of adjacent properties. The Main Drain along the central low lying area overflows during the 2 year ARI storm event and inundates the central low lying flat area.

4.3 Assessment of Existing Stormwater Pump Station

Model simulations were carried out with the two existing pumps assuming that the second axial pump is lowered and working (refer to Section 3.1). As there is no free outflow from the catchment, the depth and duration of flooding in the central low lying flat area within the catchment are related to the pump flow rates.

The simulations showed that initially the stormwater flows from all upstream catchments travel to the outlet of the Main Drain and is discharged to the Waikato River by the pumps. Then as the upstream flow increases above the pump flow rates the stormwater starts ponding at the outlet and finally flows backward and floods the whole central low lying flat area with the water level becoming horizontal over the entire area.

The MPD 100 year ARI storm event simulation indicated that the existing two pumps took about 4.8 days to reduce the flood level down to the normal water level in the Main Drain, which is considered about 600mm (1.4m RL) depth of water.

Sensitivity analysis of the pumping rates was undertaken to assess their effects on the depth and duration of flooding within the catchment. The sensitivity analysis indicated that it has a minor impact on the flood levels as this is determined by the relationship between the runoff volume and the storage characteristics of the large central low-lying flat area. However it has a significant impact on reducing the duration of flooding. The results of sensitivity analysis are presented in Table 4.2.

The model results show that during power failure or intake blockage when pumps are not working the 100 year ARI peak flood level would be increased marginally compared to that when two pumps are working. However the 5 year ARI peak flood level would be significantly increased during this worse case scenario.

Table 4.2: Assessment of Different Pumping Rates

Options	5-Year ARI		100-Year ARI	
	Duration of Flooding (days)	Peak Flood Level at Central Basin (m RL)	Duration of Flooding (days)	Peak Flood Level at Central Basin (m RL)
No Pump working	indefinite	2.30	indefinite	2.34
One existing pump with capacity of 0.075 m^3/s	8.1	2.12	17.0	2.33
Two existing pumps with capacities of 0.075 and 0.15 m^3/s	2.7	2.05	4.8	2.32
Three pumps with capacities of 0.075, 0.15, and 0.30 m^3/s	1.9	1.95	2.8	2.25
Four pumps with capacities of 0.075, 0.15, 0.30, and 0.30 m^3/s	1.6	1.90	2.0	2.18

Model simulations were conducted with the third pump with a capacity of 0.30 m^3/s (assumed double the capacity of the existing axial pump) in addition to the existing two pumps. Model results indicated that it took about 2.8 days to reduce the flood level down to the normal water level in the Main Drain during the 100 year ARI storm event. Also, it was found that the peak flood level at the central low lying basin area would be reduced by about 0.07m due to the additional pump.

Model simulations indicated that to reduce the flood level down to the normal water level in the Main Drain within 2 days during 100 year ARI storm event, an additional pump with capacity similar to the third pump would be required. Four pumps would reduce the peak flood level another 0.07m during 100 year ARI storm event at the central low lying basin area.

4.4 Assessment of Stream Erosion

There was little evidence of watercourse erosion in the flat areas of the catchment during a site visit, reflecting the slow flowing nature of the watercourses.

However the ephemeral gullies upstream may experience erosion, particularly in areas that are overgrazed. This has the potential to affect the suspended sediment load carried to the lower parts of the catchment. However management of the upper reaches of the catchment lies with individual landowners.

4.5 Assessment of Stream Water Quality

One water sample was collected from the main drain (just upstream from the pump station) at Port Waikato on 20th August 2004. The purpose of the sampling was to attain a snapshot of the water quality of the lower reach of the drain, prior to its discharge to the receiving environment (refer to Appendix B).

Limited conclusions can be made from a single sample. However observations of the Port Waikato Main Drain and the sample results suggest elevated levels of nutrients and depleted levels of dissolved oxygen in the waterway.

Other results to note from the single sample are as follows:

- Concentrations of total copper and total zinc in the Port sample *exceeded* ANZECC 2000 freshwater quality guidelines for the protection of aquatic ecosystems (95%) for total copper and total zinc.

- The concentrations of *E.coli* in the main drain sample was *below* the MfE freshwater bathing guideline acceptable/green mode running median value of 126 E.coli/100ml. (note: this guideline is not for a single sample result but a running median from regular sampling).

5. CATCHMENT FLOWS AND FLOOD LEVELS

5.1 Estimation of Flood Flows and Levels

Model simulations were carried out for the 2, 5, and 100 year ARI storm events for ED and MPD landuse scenarios with the two existing pumps assuming that the second axial pump is lowered and working (refer to Section 3.1). Table 5.1 provides the peak flows for all the sub-catchments and Table 5.2 provides the peak flood levels at various nodes along the stormwater drainage network system. Table 5.2 shows that the peak flood level within the Main Drain from Mission Road culvert to the outlet is constant.

Table 5.1: Peak Flows for 2, 5, and 100 year ARI Storm Events

Catchment Number (Refer to Figure 3)	Peak Flows (m^3/s)					
	Existing Development Landuse			Maximum Probable Development Landuse		
	2 Year ARI	5 Year ARI	100 Year ARI	2 Year ARI	5 Year ARI	100 Year ARI
1	0.25	0.48	1.16	0.25	0.48	1.16
2	0.06	0.12	0.29	0.06	0.12	0.29
3	0.11	0.20	0.47	0.13	0.24	0.53
4	0.24	0.45	1.06	0.29	0.53	1.20
5	0.13	0.24	0.53	0.18	0.31	0.65
6	0.08	0.15	0.35	0.13	0.23	0.47
7	0.08	0.15	0.34	0.11	0.20	0.41
8	0.04	0.06	0.13	0.04	0.07	0.14
9	0.02	0.04	0.09	0.03	0.05	0.11
10	0.07	0.12	0.27	0.09	0.16	0.32
11	0.03	0.06	0.14	0.04	0.07	0.16
12	0.09	0.17	0.37	0.13	0.22	0.46
13	0.10	0.18	0.40	0.13	0.23	0.47
14	0.03	0.06	0.14	0.04	0.07	0.16
15	0.07	0.14	0.33	0.09	0.16	0.37
16	0.04	0.07	0.18	0.04	0.07	0.18
17	0.07	0.13	0.31	0.08	0.15	0.35
18	0.08	0.14	0.35	0.09	0.16	0.38
19	0.08	0.15	0.37	0.10	0.18	0.41
20	0.04	0.08	0.21	0.08	0.14	0.32
21	0.15	0.28	0.70	0.26	0.47	1.04
22	0.16	0.31	0.73	0.21	0.39	0.88
23	0.08	0.14	0.31	0.10	0.18	0.37
24	0.06	0.10	0.25	0.09	0.16	0.33
25	0.02	0.04	0.10	0.03	0.05	0.11
26	0.04	0.07	0.17	0.05	0.09	0.21
27	0.06	0.11	0.26	0.07	0.12	0.28

Table 5.2: Peak Flood Levels for 2, 5, and 100 year ARI Storm Events

Node Number (Refer to Figure 3)	Peak Flood Levels (m RL)					
	Existing Development Landuse			Maximum Probable Development Landuse		
	2 Year ARI	5 Year ARI	100 Year ARI	2 Year ARI	5 Year ARI	100 Year ARI
AA010	3.06	3.13	3.25	3.07	3.15	3.27
AA030	2.38	2.49	2.63	2.44	2.53	2.66
AA040	2.38	2.49	2.63	2.44	2.53	2.66
AA050	2.38	2.49	2.62	2.44	2.53	2.66
AA055	1.90	1.99	2.29	1.91	2.05	2.32
AA060	1.90	1.99	2.29	1.91	2.05	2.32
AA080	1.90	1.99	2.29	1.91	2.05	2.32
AA110	1.90	1.99	2.29	1.91	2.05	2.32
AA130	1.90	1.99	2.29	1.91	2.05	2.32
AA160	1.90	1.99	2.29	1.91	2.05	2.32
BA010	3.38	3.44	3.63	3.37	3.44	3.63
BA020	2.78	2.90	3.42	2.78	2.92	3.43
BA040	1.90	1.99	2.29	1.91	2.05	2.32
BA050	1.90	1.99	2.29	1.91	2.05	2.32
CA010	2.01	2.63	2.69	2.07	2.63	2.71
DA010	2.38	2.83	3.06	2.47	2.95	3.07
EA010	2.29	2.35	2.46	2.32	2.38	2.49
EA020	2.08	2.12	2.29	2.10	2.14	2.32
FA010	3.06	3.13	3.24	3.07	3.15	3.26
FA020	3.05	3.12	3.24	3.07	3.14	3.26
GA010	2.11	2.34	2.70	2.19	2.61	2.73
GA020	1.90	1.99	2.29	1.91	2.05	2.32
GA040	1.90	1.99	2.29	1.91	2.05	2.32

5.2 Flood Hazard Areas

For the purpose of this study, a flood hazard is defined as the likelihood and severity of a flood occurring, and is expressed in terms of a return period.

Except where specific surveying was done for this study, the extent of each flood hazard area was determined by interpolating between ground contours that were at one metre intervals, considering reticulation details, overland flow paths and sub-catchment boundaries.

This study uses the best information available at the time. It is possible that the estimates of flood flows and the predicted extent of flooding will change over time as new information becomes available. The central part of the catchment is very low and flat, and needs more ground survey to determine accurate extents of flooding.

Flood Hazard areas have been defined for the 2, 5, and 100 year ARI events using impervious areas based on Maximum Probable Development (MPD) landuse. Figure 6 shows the extents of flooding due to the overflow of the channels and pipe drainage systems within the catchment.

The extents of flooding were confirmed with questionnaire responses, site visits, detailed survey data, and contours. Due to low conveyance capacity of the culverts under Mission Road and the Maunsell Road a number of properties are flooded along the Maunsell Road west of Mission Road and the corner of Centreway and Mission Road (refer to Section 4.2). The flood hazard map for the 2-year ARI storm event (refer to Figure 6) compares reasonably well with the questionnaire responses as shown in Figure 5.

Flood hazard maps show that during the 2, 5 and 100 year ARI storm events the Main Drain spills over and inundates a wide area in the central low lying flat area. Due to the flat nature of the central low lying floodplain area a large area is flooded even in 2 and 5 year ARI storm events.

Analysis of ED and MPD scenarios shows that an increase in stormwater flow due to the proposed development in the area would not make a significant difference to the extent of the flood hazard areas.

6. CATCHMENT ISSUES

6.1 General History of the Catchment

The Port Waikato area has been created as a result of material carried down the Waikato River and deposited at the mouth. Evidence indicates the river flow was once located as far south as Maunsell Road, (Holocene period) and as recently as 1920 the river mouth was some thousands of metres nearer the Port Waikato urban centre than it is today.

The low-lying developed area was originally a lake and is prone to flooding from both local rainfall and upland catchments, from interruption to natural drainage to the estuary because of tidal cycles, sand blocking of the outfall channel and lower drain, and occasional possible influence of Waikato River floods.

The catchment lies on the only major active fault line in the Auckland region. An earthquake with magnitude 5.9 Richter scale occurred in June 1891.

A 1942 aerial photo indicates that there were 5 or 6 buildings around the wharf area with only 2 or 3 in the present township area. Most of the present dwellings, roads and limited services were constructed in the 50's and 60's. A new subdivision was constructed during the 1980's on steeper ground to the south of the settlement with access from Maunsell Road.

The area is of significance to the Maori because of the proximity of the area to the river and sea, being traditional food sources. Currently, the Nagati Karaoa Ngati Tahinga Maori Trust owns a significant portion of land in Port Waikato.

6.2 Catchment Management Issues

This study has identified several catchment management issues in the Port Waikato Township area based on the questionnaire survey, site visits, reticulation deficiencies and previous studies as summarised below.

6.2.1 Stormwater Quantity Issues

Stormwater quantity issues are summarised below:

- The Port Waikato Township area has large steep upstream contributing catchments with unstable soils resulting in high stormwater flows which include a significant amount of erosion and farming debris.

- During a storm event the stormwater runoff from upper catchments discharges into the very low gradient drainage systems and deposits sediment and debris in the present stormwater drainage systems. This silting has the effect of filling drains and causing stormwater overflow onto some properties. This results in high level of maintenance for the drainage systems. Site visits identified a number of culverts silted up.

- Most of the existing drainage systems have low conveyance capacity resulting in overflows and frequent flooding of nearby properties. Properties built on the low-lying floodplain are significantly affected due to overflow from the Main Watercourse. Hydrological and hydraulic modelling identified stormwater flooding at several locations due to overflows of drainage systems as described in Section 4.2.

- A number of properties are affected due to local stormwater drainage problems such as overflow from culverts, drains and pipes, inadequate or lack of roadside drainage systems, inadequate drainage due to downstream higher stream flood levels, and blockage of drains and culverts.

- Stormwater is disposed through ground soakage in a significant proportion of the catchment, however, this has limited effectiveness when the water table is high and during high intensity short duration storm events.

6.2.2 Stormwater Quality Issues

The catchment does not have a large urban context therefore typical water quality issues for those types of areas are not dominant. Stormwater quality issues are summarised below:

- Sewage is disposed through septic tanks and ground soakage trenches. A number of systems are reported as vulnerable to surface flooding by stormwater or by high groundwater levels. This may result in sewage overflows to the stormwater drainage system, which discharges contaminants to the receiving environments. In addition recreational water quality guidelines may be exceeded.

- The existing drainage channels have low through-flow due to low relief, and are mostly surrounded by grassed area with domestic animals. Their quality is likely to be dominated by low dissolved oxygen, high temperature fluctuations and high nutrient inputs typical of farm drains. Site visits identified algae growth in the Main Watercourse reflecting high nutrient input from farming activities and /or sewage.

- Stormwater runoff from the catchment has the potential to impact on the aquatic or coastal receiving environments due to increased discharge of contaminants and sediments in stormwater. Problems associated with stormwater quality are potential wastewater overflows from inundated or defective septic tanks, sediment eroding from streambeds and upper hilly catchments during storm events, road and traffic related contaminant discharge, local roofing material and local farming activities.

6.3 Catchment Threats

As the Maraetai Beach is used for recreational activities and contact with the water being pumped into the bay is possible, a sanitary level of water quality being discharged at this point should be maintained at all times.

The greatest threat to the catchment is the interactions between stormwater and wastewater disposal. It is recognised that the presence of animal faecal material sourced from birds and livestock do not present as greater risk to human health as human sourced faecal material. Wastewater disposal systems and stormwater discharges need to be separated as far as practicable.

However the overall water quality of the water in the Maraetai Bay is likely to be dominated by the Waikato River water and therefore influenced by factors beyond the Port Waikato Catchment.

7. STORMWATER MANAGEMENT OPTIONS

7.1 Problems identified

A number of stormwater flooding and water quality problems have been identified within the study area, mostly in terms of inundation caused by poor reticulation capacity and the low standing ground levels.

Future housing development in the area has the potential to increase the flow and quantity of stormwater travelling through the existing reticulation system and worsen the current problems. Another problem is related to the potential for erosion of land and channels and contaminant discharge into the receiving environment and thereby affecting stream habitat.

Modelling has shown that for future development about 27% of the catchment would be likely to experience unacceptable levels of inundation as shown in Figure 6. This includes 2, 5 and 100 year storm events.

Options for avoiding or mitigating these problems have been considered in relation to FDC policy and objectives, FDC code of practice for subdivision and development, ARC TP10 guidelines (7) and low impact design techniques ARC TP124 (8). The aim is to provide, where possible, hydraulic neutrality for the new developments.

Selection and implementation of these processes should involve community participation and the implementation of any preferred option would require further investigation and evaluation.

A number of structural and non-structural options are described below for mitigating stormwater flooding and quality problems identified through this study.

7.2 Non-Structural Options

The following non-structural options are considered:

- Cleaning of catchpits, culverts, and road gutters frequently to avoid blockage,
- Carry out street sweeping frequently,
- General maintenance and seasonal clearing of watercourses,
- Implement an educational program to raise public awareness of stormwater quality issues and to promote good practice,

- Develop guidelines for inspection and maintenance by the owners of private soakholes, on-site wastewater treatment and disposal fields.

7.3 Structural Options

7.3.1 Stormwater Flooding

The following options were considered to mitigate existing flooding problems within the area:

Option-1: Western Diversion

Diversion of flow from sub-catchments 2, 3, and 4 to the West Coast by providing a pipe conduit along Maunsell Road and through the low point in the dune system where Maunsell Road joins Centreway to join the small catchment which drains to the beach (refer to Figure 7). This will require a total length of about 550m of 825mm diameter pipe, and includes upgrading of the culvert under Mission Road (refer to Figure 7). The outcome would be diverting up to 5-year ARI flows to the West Coast. Overland flow paths would be provided through roadside drains along Maunsell Road, Mission Road and Centreway Road to divert higher flows to the Main Drain. The depth of flooding over the roads would be about 150mm during the 100-year ARI flood.

Option-2: Main Drain Diversion

Diversion of flow from sub-catchments 2, 3, and 4 to the Main Drain by providing a pipe conduit along Mission Road from Maunsell Road (refer to Figure 7). A total length of about 130m of 600mm diameter pipe would be required. This option also includes upgrading of the culvert under Mission Road (refer to Figure 7). This would allow diverting up to 5-year ARI flows to the Main Drain. Overland flow paths would be provided through roadside drains along Maunsell Road, Mission Road and Centreway Road to divert higher flows to the Main Drain. The depth of flooding over the roads would be about 150mm during the 100-year ARI flood.

Option-3: Northern Diversion

Diversion of flow from sub-catchments 12 and 13 to the northern sand dunes by providing a soakage trench within the sand dunes and a pipe conduit along the 13 and 15 Cordyline Road property boundaries and through the reserve area at the end of Cordyline Road to the soakage trench (refer to Figure 7). A total length of about 180m of 525mm diameter pipe would be required. An alternative option could be to provide a soakage trench within the reserve area. In this case a total length of about 60m of 525mm diameter pipe would be required.

Option-4: Main Drain Bunding

Bunding of the Main Drain on both banks for a distance of 200m from the Mission Road culvert to protect the low-lying properties from flooding (refer to Figure 7) during the 5 year ARI storm event. A 0.5m high embankment would be provided. This part of the Main Drain needs to be cleaned and cleared of any debris or vegetation within the channel to improve the channel capacity. The lower part of the Main Drain would not be embanked so as to allow ponding and storage of flood volumes within the central low lying basin and the wetland area. The central low-lying basin area would need to be developed into a grassed and vegetated area containing a large section of the Main Drain that would store catchment runoff during storm events.

Model simulations were carried out to assess the 4 remedial options including combinations of options and results are presented in Table 7.1. In all cases two existing pumps were considered operational assuming the existing second axial pump is lowered and working and model results are compared with the existing condition.

Model results indicate that the peak flood level at the central low lying area would not be significantly reduced but the duration of flooding would be substantially reduced. However the remedial options would remove the existing flooding problems for the properties along Maunsell Road, Mission Road, Centreway Road, Cordyline Road and the low lying properties along the Main Drain up to 5 year ARI flood event.

Table 7.1: Assessment of Flood Remedial Option

Options	5-Year ARI		100-Year ARI	
	Duration of Flooding (days)	Peak Flood Level at Central Basin (m RL)	Duration of Flooding (days)	Peak Flood Level at Central Basin (m RL)
Existing condition with two existing pumps	2.7	2.05	4.8	2.32
Option 1	2.4	2.00	4.3	2.28
Option 2	2.7	2.05	4.8	2.32
Option 3	1.8	2.00	3.0	2.27
Options 1, 3 & 4	1.6	1.95	2.8	2.21
Options 2, 3 & 4	1.8	2.00	3.0	2.26

A rough order preliminary cost estimate is presented in Table 7.2 to provide an indication of the level of funding that may be required. The costs provided in Table 7.2 include a 25% contingency and 15% allowance for design, consents and contract administration.

Table 7.2: Costs of Proposed Remedial Options

Options	Remedial Works	Costs
Option 1	New 825mm diameter pipe of 550m length along the Maunsell Road west of Mission Road to the West Coast beach, upgrading existing 300mm diameter culvert to 450mm under the Mission Road, regrading Maunsell and Mission Road to divert higher flows to the Main Drain, upgrade Maunsell, Mission Road, and Centreway roadside drains, raising Maunsell, Mission Road, and Centreway verges to prevent runoff entering private properties.	$495,000
Option 2	New 600mm diameter pipe of 130m length along the Mission Road from Maunsell Road to the Main Drain, upgrading existing 300mm diameter culvert to 450mm under the Mission Road, regrading Maunsell and Mission Road to divert higher flows to the Main Drain, upgrade Maunsell, Mission Road, and Centreway roadside drains, raising Maunsell, Mission Road, and Centreway verges to prevent runoff entering private properties.	$120,000
Option 3A	New 525mm diameter pipe of 180m length from 15 Cordyline Road property boundaries to the northern sand dunes, providing soakage trenches in sand dunes.	$106,000
Option 3B	New 525mm diameter pipe of 60m length from 15 Cordyline Road property boundaries to the reserve area at the end of Cordyline Road, providing soakage trenches in the reserve area.	$50,000
Option 4	Bunding of the Main Drain on both banks for a distance of 200m from the Mission Road culvert including channel improvements of the Main Drain.	$48,000

The downstream high tide level at the West Coast causes restriction to flow and reduces the conveying capacity of the proposed pipe reticulation in Option 1. As a result a large diameter pipe is required to divert runoff from upper hilly catchments to the West Coast resulting in a high cost compared to Option 2.

Considering the cost and benefit of the proposed remedial options, the combination of Options 2, 3 and 4 is the preferred scenario. This would mitigate the existing flooding problems up to 5 year ARI flood events. However during the 100 year storm event it is expected that the large central low lying area would be flooded and a number of properties along the Main Drain would be affected. Considering the practicality and cost to mitigate the 100 year flood it would be reasonable to accept the 100 year flood within this low lying area and designate this as a flood prone area.

In addition to the above specific major options the following options are considered on a catchment-wide basis to reduce flooding problems:

- Installing additional soakage on a catchment-wide basis to reduce flows and volume. Ideally this work would be carried out following a detailed soakage investigation of the catchment to determine the feasibility of additional soakage. By identifying areas with high soakage potential and fully utilising these areas it may be possible to reduce flooding problems in other areas with poor soakage potential,
- Installing catchment-wide soakage trenches within road reserves wherever possible to dispose of road runoff and supplement the existing drainage system. These trenches would improve flooding conditions at the downstream low lying area by reducing stormwater flows and volume,
- Providing technical guidelines to the owners for improving/ upgrading existing private soakage systems to mitigate localised flooding problems,
- Raising road verges to prevent road runoff entering private properties,
- Lowering road surface to improve secondary flow paths,
- Constructing temporary ponding areas,
- Improving/upgrading capacity of the existing roadside drains,
- Retaining an appropriate proportion of pervious ground surfaces and minimising impervious areas,
- Controlling infill housing development.

7.3.2 Stormwater Quality

The following options could be considered to mitigate stormwater quality problems within the area:

- Installing a stormwater settling pond at the bottom of the sub-catchment 1 to retain high silt discharge from the upper hilly catchment (refer to Figure 7). The pond would be designed to improve stormwater quality in accordance with ARC TP10 guidelines (7) to remove at least 75% of total suspended sediments on a long-term average basis. The existing sediment trap needs to be reconfigured to be included in the proposed sedimentation device,

- Installing coarse sediment traps along Maunsell Road at the bottom of the upper hilly sub-catchments 2, 3, and 4 (Figure 7),
- Installing catchment-wide soakage trenches within road reserves wherever possible in accordance with ARC TP10 guidelines (7) to improve road runoff quality,
- Providing catchbasin silt traps within catchpits,
- Installing rain gardens where appropriate,
- Providing riparian planting along each bank of the perennial watercourses and setting aside ecological corridors,
- Developing the existing wetlands to a recreational water feature,
- Initiating a long-term program of sediment and water quality monitoring.

7.3.3 On-site Wastewater Treatment and Disposal

At present septic tank systems are the main wastewater treatment and disposal method in Port Waikato Township. Properly designed, installed and maintained septic tanks can function for many years. Routine maintenance is critical for satisfactory performance. Annual inspection is desirable to prevent tank solids from overflowing and sealing the soil in disposal fields. A system that is improperly sited, improperly maintained, or overloaded, can discharge bacteria, viruses, nitrates and hazardous contaminants to groundwater that may travel to nearby surface waters (lakes, streams, or coastal waters). This can harm lakes, streams, and bays by increasing algae growth and threatening fish and other aquatic habitats.

A previous study (3) in 1991 indicated that a number of the existing wastewater disposal systems have "failed" as a result of being located in areas with high groundwater levels. These systems may have further deteriorated by now.

The following options are considered to mitigate existing on-site wastewater treatment and disposal issues within the area:

- Provide guidance to improve private septic tank and ground soakage disposal systems,
- Assess existing groundwater quality to ascertain continuation of ground soakage for the future development,

- Undertake stormwater quality monitoring each year in the Main Drain to assess the level of nutrients,

- New septic tanks and wastewater disposal field should not be permitted within the 100 year ARI flood hazard area.

If investigations suggest ground soakage is not a viable option especially for areas located below the 100 year ARI flood level, the following options may be considered:

- Identify locations suitable for septic tank and ground disposal,

- Undertake investigation to assess the suitability of alternative disposal systems in place of soakage trenches, such as textile filter systems after primary septic tank treatment, community based systems such as spray irrigation or shallow bore disposal in the spit area, or wetland disposal in the central coastal reserve area,

- Initiate an annual inspection program to carry out all maintenance and operational requirements to ensure optimum system performance.

8. RECOMMENDATIONS

A list of recommendations based on the results of this study is presented below. The recommended options are intended to mitigate stormwater flooding and water quality problems.

8.1 Policy Recommendations

- New building should not be permitted within the 100 year ARI flood hazard area,
- New buildings should be constructed with sufficient freeboard (at least 500mm) above the 100 year ARI flood level. Where the 100 year ARI flood level is not available, a specific engineering assessment should be undertaken,
- New soakage, septic tank and wastewater disposal fields should not be permitted within the 100 year ARI flood hazard area,
- Areas sensitive to flooding should be isolated and no development should be allowed in these areas,
- In view of the dynamic nature of the township future development and investment should be limited,
- New development should not be allowed to take place in steep areas where on-site stormwater and wastewater control is not feasible.

8.2 Catchment-Wide Recommendations

- Install catchment-wide soakage trenches within road reserves wherever possible to dispose of road runoff and supplement the existing drainage system,
- Undertake soakage investigations to determine the potential for installing additional soakage on a catchment-wide basis to reduce peak flows and flood volumes,
- Provide and/or upgrade as necessary the roadside drains to prevent uncontrolled stormwater flows to the properties,
- Providing riparian planting along each bank of the perennial watercourses and set aside ecological corridors,
- Initiate a long-term program of sediment and water quality monitoring,

- Undertake a regularly scheduled maintenance program for catchpits, culverts/pipes, road gutters, and watercourses. Maintenance of soakages, septic tanks, and disposal fields on private properties is the responsibility of the property owners,
- Undertake an educational program to raise public awareness of stormwater quality issues and to promote good practice.

8.3 Mitigation of Existing Issues

- Undertake further investigation, once more information regarding contours and asset data is available, to determine more accurate flood hazard areas, and prepare a future works programme,
- Undertake Options 2, 3 and 4 to mitigate existing flooding problems within the catchment,
- Install a stormwater settling pond at the bottom of sub-catchment 1 to retain high silt discharges from the upper hilly catchment.
- Install coarse sediment traps along the Maunsell Road at the bottom of upper hilly sub-catchments 2, 3, and 4,
- Undertake conventional stormwater disposal methods to mitigate the minor localised flooding in Cobourne Place and other areas as reported in questionnaire responses (refer to Figure 5),
- Undertake suitable preventive measures to mitigate habitable basement flooding at 69 Maunsell Road by diverting runoff from the driveway of the neighbouring property,
- Undertake stormwater quality monitoring each year in the Main Drain to assess the level of nutrients,
- Provide guidance to improve private septic tank and ground soakage disposal systems.

8.4 New Development

- All new buildings, both residential and commercial, should be located outside the 100-year ARI flood plain,
- No buildings should obstruct any secondary flow path and building floor levels should be a minimum of 500mm above 100 year ARI (1% AEP) flood levels,
- The designer should provide for secondary flow paths for 100 year ARI (1% AEP) where primary flows exceed 200 l/s. The

primary design flow should generally be based on FDC code of practice for subdivision and development,

- All new development should maintain hydraulic neutrality, that is, post development peak flows, flood volumes and time of concentration should be maintained at or below the pre development levels,
- Erosion protection should be provided for any piped discharge of stormwater into the catchment watercourses,
- New developments should provide stormwater treatment in accordance with the ARC TP10 guidelines (7). In general sites should provide stormwater quality treatment of at least 75% sediment removal on an annual average basis,
- No piping of any existing perennial watercourses should be allowed.

8.5 Drainage System Maintenance

Ongoing maintenance should be undertaken to ensure continued efficiency of stormwater and wastewater systems.

- Culverts under roads and driveways should be inspected and cleaned out twice annually and after big storms to maintain maximum stormwater drainage capacity,
- Roadside drains and watercourses should be inspected annually and any accumulated litter, debris and plant material removed to maintain maximum stormwater drainage capacity,
- Appropriate erosion protection measures should be implemented where excessive erosion has been noticed along the watercourses,
- Stormwater settling ponds including inlets and outlets should be inspected for litter build-up and this should be removed as required,
- ARC Guidelines generally recommend the clean-out of deposited sediment in ponds when 10% of the permanent water volume is lost to sediment accretion. Excavated sediments from the pond may require specific disposal and may require specific resource consent,
- Maintenance of soakages, septic tanks, and disposal fields on private properties is the responsibility of the property owners.

9. REFERENCES

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6. Beca (2000), Design Rainfall Maps. Prepared for Franklin District Council, February 2000.

7. ARC (2003). Stormwater Management Devices: Design Guidelines Manual. Auckland Regional Council, Technical Publication No 10. May 2003.

8. ARC (2000). Low Impact Design Manual for the Auckland Region. Auckland Regional Council Technical Publication No 124.

FIGURES

APPENDIX B

Stream Water Quality Assessment

APPENDIX A

Community Questionnaires