Technical Report

Omahu Rezone Stormwater Management

Prepared for Hastings District Council

12 April 2012

QUALITY STATEMENT

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REVISION SCHEDULE

Rev No	Date	Description	Prepared By	Checked By	Reviewed By	Approved By
1	12-04-11	Revision – MWH review and Client initial comments	R van Bentum	W Hodson	A Leahy	R van Bentum
2	10-05-11	Final – HDC Client Comments	R van Bentum	W Hodson	A Leahy	R van Bentum
3	14-02-12	Final – HBRC HDC Comments	R van Bentum	W Hodson	A Leahy	R van Bentum
4	27-02-12	Final – HBRC comments	R van Bentum	W Hodson	A Leahy	R van Bentum
5	21-03-12	Final – M Kneebone comments	R van Bentum	W Hodson	A Leahy	R van Bentum

Hastings District Council

Omahu Rezone Stormwater Management

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1.1 Report Scope

Hastings District Council (HDC) is applying for consents to discharge stormwater as part of the proposed re-zoning to light industrial use of a strip of land along Omahu Road. The area of the proposed re-zone is outlined in Figure 1.

This report has been prepared as a technical report in support of the consent application and AEE. The report sets out the overall approach or strategy to be followed, along with detail of the generic design for the key elements of the system.

This report outlines the concept, assumptions and proposed design of the stormwater system for the proposed development. The design for the on-site system is generic and site specific design of the systems for each lot development will be required. The stormwater management approach includes a range of strategies to mitigate both stormwater quality and quantity effects arising from the development at two levels namely:

On-site - within the boundary of the privately owned lots and
Off- site - outside the privately owned land on areas to be owned and managed by HDC

Further information regarding the approach, preliminary design basis and expected performance of these mitigation strategies is provided in the following sections of this report.

1.2 Re-Zone Area

The re-zone area extends on the north-eastern side of Omahu Road from north of Ormond Road to just north of Kirkwood Road. The depth of the zone, i.e. the distance from the back of the zone to Omahu road varies from 50 to 150m.

For the purpose of this assessment, the zone has been divided into three off-site catchments (refer figure 1). Stormwater from each of the sub-catchments drains to one of three infiltration basin locations:

- Basin 1 Catchment (area = 8.1 ha)
- Basin 2 Catchment (area = 10.6 ha)
- Basin 3 Catchment (area = 17.8 ha)

The road reserve along the Omahu Road frontage of the rezone area currently drains to existing systems on the south of Omahu Road. It has been assumed that this will continue to be the case. Stormwater runoff from the upgraded Omahu Road formation will be managed and discharged to the south side of the road.

Figure 1 illustrates the zone extent, the approximate extent of swale drains to be provided at the rear of the re-zone area and the areas within which the proposed infiltration ponds will be located as well as the sub-catchments discharging to each infiltration area. The anticipated routes of designations / easement strips to enable individual properties to connect to the swale are also indicated. The stormwater system is designed to provide for gravity connections where these are practical and possible, however some areas may require to pump.

For clarity, it should be noted that the corridor for the proposed swale is located outside the proposed zone – refer to the cross section in Figure 6, Section 5.3.

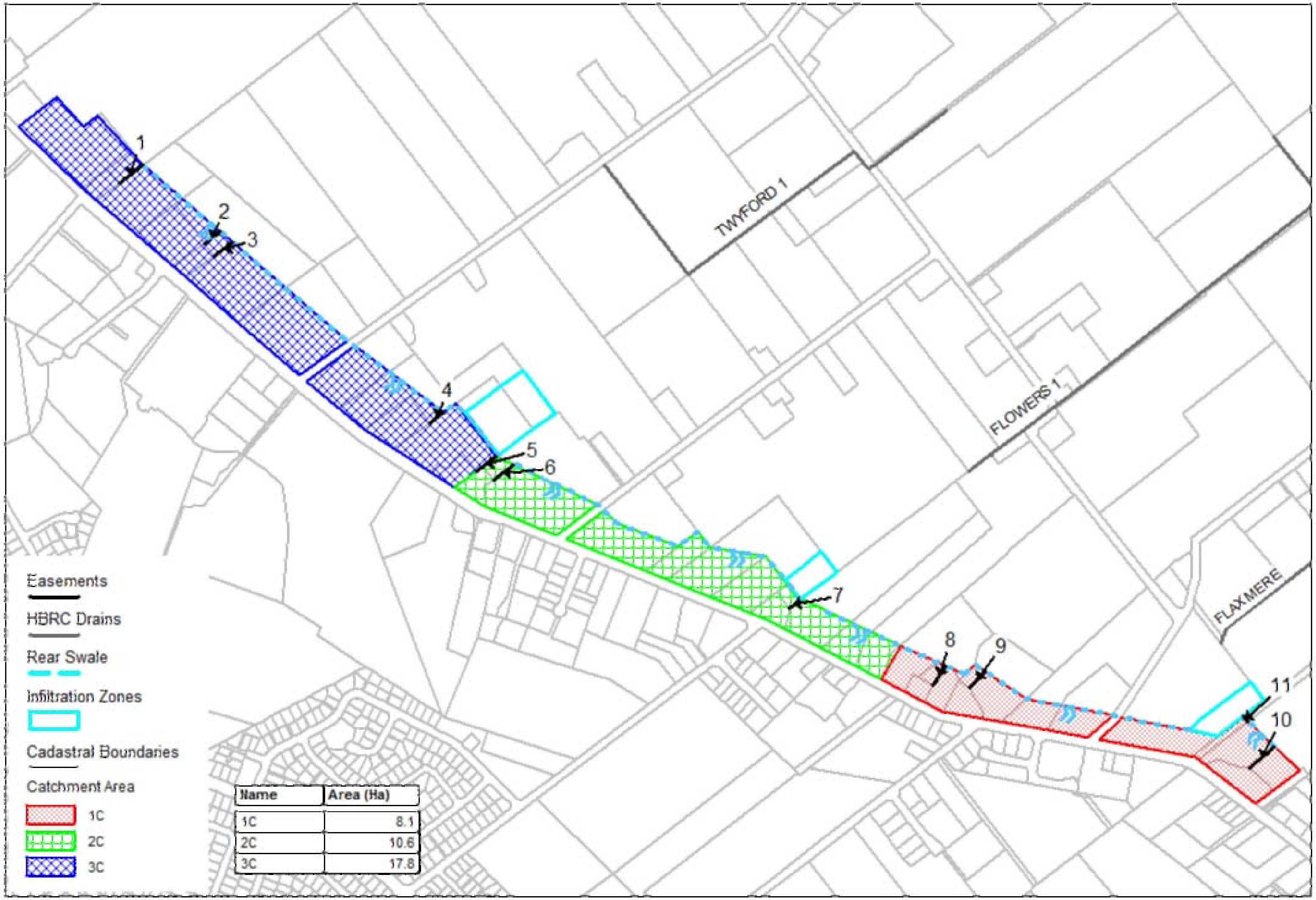

Figure 1 : Layout Plan of the Proposed Rezone Area

2 Stormwater Management Approach

2.1 Stormwater Management Philosophy

The Council wishes to implement a stormwater management strategy that:

- provides land that is 'fit for use' (which necessitates an appropriate level of flood / inundation protection);
- satisfactorily avoids, remedies or mitigates any potential adverse effects on the environment;
- ensures that the risk of contamination associated with industrial activities is adequately managed; and
- is cost effective, efficient and affordable throughout the life of the development.

Particular consideration has been given to the following principles / matters:

- The principals of Low Impact Urban Design;
- The specific characteristics of the potential stormwater receiving environments;
- Climate change
- The HBRC Stormwater Guidelines;
- The objectives of the Council's LTCCP, the Council's Engineering Code of Practice and Subdivision and Development Best Practice Design Guide; and
- On-site Stormwater Management Guideline (NZWERF/MfE 2004).

Having undertaken a comprehensive 'risk based' assessment of the issues and options available, the following key design objectives were identified:

- the minimisation of the extent (frequency and volume) of any discharge into the Raupare Stream catchment;
- the treatment, storage and disposal of stormwater as close to source as possible to reduce risks and minimise changes to the local shallow groundwater system;
- the utilisation of distributed infiltration disposal basins to reduce concentration effects; and
- the effective management of the risks of contamination and spills

2.2 Quality and Quantity Mitigation Strategy

The strategy proposed has the following four major components:

1. ***The use of on-site systems managed by individual owners / operators***
   The emphasis for the on-site systems is on providing primary treatment, quality control and flow mitigation for short duration / high frequency events. The use of on-site detention (near to source) also reduces the required size of downstream swales and ponds.
   Key mitigation methodologies:
   - The use of inert roof materials
   - The bunding of those areas within which stormwater is anticipated to become contaminated and the discharge of this water to the HDC sewer
   - The disposal of roof stormwater to ground on-site
   - The treatment of stormwater falling on hardstand areas prior to this being discharged into the off-site system
   - The attenuation of stormwater flows on-site prior to their discharge into the off-site system

2. ***The implementation of a Hastings District Council Off-site System***
   The off-site system, with its infiltration basins, will provide additional treatment protection and quantity mitigation for longer duration / low frequency rain events.
   Key mitigation methodologies:
   - Ensuring that overflows from the off-site system do not occur in events of up to 10yr ARI and are less than or indistinguishable from Greenfield ones in greater events

3. ***Monitoring and Maintenance***
On-site systems are to be monitored / maintained on an annual basis. The HDC system of swales and basins is to be monitored on a periodic basis – refer to the draft conditions in the attached application for more detail.

4. ***Regulatory Mechanisms***
A series of existing and new District Plan and By-law standards will be adopted and implemented to manage stormwater – refer to the attached application for more detail.

Key mitigation methodologies:

- The identification and control of those activities which, if not appropriately controlled, may generate unacceptable risks from accidental or negligent spills
- The requirement for stormwater systems, capable of achieving the identified level of service, to be installed and maintained on-site;
- The implementation of a regime for the monitoring and auditing of the maintenance and / or performance of on-site systems

A summary of the strategies and their specific contribution to mitigating quality and quantity effects in the off-site and on-site system is provided in Table 2-1 below:

Table 2-1 : Specific Impacts of Quality and Quantity Mitigation Strategies

System Component	**Quality Management Strategy**	**Mitigation Impact**
On-site – building roofs	All roofs to be constructed from inert materials e.g. coloursteel Pre-treatment to remove grit and detritus prior to discharge to infiltration to maintain soakage efficiency	Significantly reduced metal contaminant loads in roof runoff – predominantly zinc
On-site – yard areas	All areas where spillage of contaminants may occur to be bunded with stormwater directed to the HDC sanitary sewer.	Reduced risk of accidental contamination of stormwater runoff
	Stage 1 Sump treatment for flows up to 1 in 10 year ARI	Reduction in sediment and settleable solids loads.
	Stage 2 Humeceptor or similar device with bypass for peak flows	Potential reduction in TPH and gross solids loads
Off-site - system – swale	Filtration by grass swales provided adequate detention time	Further sediment removal particularly during minor rain events, when evaporation and infiltration are more significant
Off-site system – pond	Maintenance regime will be established to keep the surface from clogging.	Further sediment and contaminant removal prior to infiltration
	Quantity Management Strategy	
On-site – building roofs	Roof water for all event durations with 10 year ARI captured and disposed to on-site ground soakage. Optional tank storage with some re-use as a complementary strategy.	Zero discharge in frequent rain events. Assists with hydrologic neutrality with recharge of groundwater dispersed along the development
On-site – yard areas	Yard water for all rain event durations with 10 year ARI detained either in shallow above	Flow limited discharge in all normal (frequent) rain events.

	grounding ponding or shallow below ground detention with controlled discharge to the swale system	
Off-site - system – swale	Runoff flows and volumes conveyed to infiltration basins for events up to 1 in 10 year ARI	Flows contained within swale within the acceptable freeboard such that there is no uncontrolled overland flow for events up to a 1 in 10 yr ARI
Off-site – system – pond	Runoff volumes contained and disposed of to ground within one of three infiltration basins. Larger events overflow from basins to local drainage network.	Zero discharge for frequent rain events resulting in reduced flows to the Raupare in all events up to 1 in 50 yr ARI

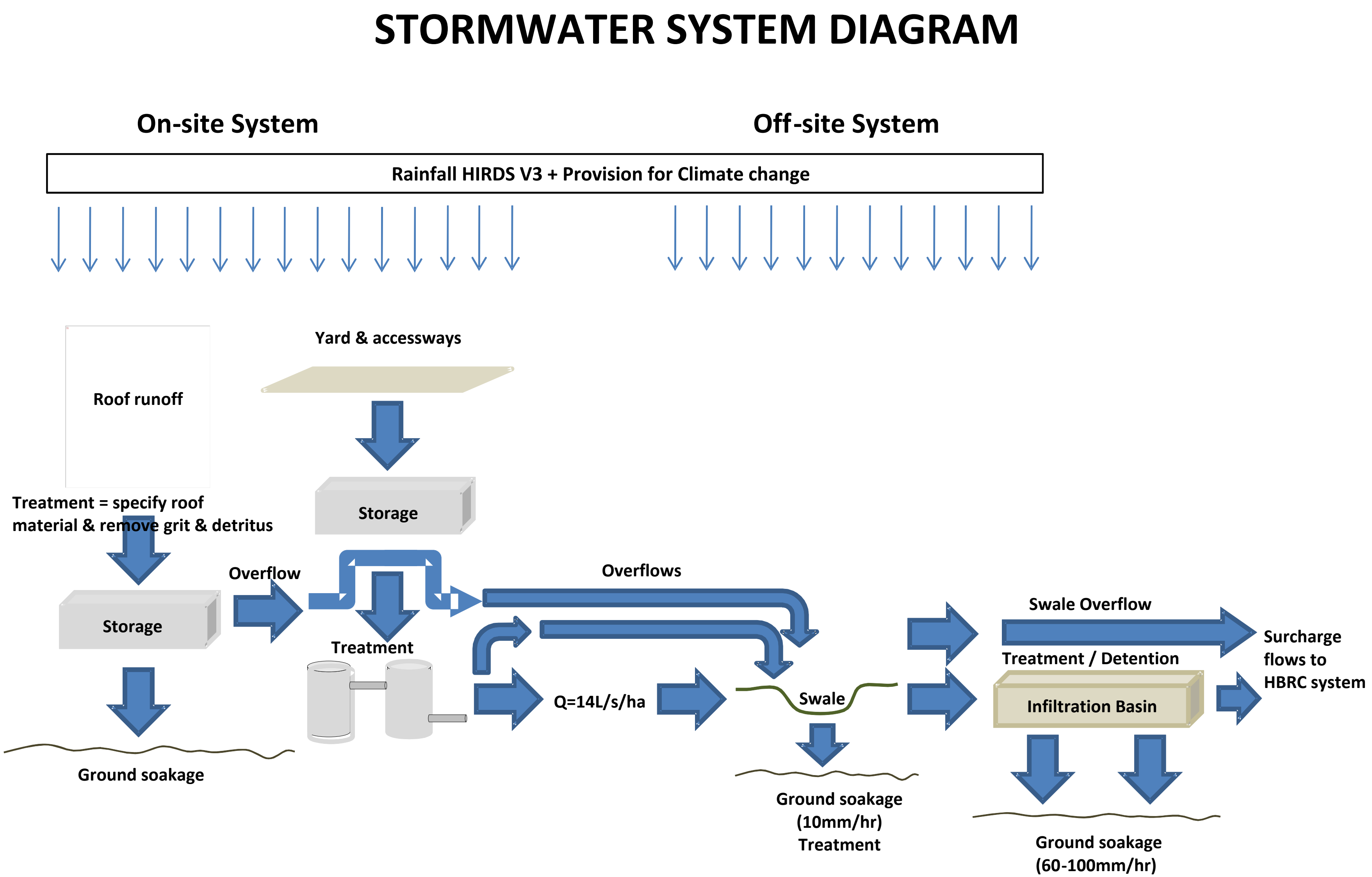

Figure 2 : Stormwater System Diagram

2.3 On-Site System Level of Service

As with all stormwater management systems, it is not possible to design the primary system for all rainfall events. This system has been designed with specific levels of service chosen for each part of the system. The on-site system is key in ensuring adequate treatment of stormwater as well as reducing the cost of mitigating the impacts of additional stormwater runoff volumes in the off-site system.

Quantity
The level of on-site stormwater runoff control is in accordance with the Hawke's Bay Regional Council (HBRC) stormwater guidelines (Hawkes Bay Waterways Guidelines – Stormwater Management – May 2009) for control of 2 year and 10 year ARI storm events.

The on-site system is designed to manage flows from all events up to a 1 in 10 yr ARI, with excess flows passed forward to the off-site system. This is achieved by managing roof runoff via on-site pre-treatment and then infiltration with storage system. Yard runoff is managed by shallow detention storage to limit runoff for all storms up to a 10 year ARI standard to 14 l/s//ha. This equates to the estimated pre-development greenfield peak runoff rate for a 2 year ARI storm (40 minute rainfall of 20mm/hr).

Quality
The level of on-site stormwater treatment control is in accordance with the Hawke's Bay Regional Council (HBRC) stormwater guidelines (Hawke's Bay Waterway Guidelines – Stormwater Management). The level of service criteria for stormwater treatment has been defined as the residential baseline for the Hastings urban area. Therefore the proposed systems have been selected on the basis that they ensure stormwater quality is at least as good as if not better than the Hastings Residential Baseline level (refer Table 3-7.

The intended level of service for the treatment, attenuation and disposal elements of the on-site system is set out in table 2.2 below.

Table 2-2 : Level of Service for On-Site System

Surface	Level of Service	Stormwater Quality Management	Stormwater Quantity Management
Roof areas (Average 35% coverage)	Up to 1 in 10 yr ARI	Specify roof material and treatment via pre-treatment device and filtration in on-site soakage system.	On-site disposal to ground.
	> 1 in 10 yr ARI	Water quality volume treated in on-site soakage system with excess flows to off-site system	Excess flows discharged directly to off-site system via piped or swale connection
Yard areas (Average 65% coverage)	Up to 1 in 10 yr ARI	All flows through sumps and water quality volume to a Humeceptor type device.	Off-site system receives discharge at a controlled rate (14 l/s/ha) after attenuation.
	> 1 in 10 yr ARI	Water quality volume to on-site treatment and additional flows to off-site system	Off-site system via an overflow weir.

2.4 Off-Site System

The off-site system has been designed to meet a level of service based on mitigating the quantity impact of the stormwater runoff on the downstream Raupare catchment as well as providing some additional quality protection where stormwater is to be disposed of by infiltration to the groundwater system.

Quantity

The off-site system has been designed to ensure that flows from all events up to a 1 in 10 yr ARI are contained within the swales with a minimum freeboard of 0.1m. For all events up to in a 1 in 10 yr ARI, all stormwater will be disposed to ground in the infiltration basin with zero runoff to the downstream catchment.

Figure 3 indicates likely locations for swale overflows as well as the routes for surcharge flows from the infiltration basins to the head of nearby HBRC drains within the Ruapare catchment. These flows are anticipated to be indistinguishable from the current greenfield flows and would be occurring at a time during which the catchment is already inundated.

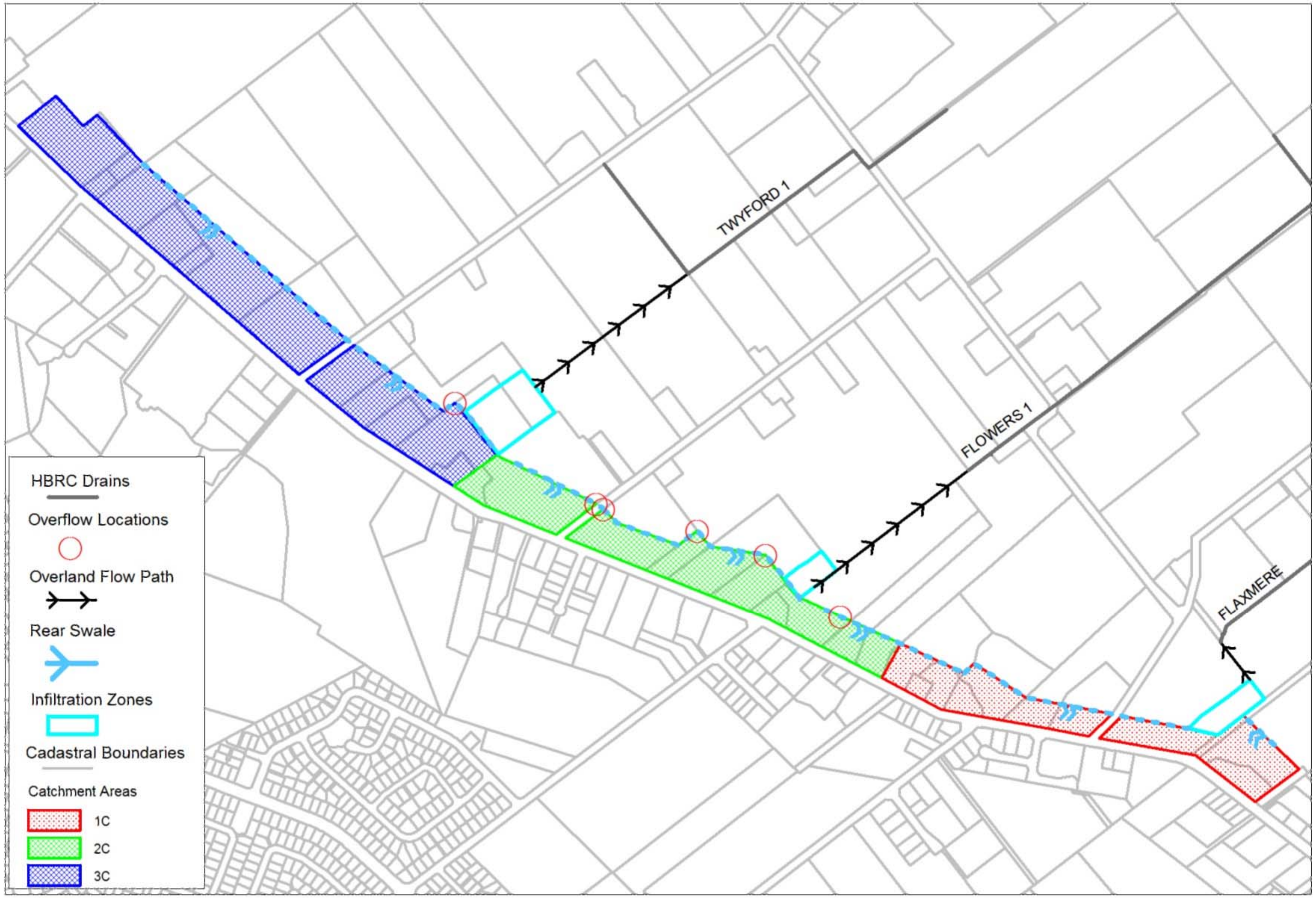

Figure 3 :Overflow Locations and Routes for Surcharge Events

Quality

Some treatment over and above that provided by on-site systems will be provided by the swale and unsaturated zone in the infiltration basin and will enhance protection of the groundwater from contamination.

The intended level of service for the swale and infiltration basin components of the off-site system is set out in Table 2-3 below.

Table 2-3 : Level of Service for Off-Site System

Surface	Level of Service	Stormwater Quality Management	Stormwater Quantity Management
Swale	Up to 1 in 10 yr ARI	Some limited sediment and hydrocarbon removal by vegetation in the swales.	Controlled All flows conveyed within the swale with freeboard of 100mm, based on 14 l/s/ha design capacity.
	> 1 in 10 yr ARI	Some limited sediment and hydrocarbon removal by vegetation in the swales.	Majority of flows retained in swale up to 50 yr ARI with zero freeboard.
Infiltration Basin	Up to 1 in 10 yr ARI	All stormwater will be filtered through the unsaturated soil zone beneath the basin.	All flows retained within the basin and disposed to ground.
	> 1 in 10 yr ARI	Additional sediment removal in fore-bay and some polishing treatment through filtration through the unsaturated soil zone beneath the basin	Flows attenuated in infiltration basin, surcharge flows for long duration events > 50 yr ARI discharged to downstream HBRC drains.

3 Design Assumptions

3.1 Rainfall and Storm Duration

3.1.1 Rainfall Data

The rainfall data used for design and modeling assessment of the capacity of the stormwater management devices was HIRDSv3 (High Intensity Rainfall Design System, developed by NIWA) with the chosen location being the junction of Twyford Road and Omahu Road. Design rainfall totals were extracted from this to represent the current rainfall (Table 3-1) and that expected in 2090 due to the predicted effects of climate change (Table 3-2). Provision for climate change can be made in two ways - built in at initial construction or with upgrade capability designed in. For the purposes of modeling we have assumed the former case.

Table 3-1 : Rainfall (mm depth) – Current Rainfall Data

ARI	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
2 Year	5.8	8.7	11.1	16.8	23.4	39	54	74.4	86.4	93.6
10 Year	9.9	15.1	19.3	29.4	39.5	63.1	84.8	113.9	134.4	148.1
20 Year	12.2	18.4	23.5	35.5	47.2	75	99.6	132	153.6	172.8
50 Year	16.0	24.4	31.2	47.5	62.1	95.1	124.5	162.8	192.2	211.8

A predicted mean annual temperature increase of 2.1 degrees Celsius was the basis for the 2090 rainfall totals. The temperature increase of 2.1 degrees is tabled as the mid-range estimate in the MfE document *Climate Change Effects and Impacts Assessment: A Guidance Manual for Local Government in New Zealand* (2008). The expected 2090 rainfall depths shown in Table 3-2 represent increases ranging from 16.2 % for the short duration storms down to a 12.5% increase for the long duration events (e.g. 24 hour).

Table 3-2 : Rainfall (mm depth) – 2090 Rainfall Adjusted for Climate Change

ARI	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
2 Year	6.8	10.1	12.8	19.2	26.4	43.2	60	81.6	96	100.8
10 Year	11.6	17.5	22.1	33.3	44.8	71.4	96	129.6	148.8	165.6
20 Year	14.2	21.5	27.3	41.2	54.8	86.4	115.2	153.6	177.6	194.4
50 Year	18.7	28.1	35.9	54.1	71.2	109.8	145.2	189.6	220.8	244.8

3.1.2 Storm Duration

The stormwater assessment has looked at storms of duration from 1 hour to 3 days for return periods of 1 in 10 years, 1 in 20 and 1 in 50 years. This was undertaken in order to understand the behavior of different parts of the proposed system such as the on-site detention system, swales and infiltration basins.

3.2 Stormwater Runoff Modeling

In order to assess stormwater flows and runoff volumes, a model of the proposed system was created in Infoworks Collection Systems (IWCS). Key assumptions and features of the model are described in the following sections.

3.2.1 Catchment Areas

As described in section 1.2 the rezone area has been divided into 3 catchments. A model was developed to assess the runoff flows from each of the three catchments. The original model had assigned the following areas:
Basin 1 = 7.2 ha, Basin 2 = 12.4 ha, Basin 3 = 17.9 ha; Total = 37.5 ha

Subsequent changes to the boundaries between the areas and exclusion of road runoff areas resulted in some changes to the final catchment areas which are reflected in figure 1:

Basin 1 = 8.1 ha, Basin 2 = 10.6 ha, Basin 3 = 17.8 ha; Total = 36.5 ha

As the final arrangement of the catchments and areas has still to be confirmed through the plan change process, the modelling work which used the earlier area split has been retained on the basis that while flows and volumes may change slightly these do not materially affect the conclusions of the work. A revised model run can be undertaken once the final area split and stormwater management strategy is confirmed.

3.2.2 Onsite System

Some of key assumptions used in modelling the stormwater runoff flows are described below. A more detailed list of key model assumptions is included in Appendix A.

Generic On-site Lot
Based on a review of similar light commercial development land within the Hastings area, the following have been used for a generic lot:

- Generic lot size of 5000 m^2 (50m x 100m)
- Building roof area of 35% or 1250 m^2
- Yard area of 65% or 3750 m^2
- Conservative assumption of 100% hard surface

Refer to Appendix C for an assessment of the site coverage and impervious surfaces investigations completed within Hasting Industrial areas.

Detention and Infiltration
- On-site infiltration is based on 400mm/hour (allowing for 1.5 safety factor reduction from assessed 600mm/hr infiltration capacity)
- Yard discharge to swale limited to 14 l/s/ha up to 1 in 10 ARI

3.2.3 Swales and Pipes

- Swale locations are expected to be along the rear boundaries of the properties as shown in Figure 1.
- Infiltration in the swale is assumed to be 10 mm/hour over the wetted area of the swale, once the swale is mature. The roughness for the swale capacity is based on a Mannings coefficient of 0.04, based on a trapezoidal section as shown in Figure 5 with short grass cover. Final design of the swale will include assessment of a range of roughness values to reflect the likely maintenance regime.
- Invert levels and grades are nominal at this stage and will need to be revised at final design once a final route is confirmed
- Swale reserve is based on a minimum 6m wide reserve with reserve increasing with swale capacity.
- Provision of at least 100mm freeboard on flow depth in swales for all Q10 events.

3.3 Model Results

Peak flows arriving at each of the three infiltration basin locations (refer figure 1) for the three return periods of 1 in 10, 1 in 20 and 1 in 50 years were calculated and results are tabulated in Table 3-3. Key findings:

- Peak flows during all events up to 1 in 10 ARI are less than 14 l/s/ha because of swale attenuation and infiltration e.g. Area 3 peak flow is 224 l/s rather than 17.9 ha x 14 l/s/ha = 250 l/s.
- In all three catchments the peak flows occur during the 6hr event at the 1 in 50 year ARI (see underlined values).
- The proposed swale cross-section with a 3 m bottom width (figure 5) has the potential to convey the majority of runoff from rainfall events up to 1 in 50 yr ARI allowing for zero freeboard

Table 3-3 : Peak Discharge for each Sub-Catchment including Climate Change Allowance m^3/s

ARI	1hr	2h	6h	12h	24h	48h	72h
Area 3 / Pond 3							
10 Year	0.224	0.224	0.224	0.224	0.224	0.146	0.103
20 Year	0.327	0.348	0.224	0.224	0.224	0.176	0.125
50 Year	0.611	0.662	**0.819**	0.471	0.224	0.222	0.159
Area 2 / Pond 2							
10 Year	0.146	0.146	0.146	0.146	0.146	0.095	0.067
20 Year	0.254	0.248	0.146	0.146	0.146	0.114	0.082
50 Year	0.485	0.462	**0.591**	0.327	0.146	0.144	0.103
Area 1 / Pond 1							
10 Year	0.104	0.104	0.104	0.104	0.104	0.0679	0.047
20 Year	0.165	0.158	0.104	0.104	0.104	0.082	0.058
50 Year	0.28	0.313	**0.377**	0.219	0.104	0.103	0.74

3.4 Infiltration Rates

3.4.1 On-site Soakage Systems

While the surface soils of the re-zone area are largely described as Twyford (silty loam or sandy loam), there is significant variability in soil texture. A narrow strip of land immediately adjacent to Omahu Road is mapped as Omahu soils being an extension of the soils found along the western side of Omahu Road. Soil investigation work to measure in-situ infiltration rates in the proposed locations for the infiltration basins has found coarser textured sands and gravels occur more frequently in the south-eastern parts of the re-zone development. This is confirmed by historical borehole logs which confirm the presence of coarse sands and gravels at shallow depth.

Infiltration rates in the Omahu soils are described as very rapid, while rates in the Twyford soils are described as very good. Work completed on surface infiltration rates for surface soils across Hastings is included in the report '*Hastings District Council: Soils of Hastings City and their Infiltration Rates and Permeabilities by* Landcare Research, October 2006. The report describes a measurement of infiltration rate at St Leonards Park of in excess of 288mm/hr. The soils are described as being in the Omahu soil series,

Soil infiltration tests were also conducted in the Irongate area in soils ranging in texture from silt to coarse gravel. Infiltration measurements ranged from 240mm/hr for silts to as much as 1800 mm/hr for sand and medium gravel.

On the basis of the bore hole logs and existing soil mapping, estimates of infiltration are based on the following assumptions:

- Well-drained sands and gravels are readily accessible, i.e. reasonably shallow soakage systems can be located within the majority of the lots to take advantage of more permeable material.
- The groundwater table is at least 1.5m below ground level
- Soakage disposal can be achieved by infiltration chambers using manufactured plastic modules such as the Humes “RainSmart” module system and wrapped with geo-textile. The chamber combines storage volume with soakage area and is able to be installed beneath the yard pavement.

Based on the range of infiltration tests undertaken within the re-zone area and the assumptions outlined above, a rate of 400mm/hr was selected for assessing generic soakage system design. This is equivalent to 65% of the lowest of the 4 tests completed in sand or gravel soils in the Irongate zone (refer Table 4-1).

Table 3-4 : Summary of Soil Soakage Tests Results for Irongate

Soakage Test No.	General Description of Predominant Soil Type	Results (mm/hour)
1	Sandy fine to coarse GRAVEL	600 – very rapid
2	Fine SAND with occasional medium gravel	1,800 – very rapid
3	Gravely (fine to very coarse) fine SAND	1,050 – very rapid
4	Gravely (fine to coarse) fine to medium SAND	600 – very rapid
5	SILT	240 – rapid

Given the variability in soil texture and infiltration rate across the re-zone area it is expected that infiltration rates specific to individual development areas would need to be determined by investigation when undertaking detailed design. In some cases on-site soakage systems may need to be located closer to the Omahu Road frontage, which will require alternative design of overflow systems to drain excess flows to the swale at the rear of the lots.

3.4.2 Off-Site Infiltration Ponds

The areas or zones in which the the stormwater infiltration basins are to be located (as shown in figure 1) have been chosen in line with the following considerations:

- Areas are located outside the re-zone area to keep land acquisition costs to a minimum and ensure the majority of the re-zone area is available for development
- Areas are down slope of the swale to enable gravity stormwater servicing for the majority of the re-zone area, with swale alignment based on preliminary design using LIDAR data for the zone. Some areas at the south-eastern end of the re-zone area may require low lift pumping.
- The re-zone area has been divided into three to enable cost effective sizing of the swale network delivering stormwater to the basins and to provide for distributed infiltration to reduce impacts of concentrated disposal of stormwater,

In the light of the wide variation in both soil texture and infiltration rates reported from regional studies and tests in other locations, specific field assessment work was undertaken to provide greater confidence in respect of the in-situ soil characteristics and infiltration rates for sizing of basins in the three proposed locations. General findings from this assessment include:

- The measured infiltration rates are in general lower than the values reported in the Landcare Research report.
- Measured infiltration rates vary significantly over relatively short distances (10’s of metres) and therefore may vary from one side of an infiltration basin to another and in line with changes in the texture of the sub-soils
- Infiltration rates increase with movement in a south-easterly direction from location 3 towards location 1 and in a southerly direction from the zone boundary towards Omahu Road. This increase in

infiltration rate coincides with an increase in the occurrence near the surface of coarser sand and gravel layers.

Locations of soil sampling are summarised in Table 3-5 and shown on figure 4. Test pits are referenced to each of the re-zone and infiltration areas (1, 2 and 3).

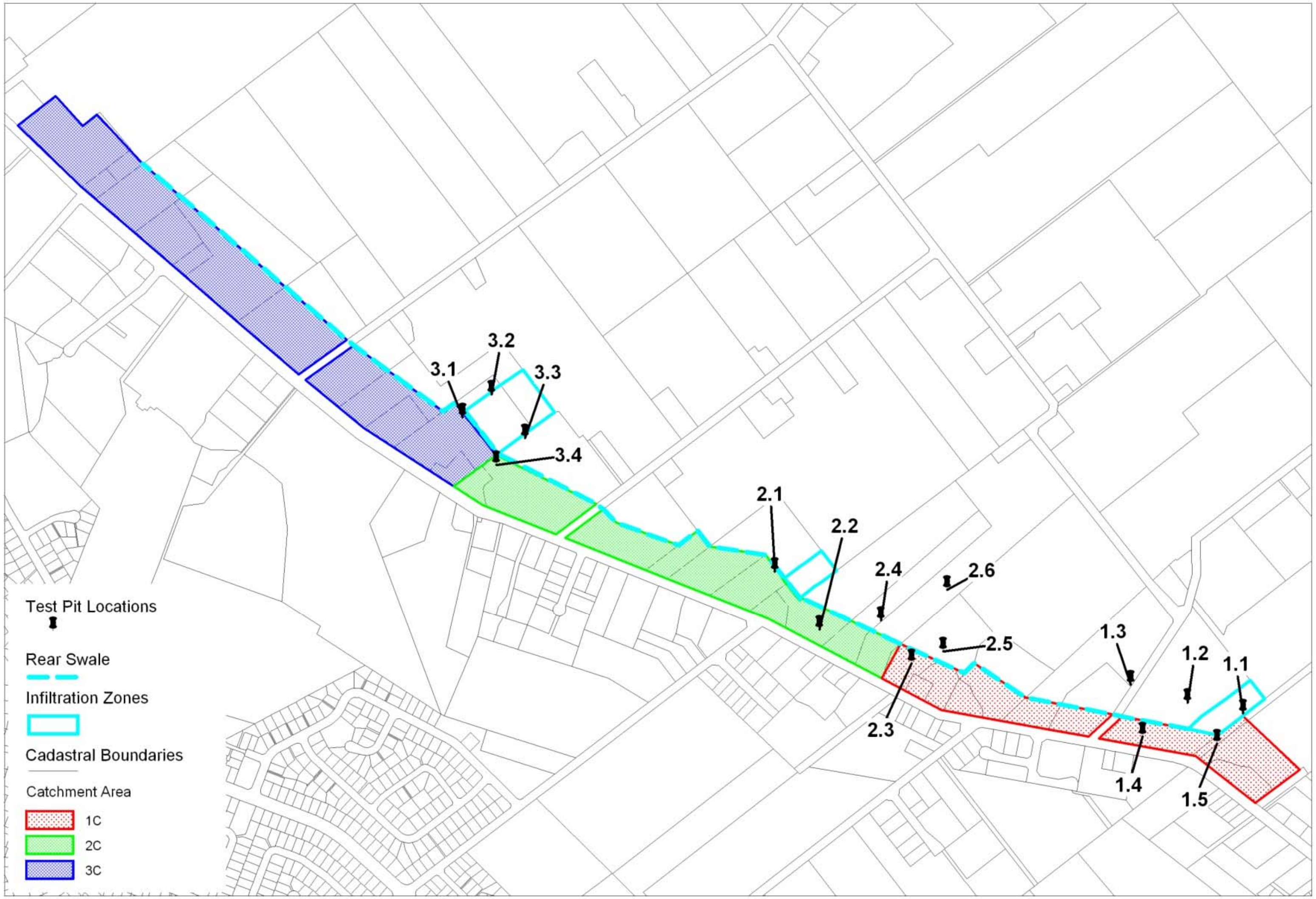

Figure 4 : Locations of Test Pits and Infiltration Assessments

Table 3-5 : Key Soil Features in Proposed Infiltration Zones

Infiltration Zone	Test Locations	Soil Texture Description	Water Table
1	1.1 and 1.5	Brown silts with small blue grey pockets at 1.6m. Small areas of shallow gravel 5-20mm size.	Water table encountered at 2 m below surface
2	2.1	Brown silts with blue grey sandy-silt from 1.8m	Water table at 2.7 m below surface
2	2.4	Brown silts over shallow gravel	Water table at 2.6 m below surface
3	3.1, 3.2, 3.3 and 3.4	Brown silts down to 3 m with small lenses of blue/grey sandy silt from 2 m depth	No water table encountered within 3 m of surface

For each basin area a small number of field infiltration tests were completed. The tests included pre-wetting and represent steady state measurements of infiltration in small confined excavations. The range in measured infiltration rates correlate with local variation in the texture of the sub-soil material. The following principles have been applied to select an infiltration rate for basin design sizing:

- The minimum measured infiltration rate over all the tests for a particular location has been used
- To allow for differences between measured and long term saturated infiltration rate as well as some decline in infiltration rate over time, a design rate of 50% of the minimum measured rate has been assumed (refer Table 3-6).

While the field assessment for basin 1 identified a design infiltration rate of 0.45 m /hr, a maximum rate of 0.225 m/hr has been assumed on the basis that the basin will be lined with a topsoil filter layer which will reduce the long term infiltration rate.

Table 3-6 : Infiltration Rate Measurements in Infiltration Zones

Infiltration Field Assessments					
Basin Location	No of test	Max Rate (m/hr)	Min Rate (m/hr)	50% of min	Design Rate (m/hr)
1	2	2.2	0.9	0.45	0.23
2	2	6	0.24	0.12	0.12
3	4	0.3	0.06	0.03	0.03

3.5 Quality Management

In respect of the assessment of stormwater quality it has been assumed that the effectiveness of stormwater treatment measures can be assessed by using the Auckland Regional Council (May 2006) contaminant load model.

3.5.1 Residential Baseline

Based on the key level of service criteria being achievement of better than residential baseline stormwater quality, the composition of the typical Hastings residential site has been based on the following drawn from a review of two sample areas of existing residential development in Hastings.

- 40% of grass and gardens (4000 m^2 per ha – including reserves)
- 15% roads (900 m^2 < 1000 vpd, and 600 m^2 of 1000 – 5000 vpd)

- 20% paved surface (2000 m^2 concrete)
- 25% roofs (2500 m2 comprising a mix of various materials, but with a majority of colour-steel, and smaller areas of painted galvanised and clay or tile products)

The resulting contaminant load from the baseline residential area is summarised in Table 3-7.

Table 3-7 : Residential Contaminant Loads – 60% impervious

Bottom of Site out-fall Loads (kg a^{-1})				Average yields TSS	Zn	Cu	TPH
TSS	Zn	Cu	TPH	kg ha^{-1} a^{-1}	g ha^{-1} a^{-1}	g ha^{-1} a^{-1}	g ha^{-1} a^{-1}
214.1	0.6	0.1	0.4	214	597	50	419

3.5.2 Treatment Assumptions

For the purpose of assessing contaminant loads for the typical lot within the re-zone area the following was assumed as regards surfaces and treatment devices:

- 35% roof area 1750 m^2 constructed from colour steel or equivalent material
- 65% yard area 3750 m^2 assumed to be equivalent to < 1000 vpd roading reflecting goods and staff and customer parking

Specific on-site treatment comprises the following:

- Roof runoff – first management option – stabilised roof materials
- Roof runoff – second management option – some form of pre-treatment prior to the ground soakage system
- Yard runoff – first management option – sumps cleaned 2 times per year
- Yard runoff – second management option – Humes interceptor or similar device

4 On-site System Design

4.1 Design Approach for Example Lot

The requirements for on-site stormwater management have been assessed by considering an example site with an area of 5000 $m^{2.}$ Key assumptions and features of the example lot include:

- Site area is 50m x 100m, comprising a 50m frontage along Omahu Road and a depth of 100m.
- Building will be a typical portal framed structure with a central ridge and downpipes along each side.
- The building will sit in one corner and have dimensions of 30 m along the frontage and 58 m depth to give an area of 0.175 ha.
- Terrain is assumed to fall at a grade of 1% from the frontage so that the rear of the site is 1m lower than the frontage.
- Building floor level is assumed to be 0.2m lower than the frontage, but 0.8 m higher than the existing level at the rear of the site.
- To form the site and establish the floor level, the rear of the site will need to be filled by an average of 0.5m and the front of the building excavated by 0.3m.
- The yard area around the building will fall from the floor level.

A plan of the example lot follows as Figure 5.

Stormwater Runoff
Stormwater runoff from both the roof and yard areas will drain to separate systems.

Roof runoff will be piped to a storage and ground soakage system located on the site. While a below ground storage and soakage system has been proposed other options may be used including above ground tank storage with infiltration beds or basins. It is expected most developments will look to locate the soakage system in the paved area. The location of the soakage system may vary depending on:
- Fall across the site
- Location of suitable high infiltration rates soils
- Building and hard stand configuration on the lot

Low impact design including the use of landscaped areas as rain gardens and detention/soakage systems is possible but will be at the discretion of the lot developer and owner. Given the high value of the land, the consent is based on 100% hard surface with engineered stormwater management.

Roof stormwater in excess of the capacity of the on-site soakage system will be discharged directly to the Council off-site swale system by a pipe or open channel connection.

Yard water will drain by means of a shallow dishing of the pavement across the front of the building leading to a kerb and channel along the side of the lot. Stormwater will be directed through two treatment devices in series before discharge to the swale section of the off-site system. Excess flows will pass over a weir to the swale drain at the rear of the zone. Some lots may need to pipe their overflow via separate easements, while some low lying areas may need to use low level pumping.

It has been assumed that earthworks will ensure that there is sufficient fall to allow for the operation of a proprietary treatment device.

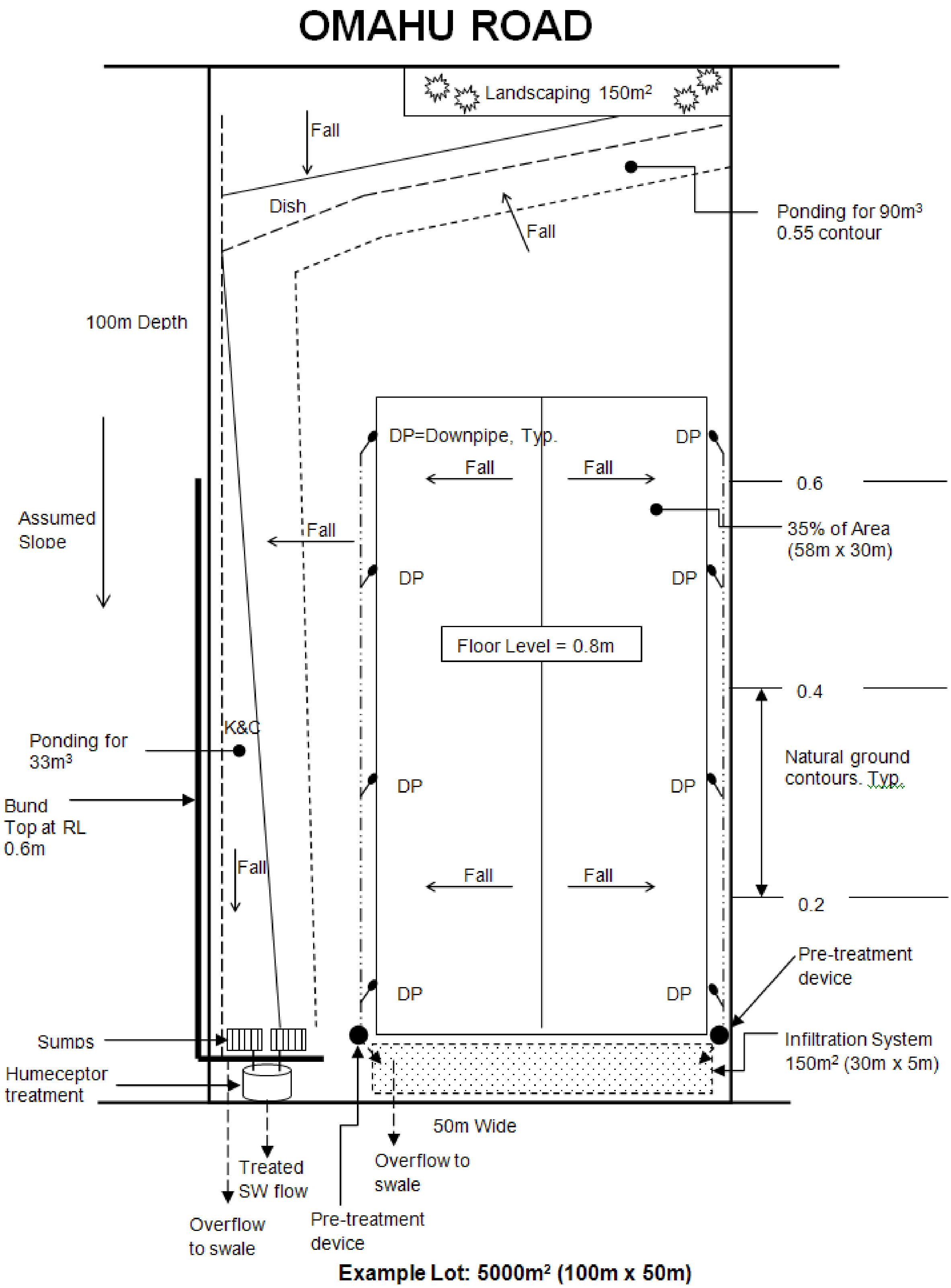

Figure 5 : Example On-site Lot

4.2 Roof-water Storage and Soakage System Design

In line with the design criteria, the system is designed to cope with all runoff from any event with a 10 year ARI, using an assumed infiltration rate of 400 mm/hr.

Downpipes along each side of the building connect to a drain along each side, both of which run to an infiltration chamber located within the lot boundary and sized for a 10 year event. Larger flows will overflow from the infiltration chamber to discharge separately to the offsite system.

The design of the infiltration system is based on a balance of the volume of storage and infiltration or soakage area. The required storage is the difference between the runoff volume and the volume soaking into the soil during the particular event. It has been assumed that storage should be no more than 1 m deep and preferably around 0.5 m deep to minimise construction costs. A range of disposal areas were assessed to arrive at an acceptable design.

In Table 4-1, the amount of storage volume is determined for an assumed soakage area of 50m^2. The assumed infiltration rate of 400mm/hr means the disposal through soakage will be 20m^3/hr. The rainfall depths are from HIRDSV3 for 2090. Rainfall depths are multiplied by 1750m^2 to give the volume of runoff for each rainfall event duration. The volume of soakage is then shown and subtracted to give the storage volume which is needed.

Table 4-1 : Soakage Area and Storage Volume for 10 Year Event

ARI	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
Rainfall 2090	11.6	17.5	22.1	33.3	44.8	71.4	96	129.6	148.8	165.6
Runoff Vol m^3	20.3	30.6	38.7	58.3	78.4	125.0	168.0	226.8	260.4	289.8
Soakage Vol m^3	3.3	6.6	10.0	20.0	40.0	120.0	240.0	480.0	960.0	1440.0
To Storage m^3	17.0	24.0	28.7	38.3	38.4	5.0	0.0	0.0	0.0	0.0

The required storage volume is the highest value shown, i.e. 38.4. A figure of 40m^3 has been adopted.

A similar calculation for a 50 year event shows that the 40m^3 storage would be filled by the peak 15 minute event. As a factor of safety, it is proposed to allow for an infiltration area of 1.5 times the calculated requirement, i.e. 75 m^2. This would allow for some reduction or variability in long-term sustainable infiltration capacity. With an area of 75 m^2 and a volume of 40m^3, the required effective storage depth will be 533 mm. This indicates that it should be possible to provide storage which is above the water table at all times.

4.3 Yard System Storage and Detention Design

The system has been designed to the following criteria:

- All stormwater falling in an event with a frequency of 1 in 10 years event will be detained and discharged, via treatment devices, to the off-site Council system at a rate no greater than 14 l/s/ha.
- Any runoff in excess of that stored for the peak 1 in 10 year event will spill directly without treatment to the off-site swale (NB: the initial first flush runoff containing the majority of the storm contaminant load will have passed through the treatment train before this occurs).

The sizing of the ponding system is based on a balance of the volume of storage and outflow to the swale system. The required storage is the difference between the runoff volume and the volume discharging at the maximum rate of 14 l/s/ha or 7 l/s for the example lot.

In Table 4-2, the amount of storage volume is determined for a discharge rate of 7 l/s from the 0.5 ha lot. The rainfall depths are from HIRDSV3 for 2090 so include a climate change allowance and are multiplied by 3250m^2 to give the volume of yard runoff for each duration. The volume of discharge to the swale is then shown and subtracted to give the storage volume which is needed.

Table 4-2 : Yard Storage Volume for 10 Year Event

ARI	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
Rainfall 2090	11.6	17.5	22.1	33.3	44.8	71.4	96	129.6	148.8	165.6
Runoff Vol m^3	37.7	56.9	71.8	108.2	145.6	232.1	312.0	421.2	483.6	538.2
Discharge Vol m^3	4.2	8.4	12.6	25.2	50.4	151.2	302.4	604.8	1209.6	1814.4
To Storage m^3	33.5	48.5	59.2	83.0	95.2	80.9	9.6	0.0	0.0	0.0

Storage is available by ponding in the area alongside the building. The maximum volume of 95m^3 is available by ponding within the yard area of the lot. One scenario proposed comprises a combination of water ponding in the bottom corner to a depth of up to 0.35 m with an elongate area of 7m x 80m in area to a depth of 0.2 m. If depths are impractical the dished pavement could be replaced with a slot drain.

To achieve this ponded volume a wall or bund will be required to be formed across the low end of the storage. The lowest point in the corner is at existing ground level. Moving toward Omahu Road, the drainage invert would be cut below the existing ground and the wall or bund will have to extend roughly 60m along the side boundary.

Other design solutions can be developed to provide the required 95 m^3 of ponding on-site.

Assessment of the runoff flows during a 1 in 50 year event, indicates that the 95 m^3 of storage will provide for an event of just less than 30 min. For longer events, the additional flow and volume will spill to the off-site swale.

4.4 On-site System Stormwater Treatment Analysis

In line with the assumptions outlined in section 3.6, an assessment of the effectiveness of the proposed treatment in the on-site system and the swale system has been completed, using the ARC Contaminant Load Model. To understand the influence of various parameters a number of scenarios were assessed including:

- Scenario 1. Both roof and yard stormwater treated via a two stage treatment train before discharge to the off-site swale (65% yard: 35% roof)
- Scenario 2. Only yard water being treated via a two stage treatment train with all roof water going to ground soakage (65% yard: 35% roof)
- Scenario 3. Scenario 2 allowing for additional treatment of the yard water discharge in the off-site swale system (65% yard: 35% roof)
- Scenario 4: Scenario 3 with 75% yard and 25% roof coverage
- Scenario 5: Scenario 3 with 55% yard and 45% roof coverage

Scenarios 4 and 5 provide an assessment of the sensitivity of the treatment performance to altered ratio's in yard and roof area on the site. The final contaminant loads are presented in Table 4-3 in terms of average yields per hectare per year for the full lot area.

Table 4-3 : Final Contaminant Loads – Various Scenarios

Scenarios	Average yields TSS	Zn	Cu	TPH
	$kg\ ha^{-1}\ a^{-1}$	$g\ ha^{-1}\ a^{-1}$	$g\ ha^{-1}\ a^{-1}$	$g\ ha^{-1}\ a^{-1}$
Hastings residential baseline – 80% impervious	**185**	**741**	**85**	**805**
Scenario 1 . Yard and roof treated prior to discharge	**23**	**215**	**22**	**447**
Scenario 2. Yard water treated and roof water to on-site soakage	**5.2**	**75.4**	**19.5**	**446**
Scenario 3. Scenario 2 with allowance for swale treatment	**3.9**	**67.6**	**16.3**	**380**
Scenario 4. Scenario 3 with 75% yard and 25% roof	**4.5**	**78**	**18.8**	**438**
Scenario 5: Scenario 3 with 55% yard and 45% roof	**3.3**	**57.2**	**13.8**	**321**

Key findings include:

- For all scenarios the average yields of suspended solids, metals and TPH are reduced by treatment to levels significantly below that of the Hastings Residential Baseline.
- The sensitivity analysis to assess the impact of yard coverage varying by ± 10% from 65% to 55 or 75% indicates relatively small changes of ± 15% in contaminant loads.
- The swale provides a measureable improvement in treatment for all four indicator parameters
- The treatment provided is considered more than adequate to achieve the required level of service and provide an additional level of protection for the aquifer underneath the off-site infiltration basin.

5 Off-site System Design and Assessment

5.1 Design Approach

As outlined in sections 2.2 and 2.3 the key level of service criteria for the off-site system includes:

- All flows for events up to 10 year ARI (2090 rainfall) are contained within the swales and disposed to ground in the infiltration basin with zero runoff to the downstream catchment
- For events from 10 year ARI to 50 year ARI (2090 rainfall) the majority of flows can be accommodated within the swale allowing for zero freeboard and some discharge of greenfield runoff to the local drainage network at very high flows
- For events from 10 year ARI to 50 year ARI, the infiltration basins are sized to contain the maximum volume in any 10 year ARI. Once the volume stored reaches the maximum level then a fixed discharge equivalent to the greenfield flow (current 2010) from the design event would be permitted
- For events beyond a 1 in 10 ARI overflows may occur from the swales and the infiltration basins, although these are not likely to be significant until events exceed a 1 in 50 year ARI. Figure 2 indicates likely routes for swale overflows and surcharge flows from the infiltration basins.

The strip along Omahu Road has been divided into three catchments with flows from each directed to a specific infiltration area as indicated in Figure 1. Stormwater runoff from road reserve along Omahu Road is assumed to discharge elsewhere and is not included in the stormwater assessment.

5.2 Off-Site System Modeling

Table 5-1 below summarises the modeled peak flows and total volume of stormwater for each of the three catchments identified in figure 1. Both the current Greenfield runoff (without an allowance for climate change) and the anticipated developed situations (with an allowance for climate change) have been modeled. Results for the 10, 20 and 50 yr ARI events of different durations are provided in Table A1 in Appendix B.

Key findings include:

- Events when swale flows exceed the on-site peak discharges of 14l/s/ha are highlighted in blue and typically peak for the 2 hr 20 yr and 6hr 50 yr ARI events.
- Runoff volumes peak during the 2 day storm events.

5.3 Swales

A typical detail for the swale is show below in Figure 6. This represents the maximum size for the swale (5.5 m) just prior to discharge to the infiltration basin. For much of the swale extent a swale width of less than 6 m will be adequate. The preferred design comprises a traditional trapezoidal section, with shallow slopes such that mowing of the bed and sides can be achieved from within the swale. Screening requirements in respect of planting of a shelter belt have yet to be confirmed.

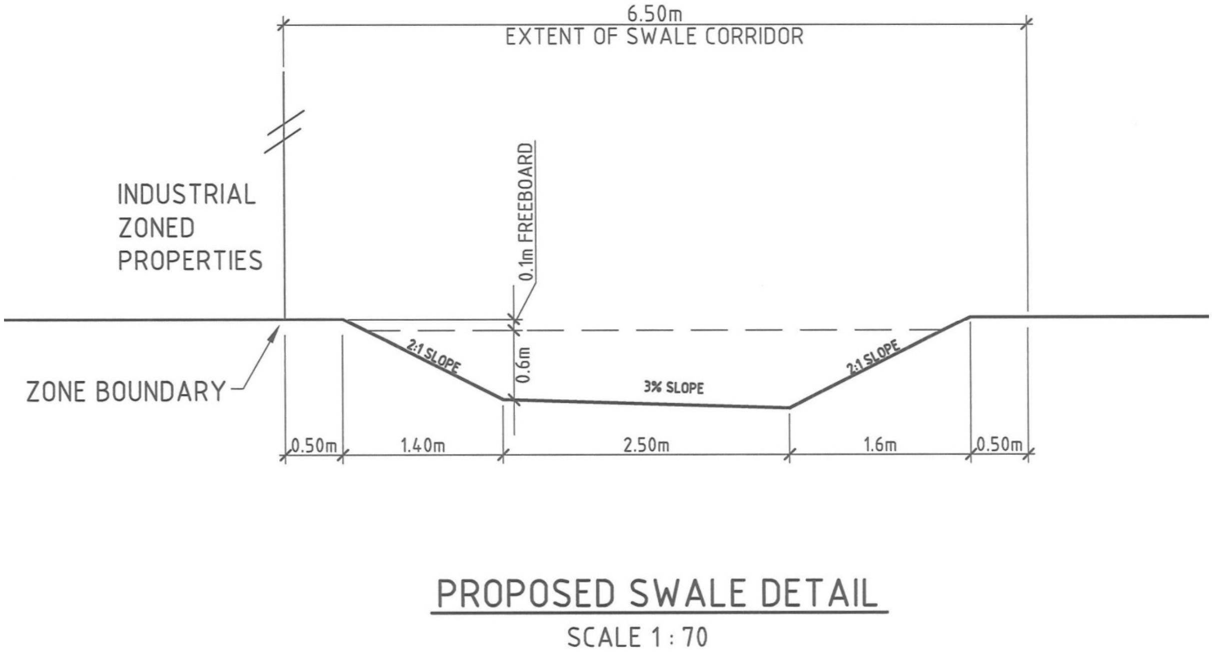

Figure 6 : Preferred Swale Design Option

Key assumptions in respect of the swale system design include:

- Infiltration in swale is 10 mm/hour over wetted area of swale
- Mannings roughness coefficient assumed to be 0.04
- Invert levels/Grades: A maximum water depth of 0.7 m has been assumed, with grades ranging from 1 in 200 to 1 in 600. Some sections of the swale will need to be incised while others will require banking. Detailed inverts and grades will need to be revised once a final route is confirmed.
- Swales are assumed to be located in a strip of land beyond the boundary of the proposed rezone area.

Steeper grades will be possible (1 in 200 to 1 in 300) at the rear of area 3, while flatter grades (1 in 500 to 1 in 600) are likely in the lower parts of the development at the rear of areas 1 and 2.

A brief assessment of the flow capacity of the proposed swale cross-section for the maximum design depth of 0.6 m (0.7m with freeboard of 0.1 m) is summarised in Table 5-1 for a range of available grades.

Table 5-1 : Flow and Velocity for Range of Swale Grades at Maximum Depth

Gradient	1 in 200	1 in 300	1 in 400	1 in 500	1 in 600
Velocity of Flow m/s	**0.87**	**0.71**	**0.62**	**0.55**	**0.50**
Flow Rate m^3/s	**1.16**	**0.94**	**0.82**	**0.73**	**0.67**

Key findings

- The proposed standard swale has adequate capacity for the modelled peak flows arriving at the swale for all events up to a 10 year ARI with 0.1 m freeboard, and the majority of events up to a 1 in 50 year ARI with zero freeboard. The maximum flow arises in area 3 during the 6 hr 50 year ARI event.
- There is some limited opportunity to reduce the swale section and easement width in the upper sections of the each sub-catchment, however access and maintenance considerations require a minimum easement width.
- Overflows, up to greenfield rates, to road side drains for events between 20 and 50 yr ARI could be provided by way of side exit weirs just upstream of road crossing culverts however these have not been modelled

5.4 Infiltration Basins

An assessment of the design infiltration pond areas based on a maximum water level of 1 m under a range of level of service scenarios was completed. Maximum volumes of storage were calculated for each of the three sub-catchment areas based on the following assumed infiltration rates. The calculation of maximum basin storage volumes using the outputs from the model, are summarised in Appendix B in Table A2.

Swale infiltration rate – 10mm/hr

Basin 3	depth 1 m	infiltration rate 30 mm/hr
Basin 2	depth 1 m	infiltration rate 120 mm/hr
Basin 1	depth 1 m	infiltration rate 225 mm/hr

Two scenarios were considered:

- pond sizes were determined at each of 10 year, 20 yr and 50 yr ARI based on providing for zero discharge to the downstream catchment (refer table 5-2)
- pond sizes for the 50 year ARI with zero discharge were modified to account for allowable greenfield discharges for events beyond a 1 in 10 year frequency (refer table 5-3)

Final pond location has yet to be determined, however infiltration zones have been identified within which specific infiltration ponds will be constructed. Final pond position will be determined following detailed swale design to ensure the majority of lots can discharge stormwater by gravity and consultation with landowners.

Key features of the proposed infiltration basin or dry pond include:

- graded swale entry with rip-rap protection and fore-bay to capture swale sediment load
- side slopes of 2 horizontal to 1 vertical
- maximum pond water depth of 1 m
- intermediate bench of 1m width at 0.5 depth as a safety and management aid
- grass and soil invert cover to basin floor to control rapid drainage (relevant for basins 1 and 2)
- pipe outlet for surcharge flows and permitted greenfield discharge
- small perimeter bund to provide for a 0.3 m freeboard

A general bank cross-section for the proposed infiltration basin / dry pond is shown in Figure 7.

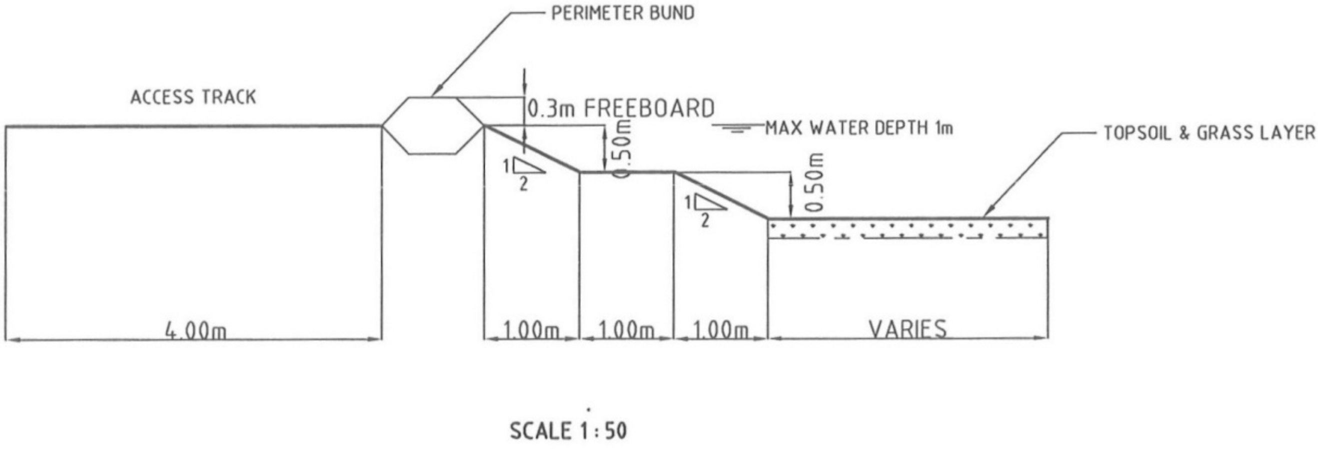

Figure 7 : Infiltration Basin or Dry Pond Typical Bank Cross-Section

Table 5-2 : Infiltration Basin Sizing with Zero Discharge

	Level of Service	Pond Volume (m3)	Critical Event
Area 3	1 in 10 year	8200	24 hr
	1 in 20 year	10000	24 hr
	1 in 50 year	12200	24 hr
Area 2	1 in 10 year	3000	6 hr
	1 in 20 year	3700	12 hr
	1 in 50 year	5000	6 hr
Area 1	1 in 10 year	1800	2 hr
	1 in 20 year	2050	2 hr
	1 in 50 year	2600	6 hr

Adjusting Infiltration Pond Sizing Allowing Greenfields Discharge

The possible impact of adjusting the pond sizing to account for discharge of typical greenfield flows for events greater than a 1 in 10 year event has been assessed on the following basis:

- No greenfield discharge is allowed from any basin until the stored volume reaches the maximum modeled for a 10 ARI event e.g. 8200 m^3 for Area 3 in a 6 hour event.
- Once the peak volume is reached a fixed discharge would be permitted equivalent to the greenfield flow for the critical 50 year ARI event which determines the maximum storage capacity e.g. Greenfield Flow = 0.08 m^3/s for Area 3 during a 1 day event. The Greenfield flow for a 24hr event with 50 yr ARI was chosen as this is beyond the critical duration for the receiving Raupare catchment.

The effective reduction in storage volume for a 1 in 50 year event depends on the time taken to reach the 10 year ARI stored volume during the higher frequency event. The minimum storage volume reductions are listed in table 5-3 and are based on 6 hours discharge in a 24 hour event for area 3 and 4 hours discharge during a 6 hour event for areas 1 and 2. The area for each infiltration basin includes an allowance for any minor embankment and an access strip around the outside of the basin for maintenance access of a minimum 7 m.

Table 5-3 : Infiltration Basin Sizing with Greenfield Flow Discharge

Sub-Catchment	Maximum Pond Volume (m^3)	Greenfield Flow * (m^3/s)	Greenfield Volume Reduction (m^3)	Reduced Pond Volume (m^3)	Land Area (ha)
Area 3	**12,200**	**0.08**	**1,750**	**10,450**	**1.4**
Area 2	**5,000**	**0.053**	**760**	**4,240**	**0.6**
Area 1	**2,600**	**0.038**	**200**	**2,400**	**0.365**

*Greenfield flow is that occurring during a 24 hr duration event with a 50 year ARI

Any greenfield discharge or surcharge flow will be directed to the existing HBRC drains as shown in figure 3. This differs from the current greenfield runoff which is a spread or distributed discharge. However it is considered that the discharges for events of less than 50 yr ARI are modest and should have little impact in the receiving HBRC drains.

At this stage a decision on how the surcharge flows from the ponds will be conveyed to the HBRC drains has not been made.

Appendix A Model Assumptions

Roof Area

- Assessment is based on a 1750m^2 of roof area for a 0.5 ha lot (35%)
- Infiltration Chamber: 0.6m deep, 40m^3 storage for 75m^2 of floor area.
- Overflow to off-site swale based on a 225mm overflow pipe of 10m length with a slope of 0.01 m/m.

Yard Area

- Assessment is based on a 3250m^2 of paved area for a 0.5 ha lot (65%)
- Outlet comprises a paved area flowing through a treatment system limited to 14 l/s
- Outlet pipe is 150mm diameter and 10m length at a depth of 0.6m
- Storage area allows for 15% of the paved lot (475m^2) to be inundated to a depth of 200mm providing 95m^3 storage
- Overflow weir 3 m wide
- Overflow level 200mm above ground level (no slope assumed for paved catchment)

Greenfield Situation

- Slopes: Areas 1 and 3 assumed to have zero slope: Area 2 has slope of 0.003
- The surface is classed as "row crops" with a soil class of "2".
- Soil class 2 is a low runoff class (1 is very low, 5 is very high) with a water holding capacity index of 0.30.
- 0 mm initial loss
- Majority of rainfall is directed into the soil store until is reaches capacity when all rainfall then becomes runoff.

Swales and Pipes

- Roughness for swales based on Manning coefficient of 0.04
- Invert levels and grades are nominal at this stage and will need to be revised at final design once a final route is confirmed
- Swale reserve is based on a minimum 6m wide reserve with reserve increasing with swale capacity
- Roughness for pipes is based on 1.5mm Colebrook White
- Headloss inferred by IWCS (Infoworks Collection Systems – name of modeling software)

Runoff Model Parameters

- Model fast response and New (UK)
- Road Surface (10) Runoff routing of 1, fixed runoff volume, impervious, 0m slope, 0.000071m initial loss, 1 fixed coefficient and initial Loss of 1mm
- Roof Surface (20) Runoff routing of 1, fixed runoff volume, impervious, 0.5m slope, 0.000071m initial loss, 1 fixed coefficient and initial Loss of 1mm
- Grass Surface (21) Runoff routing of 4, new UK runoff volume, pervious, 0m slope, 0.002m initial loss and initial Loss of 1mm

Design Thresholds

- Provision of a minimum 100mm freeboard on flow depth in swales for all Q10 events
- Evaporation was not taken into account
- Local UCWI (Urban Catchment Wetness Index) was not taken into account since the runoff is predominantly fast response.
- Local antecedent depth not taken into account. (This is the rainfall that has fallen prior to the storm event).

Appendix B Tabulated Model Results

Table A1 **Modelling Results for Off-Site Runoff Flows and Volumes**

Table A2 **Tabulated Results for Infiltration Pond Sizing**

Table A1 : Modelling Results for Off-Site Runoff Flows and Volumes

		Area 3/Pond 3		Area 2/Pond 2		Area 1/Pond 1	
	Rainfall Used	(HIRDS)	HIRDS +CC	(HIRDS)	HIRDS +CC	(HIRDS)	HIRDS +CC
10 Year		Green	Develop	Green	Develop	Green	Develop
60 min	max flow (m3/s)	0.34	0.224	0.226	0.146	0.16	0.104
	flow vol (m3)		3635		2369		1687
2 hr	max flow (m3/s)	0.229	0.224	0.152	0.146	0.108	0.104
	flow vol (m3)		4878		3177		22634
6 hr	max flow (m3/s)	0.123	0.224	0.082	0.146	0.058	0.104
	flow vol (m3)		7884		5132		3661
12 hr	max flow (m3/s)	0.083	0.224	0.055	0.146	0.039	0.104
	flow vol (m3)		10574		6881		4909
24 hr	max flow (m3/s)	0.055	0.224	0.037	0.146	0.026	0.104
	flow vol (m3)		14037		9136		6503
2 days	max flow (m3/s)	0.033	0.146	0.022	0.095	0.016	0.067
	flow vol (m3)		15143		9897		6978
3 days	max flow (m3/s)	0.024	0.103	0.016	0.067	0.011	0.047
	flow vol (m3)		9914		6531		4539

		Area 3/Pond 3		Area 2/Pond 2		Area 1/Pond 1	
	Rainfall Used	(HIRDS)	HIRDS +CC	(HIRDS)	HIRDS +CC	(HIRDS)	HIRDS +CC
20 Years							
60 min	max flow (m3/s)	0.418	**0.327**	0.278	**0.254**	0.197	**0.165**
	flow vol (m3)		4784		3137		2204
2 hr	max flow (m3/s)	0.279	**0.348**	0.185	**0.248**	0.131	**0.158**
	flow vol (m3)		6386		4177		2946
6 hr	max flow (m3/s)	0.146	0.224	0.097	0.146	0.069	0.104
	flow vol (m3)		9558		6220		4439
12 hr	max flow (m3/s)	0.098	0.224	0.065	0.146	0.046	0.104
	flow vol (m3)		12743		8921		5918
24 hr	max flow (m3/s)	0.065	0.224	0.043	0.146	0.031	0.104
	flow vol (m3)		16900		10995		7842
2 days	max flow (m3/s)	0.038	0.176	0.025	0.114	0.018	0.082
	flow vol (m3)		18442		12041		8529
3 days	max flow (m3/s)	0.028	0.125	0.019	0.082	0.013	0.058
	flow vol (m3)		12221		8032		5627

50 Years							
60 min	max flow (m3/s)	*0.547*	**0.611**	*0.363*	**0.485**	*0.257*	**0.28**
	flow vol (m3)		6882		4539		3144
2 hr	max flow (m3/s)	*0.359*	**0.662**	*0.239*	**0.462**	*0.169*	**0.313**
	flow vol (m3)		9032		5950		3127
6 hr	max flow (m3/s)	*0.185*	**0.819**	*0.123*	**0.591**	*0.087*	**0.377**
	flow vol (m3)		12958		8491		5962
12 hr	max flow (m3/s)	*0.122*	**0.471**	*0.081*	**0.327**	*0.057*	**0.219**
	flow vol (m3)		16148		10511		7493
24 hr	max flow (m3/s)	*0.08*	0.224	*0.053*	0.146	*0.038*	0.104
	flow vol (m3)		21140		13751		9817
2 days	max flow (m3/s)	*0.047*	0.222	*0.031*	0.144	*0.022*	0.103
	flow vol (m3)		23457		15302		10884
3 days	max flow (m3/s)	*0.034*	0.159	*0.023*	0.103	*0.016*	0.074
	flow vol (m3)		15688		10289		7263

Table A2 – Tabulated Analysis for Infiltration Pond Sizing

Key assumptions:

- Swale infiltration 10mm/hr
- Rainfall depths – HIRD v3 with climate change allowance to 2090

		Design Pond Area		12200			5000			2600
		Area 3/Pond 3			**Area 2/Pond 2**			**Area 1/Pond 1**		
		Greenfields			*Greenfields*			*Greenfields*		
	Rainfall Used	*(HIRDS)*	**HIRDS +CC**	**Pond vol - zero discharge HIRDs + CC**	*(HIRDS)*	**HIRDS +CC**	**Pond vol - zero discharge HIRDs + CC**	*(HIRDS)*	**HIRDS +CC**	**Pond vol - zero discharge HIRDs + CC**
10 Year										
60	max flow (m3/s)	*0.34*	0.224		*0.226*	0.146		*0.16*	0.104	
	flow vol (m3)		3635	3260		2369	1769		1687	1102
120	max flow (m3/s)	*0.229*	0.224		*0.152*	0.146		*0.108*	0.104	
	flow vol (m3)		4878	4128		3177	1977		2263.4	1093.4
360	max flow (m3/s)	*0.123*	0.224		*0.082*	0.146		*0.058*	0.104	
	flow vol (m3)		7884	5634		5132	1532		3661	151
720	max flow (m3/s)	*0.083*	0.224		*0.055*	0.146		*0.039*	0.104	
	flow vol (m3)		10574	6074		6881	-319		4909	-2111
1440	max flow (m3/s)	*0.055*	0.224		*0.037*	0.146		*0.026*	0.104	
	flow vol (m3)		14037	5037		9136	-5264		6503	-7537
2880	max flow (m3/s)	*0.033*	0.146		*0.022*	0.095		*0.016*	0.067	
	flow vol (m3)		15143	-2857		9897	-18903		6978	-21102
4320	max flow (m3/s)	*0.024*	0.103		*0.016*	0.067		*0.011*	0.047	
	flow vol (m3)		9914	-17086		6531	-36669		4539	-37581

		Design Pond Area		12200			5000			2600
		Area 3/Pond 3			Area 2/Pond 2			Area 1/Pond 1		
		Greenfields			*Greenfields*			*Greenfields*		
	Rainfall Used	*(HIRDS)*	HIRDS +CC	Pond vol - zero discharge HIRDs + CC	*(HIRDS)*	HIRDS +CC	Pond vol - zero discharge HIRDs + CC	*(HIRDS)*	HIRDS +CC	Pond vol - zero discharge HIRDs + CC
20 Year										
60	max flow (m3/s)	*0.418*	**0.327**		*0.278*	**0.254**		*0.197*	**0.165**	
	flow vol (m3)		4784	4409		3137	2537		2204	1619
120	max flow (m3/s)	*0.279*	**0.348**		*0.185*	**0.248**		*0.131*	**0.158**	
	flow vol (m3)		6386	5636		4177	2977		2946	1776
360	max flow (m3/s)	*0.146*	0.224		*0.097*	0.146		*0.069*	0.104	
	flow vol (m3)		9558	7308		6220	2620		4439	929
720	max flow (m3/s)	*0.098*	0.224		*0.065*	0.146		*0.046*	0.104	
	flow vol (m3)		12743	8243		8921	1721		5918	-1102
1440	max flow (m3/s)	*0.065*	0.224		*0.043*	0.146		*0.031*	0.104	
	flow vol (m3)		16900	7900		10995	-3405		7842	-6198
2880	max flow (m3/s)	*0.038*	0.176		*0.025*	0.114		*0.018*	0.082	
	flow vol (m3)		18442	442		12041	-16759		8529	-19551
4320	max flow (m3/s)	*0.028*	0.125		*0.019*	0.082		*0.013*	0.058	
	flow vol (m3)		12221	-14779		8032	-35168		5627	-36493

		Design Pond Area		12200			5000			2600
		Area 3/Pond 3			Area 2/Pond 2			Area 1/Pond 1		
		Greenfields			*Greenfields*			*Greenfields*		
Rainfall Used		*(HIRDS)*	HIRDS +CC	Pond vol - zero discharge HIRDs + CC	*(HIRDS)*	HIRDS +CC	Pond vol - zero discharge HIRDs + CC	*(HIRDS)*	HIRDS +CC	Pond vol - zero discharge HIRDs + CC
50 Year										
60	max flow (m3/s)	*0.547*	**0.611**		*0.363*	**0.485**		*0.257*	**0.28**	
	flow vol (m3)		6882	6507		4539	3939		3144	2559
120	max flow (m3/s)	*0.359*	**0.662**		*0.239*	**0.462**		*0.169*	**0.313**	
	flow vol (m3)		9032	8282		5950	4750		3127	1957
360	max flow (m3/s)	*0.185*	**0.819**		*0.123*	**0.591**		*0.087*	**0.377**	
	flow vol (m3)		12958	10708		8491	4891		5962	2452
720	max flow (m3/s)	*0.122*	**0.471**		*0.081*	**0.327**		*0.057*	**0.219**	
	flow vol (m3)		16148	11648		10511	3311		7493	473
1440	max flow (m3/s)	*0.08*	0.224		*0.053*	0.146		*0.038*	0.104	
	flow vol (m3)		21140	12140		13751	-649		9817	-4223
2880	max flow (m3/s)	*0.047*	0.222		*0.031*	0.144		*0.022*	0.103	
	flow vol (m3)		23457	5457		15302	-13498		10884	-17196
4320	max flow (m3/s)	*0.034*	0.159		*0.023*	0.103		*0.016*	0.074	
	flow vol (m3)		15688	-11312		10289	-32911		7263	-34857

Appendix C Site Coverage and Impervious Surfaces Investigations

Building Coverage
A survey has been undertaken of the extent of building coverage on sites within the General Industrial (I2) zone. This review was of developed sites of 5000m^2 in area or less. The review was undertaken on the 2009 Aerial photographs – the most recent urban series that the Council has. A comparison was made with the 2006 aerial photographs for those sites fronting Omahu Road.

The primary conclusion from this was that a 35% building coverage is an appropriate assumption for the proposed zone. The 50% assumption that was utilised at the commencement of our assessment was considered substantially too high.

The data indicates that:

1. The average building coverage for sites within the I2 zone is 35%. The average building coverage for sites within the Omahu zone is likewise 35%. Refer to ***Table 1*** below.
2. The average building coverage for sites of 1000m2 to 5000m2 within the I2 zone is 35%. The average building coverage for the same site size range within the Omahu zone is 34%. Refer to ***Table 2*** below.
   NB: Sites of less than 1000m^2 are no longer permitted as of right within the I2 zone. Nor are they proposed to be permitted within the new zone.
3. The average building coverage on those I2 zone sites fronting Omahu Road (developed / substantially redeveloped since 2006) is 35%. Refer to ***Table 3*** below.
   This was considered as there are indications that there has been a tendency towards a more efficient use of land over time. Advice received by the Council is that this trend is likely to continue over time.
4. Building coverage tends to be greater on smaller sites. Refer to ***Tables 4 to 6*** below:

Table 1
Building Coverage - Industrial 2 zone - Sites <=5000m^2

	Land Area (m^2)	Building Area (m^2)	No. of Sites	Average Coverage
Omahu	253982	88887	131	35%
Whakatu	10798	2967	5	27%
Tomoana	8267	4150	3	50%
Hastings	Nil	Nil	0	Nil
Total	273047	96004	139	35%

Table 2
Building coverage - Industrial 2 zone - Sites =>1000m2<=5000m2

	Land Area (m^2)	Building Area (m^2)	No. of Sites	Average Coverage
Omahu	235462	80743	106	34%
Whakatu	8479	2463	2	29%
Tomoana	8267	4150	3	50%
Hastings	Nil	Nil	0	Nil
Total	252208	87356	111	35%

Table 3
Building coverage - Industrial 2 zone sites fronting Omahu Road- post 2006

	Land Area (m^2)	Building Area (m^2)	No. of Sites	Average Coverage
Omahu	11832	4167	7	35%
In proposed Zone*	26915	5352	4	20%
Total	38747	9519	11	25%

Table 4				
Building coverage - Sites <=1000m²				
	Land Area (m^2)	Building Area (m^2)	No. of Sites	Average Coverage
Omahu	18520	8144	25	44%
Whakatu	2319	504	3	22%
Tomoana	Nil	Nil	0	Nil
Hastings	Nil	Nil	0	Nil
Total	20839	8648	28	41%

Table 5				
Building coverage - Sites =>1000m²<=2000m²				
	Land Area (m^2)	Building Area (m^2)	No. of Sites	Average Coverage
Omahu	79497	29872	55	38%
Whakatu	Nil	Nil	0	Nil
Tomoana	Nil	Nil	0	Nil
Hastings	Nil	Nil	0	Nil
Total	79497	29872	55	38%

Table 6				
Building coverage - Sites =>2000m2<=5000m2				
	Land Area (m^2)	Building Area (m^2)	No. of Sites	Average Coverage
Omahu	155965	50871	51	33%
Whakatu	8479	2463	2	29%
Tomoana	8267	4150	3	50%
Hastings	Nil	Nil	0	Nil
Total	172711	57484	56	33%

Notes:
Table 2 reflects the site sizes to be permitted within the proposed zone
Table 3 was considered to see if there is a recent trend on Omahu Road
Table 4 includes only sites <1000m2 which are not anticipated to be permitted
Table 5 includes only smaller sites anticipated to be infrequently developed
Table 6 includes the sites size anticipated to be most frequently developed

Impervious Surfaces
The Industrial 2 zone standards do not include a pervious surface requirement for the Industrial 2 zone – except by way of a 2m wide front yard - ½ of which must be landscaped. Were these rules applied to the proposed new zone, this equates to somewhere in the vicinity of 0.8% (2888m2) of the zone being landscaped/pervious. Historically these landscape areas were often only informally created and tended to become compacted or even sealed over time. The Council's monitoring of these standards has increased substantially over time. As a consequence these landscape areas are now more consistently formed (curbed & channelled) and maintained.

A survey of the 2009 Aerial Photographs indicates that the average extent of impervious surfaces on I2 zoned sites fronting Omahu Road with an area of less than 1ha is 94%.

A survey of the 2009 Aerial Photographs indicates that the average extent of impervious surfaces on the six sites already developed for intensive industrial use within the proposed zone is 93%.

Author: Tracey Gray, Strategic Planner, Hastings District Council
Date: 12 April 2012

HASTINGS DISTRICT COUNCIL

Map Produced using ArcMap

Surveyed sites of less than 1ha

Scale 1:13,637

Date: Thursday, 12 April 2012

Projection: NZTM
Datum: D_NZGD_2000

210 105 0 210 420 630 840 Meters

Original Size: A4

DATA SOURCE: Cadastral information derived from the Land Information New Zealand's Core Record System (CRS). CROWN COPYRIGHT RESERVED

DISCLAIMER: The Hastings District Council cannot guarantee that the data shown on this map is 100% accurate.

traceyk OmahuResourceConsent