Perry Group - Te Awa Lakes
Perry Group Limited
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# Horotiu West Development

## Three Waters Infrastructure Assessment

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# Horotiu West Development

Three Waters Infrastructure Assessment

Client: Perry Group Limited

Co No.: 191504

Prepared by

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## Glossary

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|---|---|
| HCC | Hamilton City Council |
| WDC | Waikato District Council |
| WRC | Waikato Regional Council |
| SMP | Stormwater Management Plan |
| CMP | Catchment Management Plan |
| WWTP | Wastewater Treatment Plant |

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# Executive Summary

The Horotiu West site is located in Horotiu about 7 km south of Ngaruawahia at the northern boundary of Hamilton City. The city boundary at the site location is delineated by the Waikato Expressway. The Horotiu West Development land area was formerly part of a larger 62 hectare block which was separated by the Waikato Expressway construction. The southern land block is now known as Te Awa Lakes and is planned to be developed as mixed use commercial and residential.

Because the two sites cross municipal boundaries, water and wastewater services will be provided by separate councils. A stormwater management plan[1] has been prepared for the Te Awa Lakes Development which includes provision for the Horotiu West Development. Stormwater from the site is planned to be treated and controlled prior to discharge to the Te Awa Lakes via a culvert under the Waikato Expressway.

The site is currently pastoral farmland. Western parts of the site are original flat to gently rolling hills. The eastern edge of the site alongside the expressway features modified topography as a result of historic sand mining. The mined areas are flat bottomed depressions several meters lower than the original ground levels.

Original ground levels at the site range from RL 16 m to RL 26 m. Excavated sand mine areas are as low as RL 13 m in some locations. Flood risk exists below RL 16 m so the site can be developed at higher levels.

The Horotiu West site is zoned Deferred Residential so it is not included in WDCs infrastructure planning for Horotiu. This does not necessarily limit the development of the site in regard to stormwater because the Horotiu West development is included in the stormwater catchment and associated CMP for the adjacent Te Awa Lakes development.

The Te Awa Lakes development is located within Hamilton City. Water and wastewater servicing for the Te Awa Lakes will be via the Hamilton City water and wastewater networks in Te Rapa north. Service capacity in northern Hamilton is sufficiently limited that it is understood that HCC is not able to consider serving Horotiu West as cross boundary network.

**Stormwater**

Stormwater will be dealt with in accordance with the Te Awa Lakes CMP and associated SMP. Discharge from the Horotiu West development will be to the Te Awa Lakes via a new culvert under the Waikato Expressway. The requirements of the Te Awa Lakes SMP can be achieved in the Horotiu West development as follows:

- Individual on-lot requirements can be accommodated within the proposed development, probably in the form of rain tanks or permeable paving (and grassed areas).
- The roading and green space concept can accommodate rain gardens and grassed areas which will be able to provide primary treatment, retention and soakage. Swales are not proposed to be used due to their long footprint and associated limitations/difficulties regarding lot access.
- Final treatment and control will occur in a system of wetlands alongside the Waikato Expressway. The wetlands will be treatment and detention wetlands in accordance with the Te Awa Lakes CMP. Attenuation will not be provided but the wetlands will be capable of conveying and discharging flows from large events (larger than the water quality design storm).

The preliminary assessment undertaken for this report has shown that the wetland storage requirements can be accommodated within the concept plan footprint. Final requirements will be implemented as part of design and construction to align with the CMP.

**Wastewater**

Horotiu West and adjacent residential zoned areas (also undeveloped) are not included in WDC wastewater infrastructure planning. The Horotiu wastewater network and the Ngaruawahia network to

[1] Te Awa Lakes Stormwater Management Plan, CKL, 30 October 2017

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which it discharges are known to have capacity issues. WDC are in the process of planning for the implementation of a new major pump station direct to the Ngaruawahia WWTP.

A number of existing wastewater sub-catchments will be diverted to the proposed major pump station. The proposed pump station provides an opportunity for Horotiu West and adjacent residential areas to also be serviced without placing additional loading on the Horotiu network. Planning of the pump station is not yet so far progressed that additional areas cannot be included.

All wastewater servicing options ultimately involve discharge to the WWTP via the proposed WDC major pump station. Several options to discharge to, or utilise the major pump station have been identified based on discussions with Perry and WDC as follows:

- Option 1 – Privately owned pump within the Horotiu West development.
- Option 2 – Council owned minor pump station within the Horotiu West development.
- Option 3 – Locate the proposed major pump station within the Horotiu West development.

Further assessment is required to determine the impact Horotiu West will have on the scale of the proposed pump station and rising main, and also to asses potential options related to the number and location of pump stations. At the time of writing sufficient information is not available to undertake more detailed assessment so it is a future action.

**Water**

Horotiu West is not included in WDC water infrastructure planning. The proposed development will need to be assessed in the WDC water network model to determine whether any service level issue will arise as a result. Preliminary water demand figures have been provided to WDC at the time of writing and modelling is yet to be carried out.

Horotiu has an existing water network which, subject to modelling, can be extended to service Horotiu West. Principal mains (150 mm or larger) are located on Horotiu Bridge Road and rider mains (63 mm) extend to the site boundary on Great South Road and Kernott Road. Rider mains are not sufficient to service development but the principal mains can be extended through the site to create a loop main as is usual practice.

**Future actions**

The information in this report shows that it is possible service the development in conjunction with the Te Awa Lakes stormwater, and WDC water and wastewater networks (existing and planned respectively). Notwithstanding, a number of future actions are required to confirm capacity and scale of impacts on the water and wastewater networks as follows:

a. Test the addition of the Horotiu West development in the WDC water network model to determine whether there are any adverse effects on service levels in Horotiu.
b. Consider the impacts of increased wastewater flow from Horotiu West on the following:
    - The scale of the proposed major pump station.
    - The size of the proposed rising main direct to the WWTP.
    - The operation of the Ngaruawahia WWTP treatment process and associated discharge.
c. Consider the various options available for location and ownership of the proposed major pump station and minor pump stations required to service parts of Horotiu and the Horotiu West development.

It is noted that the wastewater servicing options may be affected by the relative timing of Horotiu West and WDCs proposed major pump station. Not all option will remain viable if one aspect is progressed prior to the options being assessed and adopted.

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# 1.0 Introduction

AECOM New Zealand Limited (AECOM) has been engaged by Perry Group Limited to undertake a Three Waters Assessment for a District Plan change process involving a proposed residential development. The proposed development is located at Horotiu in the Waikato District and is part of the wider Te Awa Lakes Development, the balance of which is located within Hamilton City.

The proposed development is located west of the Waikato River and immediately north of the Waikato Expressway which also forms the northern boundary of Hamilton City. Access to the site is from Great South Road and Kernott Road, Horotiu. The total area of the proposed development is about 17 hectares.

The proposed development will comprise up to about 285 residential lots that will be serviced from Horotiu via Great south Road and a new internal road network.

Figure 1 shows a preliminary concept plan of the proposed development.


**Figure 1 Concept plan (option 2)**

Note that the initial concept plan is subject to change as the development detail is advanced.

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# 2.0 Scope of this report

The purpose of this report is to identify three waters management options and constraints and provide a recommendation of the methods to service the property for residential development to support resource consent and approvals.

The scope of this assessment includes the following:

- Review existing stormwater drainage management proposals associated with Te Awa Lakes to the south of the Waikato Expressway.
- Development of further detail for the purposes of confirming the likely scale and viability of a stormwater solution, including its ability to align with the already approved Te Awa Lakes and associated existing Resource Consents.
- Determine the location and size of existing Waikato District Council (WDC) water and wastewater services in the vicinity of the site.
- Liaise with WDC to determine how the development sits in relation to Horotiu infrastructure planning and future capacity including the need for assessment of upgrades to public infrastructure to service the site.
- Develop conceptual water and wastewater servicing solutions for the site to a level of detail suitable to support planning approvals and overall subdivision planning.

# 3.0 Site description

The site is located in Horotiu about 7 km south of Ngaruawahia at the northern boundary of Hamilton City. The city boundary at the site location is delineated by the Waikato Expressway. The Horotiu West Development land area was formerly part of a larger 62 hectare block which was separated by the Waikato Expressway construction. The southern land block is now known as Te Awa Lakes and is planned to be developed as mixed use commercial and residential.

Because the two sites cross municipal boundaries, water and wastewater services will be provided by separate councils. A stormwater management plan[2] has been prepared for the Te Awa Lakes Development which includes provision for the Horotiu West Development. Stormwater from the site is planned to be treated and controlled prior to discharge to the Te Awa Lakes via a culvert under the Waikato Expressway.

The site is currently pastoral farmland. Western parts of the site are original flat to gently rolling hills. The eastern edge of the site alongside the expressway features modified topography as a result of historic sand mining. The mined areas are flat bottomed depressions several meters lower than the original ground levels.

Original ground levels at the site range from RL 16 m to RL 26 m. Excavated sand mine areas are as low as RL 13 m in some locations.

# 4.0 Stormwater

## 4.1 Te Awa Lakes SMP and Horotiu West

### 4.1.1 Introduction

The Te Awa Lakes (east) Development centres on a recreational lake system into which development will discharge following treatment and control. The lakes are consented to discharge to the Waikato River via an existing gully stream downstream of Hutchinson Road.

An SMP has been developed for Te Awa Lakes which includes the proposed western development. The inclusion of the western development within the overall stormwater treatment and control scheme

[2] Te Awa Lakes Stormwater Management Plan, CKL, 30 October 2017

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is also acknowledged in resource consent documents. The Horotiu West development stormwater solution is therefore based on achieving and aligning with the requirements and principals of the Te Awa Lakes SMP and associated resource consents.

The Te Awa Lakes SMP references HCCs District Plan and Infrastructure Technical Specifications stormwater requirements and solutions. It is acknowledged that the Horotiu West is not located within Hamilton City so the Hamilton District Plan requirements cannot apply. WDC have however adopted HCCs design standards so they are applicable.

The Horotiu West site naturally discharges to the Te Awa Lakes site within Hamilton City and will continue to do so when developed. It is therefore appropriate for the Horotiu West site to continue to align with the Te Awa Lakes SMP and the associated HCC guidance and solutions.

### 4.1.2 Context

This report should be read in conjunction with the Te Awa Lakes SMP. Key requirements from the SMP are presented in the following section referenced to the SMP where key to the development of the Horotiu West solution concept in this report. A comprehensive standalone assessment has not been undertaken.

### 4.1.3 Stormwater treatment and control requirements

Table 1 lists the key stormwater requirements outlined in the Te Awa Lakes Stormwater Management Plan. The stormwater requirements are the minimum standard to be achieved to be in compliance with the SMP and associated resource consents.

**Table 1 Horotiu West stormwater requirements**

| Item | Requirement | SMP reference |
|---|---|---|
| Primary (piped) collection network | The west development is residential so will provide a piped system for up to the 2 year event. Conveyance of flows through the lake system is allowed for. | 4.34, 4.35 |
| Overland flow | Overland flow form the western development will be via the expressway culvert and the lake system. | 4.36, Figure 6 |
| Water quality volume | Although the eastern lakes will hold a large water volume, the west development is to hold its own water quality volume for treatment purposes. | 6.28 |
| Extended detention | Although extended detention will be provided in the lakes for the eastern development, the west development is to carry out its own extended detention. | 6.28 (also refer to 4.24 and 4.25) |
| Peak flow attenuation | Flow attenuation is not required for this development due to the location of the subject site within the catchment and the site's proximity to the Waikato River.<br><br>It is deemed that extended detention is sufficient to protect the natural gully, and attenuation upstream of the lakes will have negligible benefit. | 4.26 |
| Flood control | 100 year ARI event attenuation is not required as there is no risk of downstream flooding (the lakes will have the capacity to convey flood flows) | 4.27 |
| | The culvert under the expressway is required to convey the 100 year ARI evet without heading up more than one diameter or creating a flood risk to buildings on the west development site. | 6.29 |
| Lined wetlands | Line wetlands to retain water for the lake and for the purpose of permanent water quality volume retention and plant health. | 7.23 |
| Treatment for areas with | For runoff within the road reserve rain gardens sized at 6% of the WQV will provide adequate treatment | 5.6 |

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| Item | Requirement | SMP reference |
|---|---|---|
| downstream wetlands Note 1 | Runoff from within residential areas can be managed in a variety of ways such as tanks, rain gardens, permeable paving and grassed areas. | 5.7, Table 11 |

Note 1 – the west development has a wetland system so requirements for catchments with no downstream wetland have not been listed.

## 4.2 Flooding

The Horotiu CMP states that WRCs Waikato River model indicates that the Waikato River 1 % AEP (with no climate change) flood levels range from about RL 15 m to RL 14.8 m within the Horotiu Structure Plan Area (not including climate change). Section 3.17 of the Te Awa Lakes SMP states that Waikato District Council planning maps indicate a flood level of RL 15.87 m based on interpolation of 1958 flood levels between Hamilton and Ngaruawahia.

Most of the site is well above the Waikato River flood level. A low point in the natural topography adjacent to the Waikato River Bridge is at or above RL 15.88 m (based on HCC 1 m LiDAR). Areas in the north eastern corner of the site that have been sand mined are below the estimated flood levels and could not be developed without filling (currently at RL 13 m). Low areas could be used for stormwater management without filling.

It is anticipated that the adoption of standard freeboard to floor levels above a minimum ground level of RL 16 m (i.e. RL 16 m plus freeboard) should be sufficient to avoid flooding of buildings. The Te Awa Lakes SMP recommends RL 15.87 m plus freeboard.

## 4.3 Primary collection network

At source treatment elements are require so that the development complies with the Te Awa Lakes CMP. Suitable solutions include tanks, rain gardens, permeable paving and grassed areas in various combinations.

The proposed development has not been planned to a level of detail where the exact location and design of individual at source elements can be provided.

- Individual on-lot requirements can be accommodated within the proposed development, probably in the form of rain tanks or permeable paving (and grassed areas). Final requirements will be implemented as part of design and construction to align with the CMP.
- The roading and green space concept can accommodate rain gardens and grassed areas which will be able to provide primary treatment, retention and soakage. Swales are not proposed to be used due to their long footprint and associated limitations/difficulties regarding lot access.

A piped collection system will convey stormwater from the source devices to a centralised wetland system which will carry out final treatment and control. The primary piped system is required to convey a 2 year ARI storm event.

## 4.4 Overland flow

Overland flows which exceed the capacity of the primary system will be directed along roads to the centralised wetland. The centralised wetland will have sufficient volume and discharge capacity to convey flood flows under the expressway to the Te Awa Lakes.

Refer to Section 4.5 and Section 4.6 for more detail related to the wetland outlet structure and the expressway culvert.

## 4.5 Final treatment and control (wetland)

Final treatment and control will occur in a system of wetlands alongside the Waikato Expressway. The wetlands will be treatment and detention wetlands in accordance with the Te Awa Lakes CMP. Attenuation will not be provided but the wetlands will be capable of conveying and discharging flows from large events (larger than the water quality design storm).

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A concept design of the wetland has been undertaken to demonstrate the required volumes can be accommodated within the proposed footprint. The concept design is based on the total wetland system areas and volumes. The wetlands will need to be split, and appropriate sub catchments defined at the detailed design stage.

Refer to Appendix B for concept design calculations and tables.

#### 4.5.1 Catchment parameters

Table 2 lists the catchment parameters used for the concept design. Only post development parameters are provided because pre development parameters are only relevant where attenuation is required. Pre development parameters are contained in the tables in Appendix B.

**Table 2 Catchment parameters**

| Item | Parameter |
|---|---|
| Total development area | 17.1 hectares |
| Post development impervious area | 10.0 hectares |
| Post development pervious area | 7.1 hectares |
| Impervious curve number (CN) | 98 |
| Pervious curve number (CN) | 74 |
| Post development weighted curve number | 88 |
| **Design rainfall (all including climate change adjustments)** | |
| Extended Detention | 24 mm |
| Water Quality Storm | 24.1 mm |
| 2 year 24 hour | 72.2 mm |
| 10 year 24 hour | 110.7 mm |
| 100 year 24 hour | 184.6 mm |

All of the parameters used in the assessment generally align with those used in the Te Awa Lakes CMP with the exception of the extended detention depth. The CMP used an extended detention depth of 28.9 mm whereas this assessment uses 24 mm in accordance with typical Hamilton City requirements.

Impervious areas are based on measurement of the proposed building footprints and roads plus an allowance for additional 100 $m^2$ of hardstand area per lot.

The assessment results are shown in Table 3. Volumes are totals for the entire development and may be split during detailed design. Retention volume has not been accounted for in the wetland concept but may be accommodated in the primary treatment and collection network where practical.

**Table 3 Stormwater assessment results – design volumes and flows**

| Item | Design output |
|---|---|
| Retention volume (5 mm) | 855 $m^3$ |
| Water quality volume | 2,221 $m^3$ |
| Permanent wetland storage volume | 1,480 $m^3$ |
| Extended detention volume | 2,213 $m^3$ |
| 2 year post development peak flow rate | 1,358 L/s |
| 10 year post development peak flow rate | 2,347 L/s |
| 100 year post development peak flow rate | 4,323 L/s |

Peak flow rates are from the development and do not include discharges from the Waikato Expressway. The expressway drainage regime is yet to be finalised. If the expressway discharges to the wetland, the outlet spillway will need to be designed for a larger flow rate.

#### 4.5.2 Outlet structure

The wetland outlet structure requires a primary extended detention outlet and a secondary overflow to achieve the requirements of the CMP. The outlet structure is proposed to comprise a circular manhole riser connected directly to the upstream end of the expressway culvert (see Section 0)

A circular weir assessment has been carried out to determine the size of the riser required to pass the 100 year storm event at a reasonable depth. A standard 2300 mm riser pipe is capable of passing the peak design flow with a depth of about 200 mm above the weir.

The outlet structure is proposed to be a 2,300 mm diameter riser pipe as follows:

- Primary extended detention outlet orifice – diameter to be confirmed at detailed design.
- Secondary service outlet slot about 100 mm deep – width to be confirmed at detailed design.
- Circular weir overflow via 2,300 mm dimeter manhole riser.

A service outlet is not strictly required but has been included as good practice. A small service outlet will reduce the use of the overflow and will aid in transitioning flow from the primary orifice to large volume spilling. The service outlet has not been accounted for in sizing the circular weir (conservative).

The 2300 mm diameter manhole is too large for a standard scruffy dome grate. Blockages are not expected due to the scale of the structure, the culvert diameter, and the nature of the development runoff (e.g. no large debris is expected).

#### 4.5.3 Wetland arrangement

Conceptual wetland surface areas have been measured from the overall concept plan and taken as the maximum allowable water level area. A depth-area and depth- volume profile has been developed so that the concept design volumes could be assessed against the available volumes.

The following wetland arrangement is proposed based on the areas, volumes and the parameters in Section 4.5.1 and 4.5.2.

**Table 4 Proposed wetland arrangement**

<table>
<tr><th>Wetland No. / Depth above or below max water level</th><th>Total area ($m^2$)</th><th>Total volume ($m^3$)</th><th>Inverse volume ($m^3$)</th><th colspan="2">Function</th></tr>
<tr><td>+0.3 (ground level)</td><td colspan="4" rowspan="3">Freeboard see note</td><td rowspan="9">Unlined</td></tr>
<tr><td>+0.2</td></tr>
<tr><td>+0.1</td></tr>
<tr><td>0.0 (max. water level)</td><td>9720</td><td>11688</td><td>0</td><td rowspan="2">Circular weir</td></tr>
<tr><td>-0.1</td><td>9295</td><td>10737</td><td>951</td></tr>
<tr><td>-0.2</td><td>8869</td><td>9829</td><td>1859</td><td>Service outlet slot</td></tr>
<tr><td>-0.3</td><td>8444</td><td>8963</td><td>2725</td><td rowspan="3">EDV (min 2213$m^3$, 2469 $m^3$ available)</td></tr>
<tr><td>-0.4</td><td>8018</td><td>8140</td><td>3548</td></tr>
<tr><td>-0.5</td><td>7593</td><td>7359</td><td>4328</td></tr>
<tr><td>-0.6</td><td>7167</td><td>6621</td><td>5066</td><td rowspan="4">Available for permanent water volume makeup (banded bathymetry)</td><td rowspan="4">Lined</td></tr>
<tr><td>-0.7</td><td>6742</td><td>5926</td><td>5762</td></tr>
<tr><td>-0.8</td><td>6316</td><td>5273</td><td>6414</td></tr>
<tr><td>-0.9</td><td>5891</td><td>4663</td><td>7025</td></tr>
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| Wetland No. / Depth above or below max water level | Total area ($m^2$) | Total volume ($m^3$) | Inverse volume ($m^3$) | Function | |
|---|---|---|---|---|---|
| -1.0 | 5465 | 4095 | 7593 | | |
| -1.1 | 5191 | 3562 | 8125 | | |
| -1.2 | 4917 | 3057 | 8631 | | |
| -1.3 | 4643 | 2579 | 9109 | | |
| -1.4 | 4369 | 2128 | 9559 | | |
| -1.5 | 4095 | 1705 | 9983 | | |
| -1.6 | 3821 | 1309 | 10378 | | |
| -1.7 | 3547 | 941 | 10747 | | |
| -1.8 | 3273 | 600 | 11088 | | |
| -1.9 | 2999 | 286 | 11401 | | |
| -2.0 | 2725 | 0 | 11688 | | |

Note: Wetland freeboard allows for a buffer from the top water level to the top of the wetland formation. It is expected that building freeboard will be a nominal amount above the top of the wetland formation, not the maximum water level.

The wetland may be completely lined up to the maximum water level to maximise the volume of water discharged to the Te Awa Lakes (as implied in the CMP). The minimum requirement is for the wetland to be lined up to the permanent storage volume level so that water is remained for wetland and plant health.

The option to leave the upper portion of the wetland unlined is to allow for the maintenance of groundwater levels in conjunction with primary treatment devices and runoff volume reduction.

Table 5 lists the concept parameters for banded bathymetry within the permanent storage zone. The recommended bathymetry can be achieved within the concept plan footprint. Refer to the calculation tables in Appendix B for more detail reading the proportions adopted.

**Table 5 Wetland permanent storage zone make-up**

| Permanent storage zone volume ($m^3$) | | 1480 |
|---|---|---|
| 0 m to 0.2 m deep | Volume ($m^3$) | 563 |
| | Area ($m^2$) | 4327 |
| 0.2 m to 0.35 m deep | Volume ($m^3$) | 563 |
| | Area ($m^2$) | 2009 |
| 0.35 m to 1.2 m deep | Volume ($m^3$) | 178 |
| | Area ($m^2$) | 222 |
| 0.35 m to 1.5 m deep | Volume ($m^3$) | 178 |
| | Area ($m^2$) | 161 |
| **Total area of permanent storage** | | **6720** |
| Forebay (15% WQV, 1m deep) | Volume ($m^3$) | 333 |
| | Area ($m^2$) | 333 |
| **Total area required incl. forebay ($m^2$)** | | **7,053** |
| **Permanent storage area available ($m^2$)** | | **7,167** |

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The concept assessment and output parameters in Table 5 show that the concept plan wetland area is sufficient to achieve the requirements.

### 4.6 Waikato Expressway culvert

An existing 1050 mm dimeter culver is located under the Waikato Expressway. The culvert is lower than the proposed Te Awa Lakes level so would operate submerged if it was to remain.

The CMP undertook an assessment of the culvert capacity and states that the culvert could be capable of passing a 100 year event with an inlet headwater if about RL 19.7 m. Such a headwater level is well above the Waikato River flood level and would have a significant effect on the developable level within the site. The culvert is recommended to be upgraded for the following reasons:

- To raise it above the proposed lake level so it does not operate submerged.
- To raise it above submerged levels so that it can be inspected and maintained as a culvert under a state highway.
- To increase the culvert diameter and capacity sufficient to limit headwater levels to no more than RL 16 m which is the design flood level associated with the Waikato River.

A preliminary culvert sizing has been undertaken assuming the Te Awa Lakes will be at a design level of RL 13.5 m and the culvert maximum allowable headwater is RL 16 m. The sizing resulted in a minimum culvert size of 1500 mm to be able to convey the design 100 year ARI flow rate of 4.3 $m^3/s$ (refer Table 3).

The preliminary culvert size is subject to final design lake levels and final design of the Horotiu West wetland layout, levels and outlet structure. The final design culvert slope will affect the flow regime through the culvert (e.g. supercritical) and the resulting outlet energy dissipation and erosion control requirements; these aspects have not been considered in detail at this stage.

## 5.0 Wastewater

### 5.1 Conceptual design parameters

Table 6 presents the results of an assessment of potential wastewater flow from the developed site based on the HCC infrastructure Technical Specifications method of calculation. Refer to Appendix B for a calculation sheet.

**Table 6 Estimated wastewater requirements**

| Item | Parameter |
|---|---|
| Site are used in calculation | 17 hectares |
| Average daily flow | 2.4 L/s |
| Peak daily (dry weather) flow | 6.7 L/s |
| Peak wet weather flow | 9.9 L/s |
| Design pumped flow rate (minimum) | 10.0 L/s |
| Pump station emergency storage volume (9 hour) | 77.5 $m^3$ |

### 5.2 Horotiu wastewater network

The Horotiu wastewater network discharges to Ngaruawahia via several pump stations. The nearest pump stations to the proposed development are located on Washer Road and Gateway Drive some 700 m away. Neither pump station can be reached by a gravity discharge from the development.

The proposed development is not located in the future residential zone of Horotiu. Properties adjacent to the site are zoned residential and will require a new collection network and pump station in order to

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be serviced. It is possible that the residential development along Horotiu Bridge Road and Kernott Road could be serviced by a common network and pump station.

Discussions with WDC water asset management staff indicate that the following networks changes are planned:

- A new major wastewater pump station will be constructed in the vicinity of the Horotiu Bridge in the period 2018-2021.
- A new rising main will be constructed from the new pump station direct to the Ngaruawahia Wastewater Treatment Plant (the WWTP).
- The Washer Road and Gateway Drive pump station catchments will be diverted to the new pump station (by gravity is possible).
- Some existing residential areas of Horotiu may also be serviced by the new pump station to further reduce loading on the existing Horotiu network and the Ngaruawahia network to which it currently discharges.

WDC indicated that it would be logical for the Horotiu West development to connect to the new pump station which will be located nearby. Connection of the proposed development to the new pump station eliminates capacity issues that would have arisen had connection to the existing network been required. It was however noted that initial planning for the major pump station does not include the Horotiu West site, or adjacent undeveloped residential zoned areas.

A number of potential servicing options were discussed with WDC in relation to the proposed pump stations. The options and implications are discussed in Section 5.4.

## 5.3 Development wastewater network – internal servicing

Existing site contours indicate that all areas should be able to be serviced by gravity to a single location near the north eastern boundary of the site alongside the Waikato River. Some areas may require minor filling or contouring to achieve minimum pipe cover depending on the final design of the subdivision.

The proposed wastewater network for the site is to provide a gravity fed system that conveys wastewater from the site (and adjacent land if necessary) to the north east corner to a new pump station. A 150 mm diameter collection network should be sufficient for the development based on the estimated peak flows.

A 100 to 125mm diameter rising main (based on 0.9-1.2 m/s) could deliver the design pump flow rate of 10 L/s if the development is to be serviced by a dedicate pump station (refer to Section 5.4 for alternative options).

A conceptual plan identifying the key wastewater infrastructure is shown in Appendix A.

## 5.4 External servicing options

All external servicing options ultimately involve discharge to the WWTP via the proposed WDC major pump station. Several options to discharge to, or utilise the major pump station have been identified based on discussions with Perry and WDC as follows:

- Privately owned pump within the Horotiu West development.
- Council owned minor pump station within the Horotiu West development.
- Locate the proposed major pump station within the Horotiu West development.

The options are outlined in the following sections.

### 5.4.1 Option 1 – Privately owned pump station

The Horotiu West development is a private development that will remain with Perry as the owners. It is understood that the development assets will not be vested to WDC. On this basis it is possible that the pump station and rising main would remain in the ownership of Perry's.

**D R A F T**

The adjacent undeveloped residential zoned areas would not be serviced by the Horotiu West pump station if it remains private. WDC would benefit from not having to own and operate the development pump station but would most likely have to implement a public pump station for the adjacent residential zoned area.

Two pump stations would be located in the block of land between Horotiu Bridge Road and the Waikato Expressway; one owned by Perry and one owned by WDC.

#### 5.4.2 Option 2 – Council owned minor pump station

The Horotiu West development and the adjacent undeveloped residential zoned areas could be serviced by a single public pump station. WDC would still have to own and operate one additional pump station but is would be larger than the public pump station in Option 1.

One pump station would be located in the block of land between Horotiu Bridge Road and the Waikato Expressway; owned by WDC.

#### 5.4.3 Major pump station

It may be possible to relocate the proposed major pump station to the low point in the Horotiu West site and gravitate all of the associated catchment to that location.

The Horotiu West development and the adjacent undeveloped residential zoned areas and the planned industrial areas could be serviced by the major pump station. WDC would not have to own and operate any additional pump stations apart from the planned major pump station.

This option will require Perry to accommodate the pump station within the planned development, inclusive of access for operation and maintenance. WDC would benefit because alternative land next to the Horotiu Bridge would not be required. The major rising main would be slightly longer but not much in the context of the overall length.

#### 5.4.4 Discussion and further assessment requirements

At the time of writing neither Perry nor WDC are able to commit to locating the major pump station within Horotiu West land. However, the option is generally acknowledged as having benefit in terms of the optimisation of the number of pump stations in the area, and a potentially easy pathway to securing pump station land.

All of the options will have the same, or similar, impact on the capacity of the proposed major pump station. WDC have indicated they will also need to check that the additional development can be accommodated at the WWTP. It is expected that the scale of the development is small in comparison to the WWTP service area so would be considered to be minor.

Sufficient information was not available at the time of writing to assess the relative scale of the WDC major pump station with and without the development. Assessment will be required because it will inform the following aspects:

- The impact the development will have on pump size and rising main size.
- The feasibility of gravitating existing areas of Horotiu to the Horotiu West site.
- The scale of land required on the Horotiu West site for the different size pump stations.
- Potential benefits of increased flow into the major pump station that could reduce the risk of septicity in the long rising main (this may occur regardless).

WDC planning for the major pump station is currently limited. WDC will be securing land for the pump station near the Horotiu Bridge in 2018. Detailed hydraulic design and sizing of the pump station has not been undertaken yet.

In summary it is believed that the proposed major pump station and associated diversions provides a viable solution for the Horotiu West development to be serviced. It is acknowledged that more detailed assessment is warranted so that both Perry and WDC can understand the implications of the options in terms of pump stations sizes and land requirements. The nature and responsibility of parts of the assessments will depend on WDC progress at the time and may be carried out by Perry in lieu of information from WDC.

**D R A F T**

THIS SECTION MAY CHANGE DEPENDING ON WHAT COMES FROM WDC IN THE COMING DAYS

# 6.0 Water

Existing 63 mm rider mains are located along Great South Road and Kernott Road to within about 10 m and 80 m of the site boundary respectively. Rider mains are not sufficient to service the development.

The nearest principal mains (150 mm or larger) are located on Horotiu Bridge Road. Pending detailed assessment is it anticipated that principal mains at least 150 mm in diameter will need to be extended to the site along Great South Road and Kernott Road. Two principal main extensions are recommended so that a ring main is created through the development.

Discussions with WDC indicate that the proposed ring main layout is acceptable. WDC have been provided with concept water network design flows to test in the district water model. At the time of this report, results were not yet available for the model assessment so capacity issues, if any, are not yet known.

A conceptual plan identifying the principal water network ring main is shown in Appendix A.

# 7.0 Summary

The Horotiu West site is zoned Deferred Residential so it is not included in WDCs infrastructure planning for Horotiu. This does not necessarily limit the development of the site in regard to stormwater because the Horotiu West development is included in the stormwater catchment and associated CMP for the adjacent Te Awa Lakes development.

The Te Awa Lakes development is located within Hamilton City. Water and wastewater servicing for the Te Awa Lakes will be via the Hamilton City water and wastewater networks in Te Rapa north. Service capacity in northern Hamilton is sufficiently limited that it is understood that HCC is not able to consider serving Horotiu West as cross boundary network.

## 7.1 Stormwater

Stormwater will be dealt with in accordance with the Te Awa Lakes CMP and associated SMP. Discharge from the Horotiu West development will be to the Te Awa Lakes via a new culvert under the Waikato Expressway. The requirements of the Te Awa Lakes SMP can be achieved in the Horotiu West development as follows:

- Individual on-lot requirements can be accommodated within the proposed development, probably in the form of rain tanks or permeable paving (and grassed areas).
- The roading and green space concept can accommodate rain gardens and grassed areas which will be able to provide primary treatment, retention and soakage. Swales are not proposed to be used due to their long footprint and associated limitations/difficulties regarding lot access.
- Final treatment and control will occur in a system of wetlands alongside the Waikato Expressway. The wetlands will be treatment and detention wetlands in accordance with the Te Awa Lakes CMP. Attenuation will not be provided but the wetlands will be capable of conveying and discharging flows from large events (larger than the water quality design storm).

The preliminary assessment undertaken for this report has shown that the wetland storage requirements can be accommodated within the concept plan footprint. Final requirements will be implemented as part of design and construction to align with the CMP.

## 7.2 Wastewater

Horotiu West and adjacent residential zoned areas (also undeveloped) are not included in WDC wastewater infrastructure planning. The Horotiu wastewater network and the Ngaruawahia network to

**D R A F T**

which it discharges are known to have capacity issues. WDC are in the process of planning for the implementation of a new major pump station direct to the Ngaruawahia WWTP.

A number of existing wastewater sub-catchments will be diverted to the proposed major pump station. The proposed pump station provides an opportunity for Horotiu West and adjacent residential areas to also be serviced without placing additional loading on the Horotiu network. Planning of the pump station is not yet so far progressed that additional areas cannot be included.

Further assessment is required to determine the impact Horotiu West will have on the scale of the proposed pump station and rising main, and also to asses potential options related to the number and location of pump stations. At the time of writing sufficient information is not available to undertake more detailed assessment so it is a future action.

### 7.3 Water

Horotiu West is not included in WDC water infrastructure planning. The proposed development will need to be assessed in the WDC water network model to determine whether any service level issue will arise as a result. Preliminary water demand figures have been provided to WDC at the time of writing and modelling is yet to be carried out.

Horotiu has an existing water network which, subject to modelling, can be extended to service Horotiu West. Principal mains (150 mm or larger) are located on Horotiu Bridge Road and rider mains (63 mm) extend to the site boundary on Great South Road and Kernott Road. Rider mains are not sufficient to service development but the principal mains can be extended through the site to create a loop main as is usual practice.

### 7.4 Future actions

The information in this report shows that it is possible service the development in conjunction with the Te Awa Lakes stormwater, and WDC water and wastewater networks (existing and planned respectively). Notwithstanding, a number of future actions are required to confirm capacity and scale of impacts on the water and wastewater networks as follows:

d. Test the addition of the Horotiu West development in the WDC water network model to determine whether there are any adverse effects on service levels in Horotiu.

e. Consider the impacts of increased wastewater flow from Horotiu West on the following:

   - The scale of the proposed major pump station.
   - The size of the proposed rising main direct to the WWTP.
   - The operation of the Ngaruawahia WWTP treatment process and associated discharge.

f. Consider the various options available for location and ownership of the proposed major pump station and minor pump stations required to service parts of Horotiu and the Horotiu West development.

It is noted that the wastewater pump station options provided in Section 5.4 may be affected by the relative timing of Horotiu West and WDCs proposed major pump station. Not all option will remain viable if one aspect is progressed prior to the options being assessed and adopted.

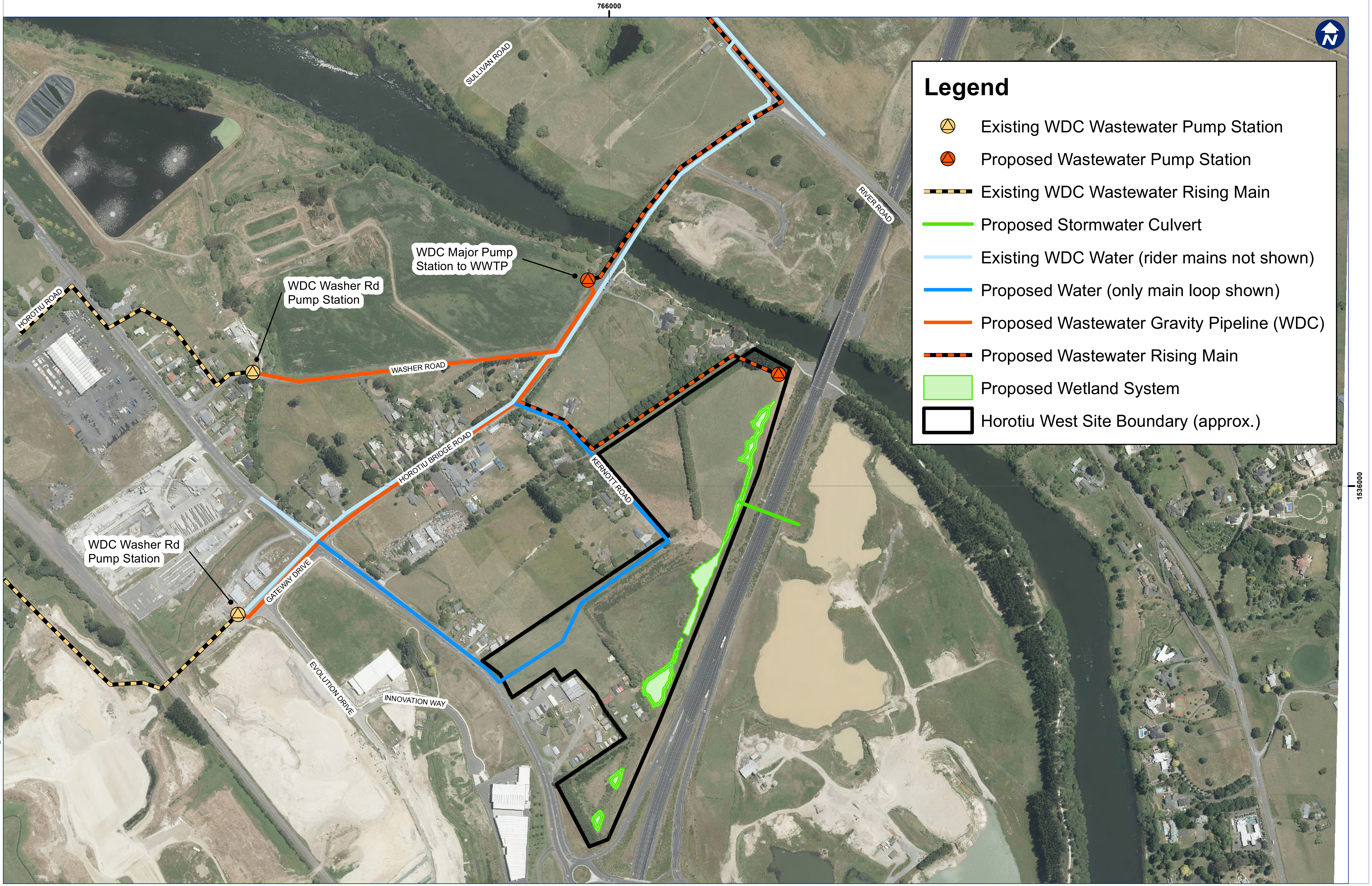
## 8.0 References

a. Te Awa Lakes Stormwater Management Plan, CKL, 30 October 2017

b. Waikato District Council Catchment Management Plan – Horotiu Structure Plan Area, T+T, March 2015

# Appendix A

## Infrastructure Schematic Plans




12 Mar 2018 11:57

P:\605X\60561768\4. Tech Work Area\4.99 GIS\02_Maps\2018-03-08 3W plan.mxd PDF File Name -



| Rev. | By | App. | Description | Date |
|---|---|---|---|---|

Notes:

| | | | |
|---|---|---|---|
| Printed | 12 March 2018 | | |
| Approved | JF | Date | 12-03-2018 |
| Designed | CH | Checked | |
| Drawn | CH | Checked | |
| File Name | | | |



Map features depicted in terms of NZTM projection.

Data Sources:
NZ Topographical Features – LINZ NZ National Topo Dataset 2014
Cadastral Boundaries – LINZ NZ Cadastral Dataset 2014

AECOM

Project: **PERRY DEVELOPMENTS - HOROTIU WEST**

Title: **CONCEPTUAL WATER SERVICING PLAN**

Scale: 1:6,000 (A3 size) 0.1 0.05 0 0.1 Kilometres

Status: CONCEPTUAL | Map No. **W-SK-01** | Sheet No. 1 of 1 | Rev. **A**

# Appendix B

## Calculations

**Concept Stormwater**

## TP108 / TP10 Catchment Curve Numbers

Sub catchment areas

| Sub Catchment ID | Existing Area (m2) | Design Area (m2) | Increase (m2) |
|---|---|---|---|
| 1 | 170946.0 | 170946.0 | 0.0 |
| 2 | | | 0.0 |
| 3 | | | 0.0 |
| 4 | | | 0.0 |
| 5 | | | 0.0 |
| 6 | | | 0.0 |
| 7 | | | 0.0 |
| 8 | | | 0.0 |
| 9 | | | 0.0 |
| TOTALS | 170946.0 | 170946.0 | 0.0 |

| | |
|---|---|
| | Input Cell |
| | Cell reference from/to another sheet |
| | Calculated cell - do not change |

| | Area Type | Soil Class | Impervious % | CN Impervious | CN Pervious | Weighted CN |
|---|---|---|---|---|---|---|
| 1 | Grass / Pasture | A | 0.0% | 98 | 74 | 74 |
| 2 | Road | n/a | 100.0% | 98 | 74 | 98 |
| 3 | Bush | A | 0.0% | 98 | 74 | 74 |
| 4 | Residential Lifestyle | A | 15.0% | 98 | 74 | 78 |
| 5 | Residential light | A | 55.0% | 98 | 74 | 87 |
| 6 | Residential heavy | A | 65.0% | 98 | 74 | 90 |

* Adjust CN values for local soil class

* Area types may be changed to suit project catchment characteristics (to maintain cell calculations always match 1-6)

CATCHMENTS - Existing

| | | 1 | 2 | 3 | 4 | 5 | 6 | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Sub Catchment ID | Area (m2) | Grass / Pasture (m2) | Road (m2) | Bush (m2) | Residential Lifestyle (m2) | Residential light (m2) | Residential heavy (m2) | Impervious % Existing | Impervious Area (m2) | Pervious Area (m2) |
| 1 | 170946.0 | 170946.0 | | | | | | 0% | 0.0 | 170946.00 |
| 2 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 3 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 4 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 5 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 6 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 7 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 8 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| 9 | 0.0 | | | | | | | 0 | 0.0 | 0.00 |
| TOTAL | 170946.0 | 170946.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | | 0.0 | 170946.0 |
| SCS Curve Number (CN) | | 74 | 98 | 74 | 78 | 87 | 90 | | | |
| SCS WEIGHTED CN | | 74 | | | | | | | | |

| | | |
|---|---|---|
| SCS WEIGHTED CN | 1 | 74 |
| | 2 | 0 |
| | 3 | 0 |
| | 4 | 0 |
| | 5 | 0 |
| | 6 | 0 |
| | 7 | 0 |
| | 8 | 0 |
| | 9 | 0 |

CATCHMENTS - Future

| Sub Catchment ID | Area (m2) | Grass (m2) | Road (m2) | Bush (m2) | Crops (m2) | Residential light (m2) | Residential heavy (m2) | Impervious % Future | Impervious Area (m2) | Pervious Area (m2) | Additional Impervious Area (m2) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 170946.0 | 53648.1 | 68270.8 | | | | 49027.1 | 59% | 100138.4 | 70807.56 | 100138.4 |
| 2 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 3 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 4 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 5 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 6 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 7 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 8 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| 9 | 0.0 | | | | | | | 0 | 0.0 | 0.00 | 0.0 |
| TOTAL | 170946.0 | 53648.1 | 68270.8 | 0.0 | 0.0 | 0.0 | 49027.1 | | 100138.4 | 70807.6 | 100138.4 |
| SCS Curve Number (CN) | | 74 | 98 | 74 | 78 | 87 | 90 | | | | |
| SCS WEIGHTED CN | | 88 | | | | | | | | | |

| | | |
|---|---|---|
| SCS WEIGHTED CN | 1 | 88 |
| | 2 | 0 |
| | 3 | 0 |
| | 4 | 0 |
| | 5 | 0 |
| | 6 | 0 |
| | 7 | 0 |
| | 8 | 0 |
| | 9 | 0 |

# Concept Stormwater

## TP108 Rainfall

| | |
|---|---|
| | Input Cell |
| | Use in HEC model |
| | Cell reference from/to another sheet |
| | Calculated cell - do not change |

Project Name: Horotiu West Wetland (combined assessment, split for detailed design)

| | Catchment Name | | Horotiu West | |
|---|---|---|---|---|
| | Development Phase | Units | Pre | Post |
| All Storms | % Impervious | | 0% | 59% |
| | Catchment Area - ha | Perv | 17.0946 | 7.0808 |
| | | Imp | 0.0000 | 10.0138 |
| | | Total - ha | 17.0946 | 17.0946 |
| | | A - km2 | 0.1709 | 0.1709 |
| | SCS Weighted Curve Number | | 74 | 88 |
| | Initial abstraction (IA) | mm | 5.0 | 2.1 |
| | Channelisation factor (C) | | 1.0 | 0.7 |
| | Catchment length (L) | km | 0.526 | 0.526 |
| | Catchment slope (Sc) | m/m | 0.0133 | 0.0133 |
| | Channelisation factor | | 0.59 | 0.79 |
| | Time of Concentration (tc) | hours | 0.45 | 0.27 |
| | SCS Lag for HEC - HMS....(tp) | hours | 0.30 | 0.18 |
| | Potential catchment retention (S) | mm | 89 | 34 |
| 2 Year | 24hr Precipitation (P24) | mm | 64 | 72 |
| | c* | | 0.23 | 0.50 |
| | q* (from Figure 5.1 of TP 108) | m3/km2mm | 0.030 | 0.110 |
| | Peak Flow Rate (qp) | m3/s | 0.326 | 1.358 |
| | Runoff depth (Q24) | mm | 23 | 47 |
| | Runoff volume (V24) | m3 | 3971 | 8040 |
| 10 Year | 24hr Precipitation (P24) | mm | 93 | 111 |
| | c* | | 0.32 | 0.61 |
| | q* (from Figure 5.1 of TP 108) | m3/km2mm | 0.040 | 0.124 |
| | Peak Flow Rate (qp) | m3/s | 0.633 | 2.347 |
| | Runoff depth (Q24) | mm | 43 | 82 |
| | Runoff volume (V24) | m3 | 7418 | 14099 |
| 100 Year | 24hr Precipitation (P24) | mm | 148 | 185 |
| | c* | | 0.44 | 0.72 |
| | q* (from Figure 5.1 of TP 108) | m3/km2mm | 0.057 | 0.137 |
| | Peak Flow Rate (qp) | m3/s | 1.441 | 4.323 |
| | Runoff depth (Q24) | mm | 88 | 154 |
| | Runoff volume (V24) | m3 | 15037 | 26249 |

q* Lookup Table
For min tc (0.17hr*)

| c* | q* |
|---|---|
| 0.05 | 0.026 |
| 0.1 | 0.031 |
| 0.15 | 0.048 |
| 0.2 | 0.06 |
| 0.25 | 0.073 |
| 0.3 | 0.084 |
| 0.35 | 0.096 |
| 0.4 | 0.106 |
| 0.45 | 0.115 |
| 0.5 | 0.124 |
| 0.55 | 0.13 |
| 0.6 | 0.139 |
| 0.65 | 0.145 |
| 0.7 | 0.15 |
| 0.75 | 0.155 |
| 0.8 | 0.16 |
| 0.85 | 0.161 |
| 0.9 | 0.163 |
| 0.95 | 0.164 |
| 1 | 0.166 |

* for tc >0.17hr use graph and input manually when prompted by target cell showing "USE GRAPH"

Rainfall Lookup Table

| ARI | Depth | Depth w/cc |
|---|---|---|
| Quality Storm | 21.2 | 24.1 |
| 2 | 63.6 | 72.2 |
| 5 | - | - |
| 10 | 92.6 | 110.7 |
| 20 | - | - |
| 50 | - | - |
| 100 | 147.9 | 184.6 |

Notes regarding revised TP108 depths:

When using TP108 depths climate change is not included and should be applied if required.

If not using TP108 depths, design primary system based on local Council or CMP requirements.

For non Auckland areas use revised rainfall from HIRDS or local Council Codes and adjust as appropriate for climate change as per MfE requirements.

NZTA's requirement based on 10yr ARI may supersede local council design requirements.

| | Catchment Name | | PRE-DEV | Horotiu West - WQV EDV SPLIT | |
|---|---|---|---|---|---|
| | Land type | Units | Pervious | Pervious | Impervious |
| All Storms | % Impervious | | 0% | 0% | 100% |
| | Catchment Area - ha | Perv | 17.0946 | 7.0808 | 0.0000 |
| | | Imp | 0.0000 | 0.0000 | 10.0138 |
| | | Total - ha | 17.0946 | 7.0808 | 10.0138 |
| | | A - km2 | 0.1709 | 0.0708 | 0.1001 |
| | SCS Weighted Curve Number | | 74 | 74 | 98 |
| | Initial abstraction (IA) | mm | 5.0 | 5.0 | 0.0 |
| | Channelisation factor (C) | | 1.0 | 1.0 | 0.6 |
| | Catchment length (L) | km | 0.526 | 0.526 | 0.526 |
| | Catchment slope (Sc) | m/m | 0.0133 | 0.0133 | 0.0133 |
| | Channelisation factor | | 0.59 | 0.59 | 0.96 |
| | Time of Concentration (tc) | hours | 0.45 | 0.45 | 0.21 |
| | SCS Lag for HEC - HMS....(tp) | hours | 0.30 | 0.30 | 0.14 |
| | Potential catchment retention (S) | mm | 89 | 89 | 5 |
| | Retention / Runoff depth | mm | 5 | 5 | 5 |
| | Runoff volume (V24) | m3 | 855 | 354 | 501 |
| | Total retention | m3 | | 855 | |
| WQV | 1/3 2yr 24hr Precipitation (P24) | mm | 24.1 | 24.1 | 24.1 |
| | Runoff depth (Q24) | mm | 3 | 3 | 20 |
| | Runoff volume (V24) | m3 | 574 | 238 | 1983 |
| | Total WQV | m3 | | 2221 | |
| | Permanent storage volume (WQV/2 x 0.75) | m3 | | 1480 | |
| EDV | 24hr Precipitation (P24) | mm | 24.0 | 24.0 | 24.0 |
| | Runoff depth (Q24) | mm | 3 | 3 | 20 |
| | Runoff volume (V24) | m3 | 570 | 236 | 1976 |
| | Total EDV | m3 | | 2213 | |
| | EDV volume in accordance with AC unitary plan (E10.6.3.1.1) without retention | | | 1642 | |
| | EDV volume in accordance with AC unitary plan (E10.6.3.1.1) with retention | | | 788 | |

| | | | | ITS requirements |
|---|---|---|---|---|
| Permanent storage zone volume (m3) | | | 1480 | |
| 0-0.2m deep | Volume (m3) | 38% | 563 | 40% ± 5 |
| | Area (m2) | 0.13 | 4327 | |
| 0.2-0.35 deep | Volume (m3) | 38% | 563 | 40% ± 5 |
| | Area (m2) | 0.28 | 2009 | |
| 0.35-1.2 deep | Volume (m3) | 12% | 178 | 10% ± 2 |
| | Area (m2) | 0.8 | 222 | |
| 0.35-1.5 deep | Volume (m3) | 12% | 178 | 10% ± 2 |
| | Area (m2) | 1.1 | 161 | |
| Total area of permanent storage | | | 6720 | |
| Forebay (15% WQV, 1m deep) | Volume (m3) | 15% | 333 | |
| | Area (m2) | 1 | 333 | |
| Total area required incl. forebays | | | 7053 | |
| Permanent storage area available | | OK | 7167 | |

## Concept Stormwater

**Circular outlet weir**

| | | |
|---|---|---|
| 100 Year ARI design flow (L/s) | 4323.258529 | From TP108 assessment |
| Diameter (d) | 21.5 | Standard 2100mm manhole crest diameter (decimeters) |
| Height over wier (h) | 20.93 | Solver cell to manipulate (decimeters) |
| h/d | 0.97 | Equation component |
| Coefficient of discharge (Cd) | 0.60 | Per equation 4.5 |
| Flow (Q, L/s) | 4323.258223 | Per equation 4.5, Solver target cell |

| | | |
|---|---|---|
| **Solution height over weir (m)** | **0.21** | Rounded, use in Area_Volume design |

**Equation 4.5:** Circular weir (Addison 1941)

$$Q = C_d\left[10.12\left(\frac{h}{d}\right)^{1.975} - 2.66\left(\frac{h}{d}\right)^{3.78}\right](d)^{5/2}$$

**where**
$Q$ = discharge (liters/sec)
$d$ = diameter of circular orifice (decimeters)
$h$ = height over the weir (decimeters)
$C_d$ = coefficient of discharge as given by:

$$C_d = 0.555 + \frac{1}{110\left(\frac{h}{d}\right)} + 0.041\left(\frac{h}{d}\right)$$

Iteration notes:
1 - single 1050mm (std scruffy dome size) yeilded a 1.3m height which was too much to be able to fit in the EDV
2 - halved the design flow therefore assuming two 1050mm manholes (std scruffy dome size) which yeilded 0.7m - still too high
3 - single 1500mm yeilded a 0.8m height which was too much to be able to fit in the EDV
4 - single 1800mm yeilded a 0.5m height which was too much to be able to fit in the EDV
5 - single 2100mm yeilded a 0.3m height which was OK to be able to fit in the EDV and permanent volume
6 - single 2400mm yeilded a 0.2m height which was OK to be able to fit in the EDV and permanent volume

# Concept Stormwater

| AREA-DEPTH | | | | | | | VOLUME-DEPTH | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wetland No. / Depth above or below max water level** | **1** | **2** | **3** | **4** | **5** | **TOTAL** | **1** | **2** | **3** | **4** | **5** | **TOTAL** | **INVERSE VOLUME** | | | |
| 0.3 | FREEBOARD | | | | | | | | | | | | | | UNLINED | |
| 0.2 | | | | | | | | | | | | | | | | |
| 0.1 | | | | | | | | | | | | | | | | |
| **0** | **540** | **580** | **2950** | **3015** | **2635** | **9720** | **615** | **658** | **4235** | **3258** | **2923** | **11688** | **0** | CIRCULAR WEIR | | |
| -0.1 | 515 | 553 | 2861 | 2859 | 2508 | 9295 | 562 | 601 | 3944 | 2964 | 2665 | 10737 | 951 | | | |
| -0.2 | 490 | 526 | 2771 | 2702 | 2380 | 8869 | 512 | 547 | 3663 | 2686 | 2421 | 9829 | 1859 | SERVICE OUTLET | | |
| -0.3 | 465 | 499 | 2682 | 2546 | 2253 | 8444 | 464 | 496 | 3390 | 2423 | 2189 | 8963 | 2725 | EDV (min 2213m³) | | 866 |
| -0.4 | 440 | 472 | 2592 | 2389 | 2125 | 8018 | 419 | 447 | 3127 | 2177 | 1971 | 8140 | 3548 | | | 1689 |
| -0.5 | 415 | 445 | 2503 | 2233 | 1998 | 7593 | 376 | 401 | 2872 | 1946 | 1764 | 7359 | 4328 | | | 2469 |
| -0.6 | 390 | 418 | 2413 | 2076 | 1870 | 7167 | 336 | 358 | 2626 | 1730 | 1571 | 6621 | 5066 | AVAILABLE FOR PERMANENT WATER VOLUME MAKEUP (BANDED BATHYMETRY) AND LINED TO RETAIN WATER | LINED | |
| -0.7 | 365 | 391 | 2324 | 1920 | 1743 | 6742 | 298 | 318 | 2389 | 1530 | 1390 | 5926 | 5762 | | | |
| -0.8 | 340 | 364 | 2234 | 1763 | 1615 | 6316 | 263 | 280 | 2161 | 1346 | 1223 | 5273 | 6414 | | | |
| -0.9 | 315 | 337 | 2145 | 1607 | 1488 | 5891 | 230 | 245 | 1942 | 1178 | 1067 | 4663 | 7025 | | | |
| **-1** | **290** | **310** | **2055** | **1450** | **1360** | **5465** | **200** | **213** | **1733** | **1025** | **925** | **4095** | **7593** | | | |
| -1.1 | 272 | 291 | 1991 | 1365 | 1273 | 5191 | 172 | 182 | 1530 | 884 | 793 | 3562 | 8125 | | | |
| -1.2 | 254 | 271 | 1926 | 1280 | 1186 | 4917 | 146 | 154 | 1334 | 752 | 670 | 3057 | 8631 | | | |
| -1.3 | 236 | 252 | 1862 | 1195 | 1099 | 4643 | 121 | 128 | 1145 | 628 | 556 | 2579 | 9109 | | | |
| -1.4 | 218 | 232 | 1797 | 1110 | 1012 | 4369 | 98 | 104 | 962 | 513 | 451 | 2128 | 9559 | | | |
| -1.5 | 200 | 213 | 1733 | 1025 | 925 | 4095 | 78 | 82 | 786 | 406 | 354 | 1705 | 9983 | | | |
| -1.6 | 182 | 193 | 1668 | 940 | 838 | 3821 | 58 | 62 | 616 | 308 | 266 | 1309 | 10378 | | | |
| -1.7 | 164 | 174 | 1604 | 855 | 751 | 3547 | 41 | 43 | 452 | 218 | 186 | 941 | 10747 | | | |
| -1.8 | 146 | 154 | 1539 | 770 | 664 | 3273 | 26 | 27 | 295 | 137 | 115 | 600 | 11088 | | | |
| -1.9 | 128 | 135 | 1475 | 685 | 577 | 2999 | 12 | 12 | 144 | 64 | 53 | 286 | 11401 | | | |
| **-2** | **110** | **115** | **1410** | **600** | **490** | **2725** | **0** | **0** | **0** | **0** | **0** | **0** | **11688** | | | |

# Culvert Calculator Report
# Horotiu West Expressway

Solve For: Section Size

| Culvert Summary | | | | | |
| --- | --- | --- | --- | --- | --- |
| Allowable HW Elevation | 16.00 | m | Headwater Depth/Height | 1.20 | |
| Computed Headwater Elevation | 15.83 | m | Discharge | 4.3200 | $m^3/s$ |
| Inlet Control HW Elev. | 15.75 | m | Tailwater Elevation | 13.50 | m |
| Outlet Control HW Elev. | 15.83 | m | Control Type | Entrance Control | |

| Grades | | | | | |
| --- | --- | --- | --- | --- | --- |
| Upstream Invert | 14.00 | m | Downstream Invert | 13.50 | m |
| Length | 75.00 | m | Constructed Slope | 0.006667 | m/m |

| Hydraulic Profile | | | | | |
| --- | --- | --- | --- | --- | --- |
| Profile | S2 | | Depth, Downstream | 0.96 | m |
| Slope Type | Steep | | Normal Depth | 0.96 | m |
| Flow Regime | Supercritical | | Critical Depth | 1.08 | m |
| Velocity Downstream | 3.59 | m/s | Critical Slope | 0.004748 | m/m |

| Section | | | | | |
| --- | --- | --- | --- | --- | --- |
| Section Shape | Circular | | Mannings Coefficient | 0.013 | |
| Section Material | Concrete | | Span | 1.52 | m |
| Section Size | 1500 mm | | Rise | 1.52 | m |
| Number Sections | 1 | | | | |

| Outlet Control Properties | | | | | |
| --- | --- | --- | --- | --- | --- |
| Outlet Control HW Elev. | 15.83 | m | Upstream Velocity Head | 0.50 | m |
| Ke | 0.50 | | Entrance Loss | 0.25 | m |

| Inlet Control Properties | | | | | |
| --- | --- | --- | --- | --- | --- |
| Inlet Control HW Elev. | 15.75 | m | Flow Control | N/A | |
| Inlet Type | Square edge w/headwall | | Area Full | 1.8 | $m^2$ |
| K | 0.00980 | | HDS 5 Chart | 1 | |
| M | 2.00000 | | HDS 5 Scale | 1 | |
| C | 0.03980 | | Equation Form | 1 | |
| Y | 0.67000 | | | | |

# Populations

| | | | |
|---|---|---|---|
| **Job No.** | 60561678 | | |
| **Computed:** | C HARDY | **Date:** | 27/02/2018 |
| **Checked:** | | **Date:** | |

| | |
|---|---|
| **Project** | Perry Horotiu West |
| **Consultant** | AECOM Ltd |
| **Date** | 8/03/2018 |

**Poulations**

Catchment Data

| | | |
|---|---|---|
| Area | 18.4 | hectares |
| Density HCC | 45 | persons/hectare |
| Persons per dwelling | 2.7 | |
| Masterplan (MP) Concept 1 | 285 | dwellings |
| MP Concept 2 | 254 | dwellings |

| Population Case | Population | Person/hectare |
|---|---|---|
| HCC ITS WW standard | 828 | 45 |
| MP Concept 1 | 770 | 42 |
| MP concept 2 | 686 | 37 |

**Use masterplan concept 1**

## Water Supply Concept Design

**Job No.** 60561678
**Computed:** C HARDY **Date:** 27/02/2018
**Checked:** **Date:**

**Project** Perry Horotiu West
**Consultant** AECOM Ltd
**Date** 8/03/2018

**Calculation of Flows**

Catchment Data

| | | |
|---|---|---|
| Assumed Population Density | 42 | person/ha |
| Water Consumption | 260 | L/person/day |
| Peaking Factor | 5.0 | |
| Fireflow (FW2) | 25 | L/s |
| Fireflow (FW3) | 50 | L/s |
| Fireflow (FW4) | 100 | L/s |

| Design Case | Catchment Area (Ha) | ADF (m3/day) | ADF (L/s) | PDF (m3/day) | PDF (L/s) | Combined (FW2 + ADF) | Combined (FW3 + ADF) | Combined (FW4 + ADF) | |
|---|---|---|---|---|---|---|---|---|---|
| Horotiu West (HW) | 18.4 | 200200 | 2.3 | 1001000 | 11.6 | 27.3 | 52.3 | 102.3 | HW development only |
| Residential block | 20.5 | 223049 | 2.6 | 1115245 | 12.9 | 27.6 | 52.6 | 102.6 | Residential zoned land on same block |
| HW + Residential block | 38.9 | 423249 | 4.9 | 2116245 | 24.5 | 29.9 | 54.9 | 104.9 | HW development plus residential land on same block |
| | 0 | 0 | 0.0 | 0 | 0.0 | 25.0 | 50.0 | 100.0 | |

**Pipe Sizing Calculations**

| | | |
|---|---|---|
| Roughness for pipes | 0.06 | mm |
| Fluid Viscosity n | 0.0012 | kg/m-s |
| Density | 1000 | kg/m3 |
| Reynolds Number | Re = (ρ*V*D)/μ | |
| Headloss Coeff | K=λL/(2gDA^2) | |

*All conceptual pipelines are base don a two ended servicable supply (ie peak flows are half the total)*

| Design Case | | Trial Dia. (mm) | Area ($m^2$) | Length (m) | k/D | Q (l/s) | Re | l | K | $h_L$ (m) | $h_L$/Q | V (m/s) | $H_L$/L (m/km) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Horotiu West (HW) | **Demand Peak** | 150 | 0.02 | 2400 | 0.0004 | 5.8 | 40976 | 0.02 | 60153.20 | 2.02 | 348.46 | 0.33 | 0.84 |
| | **FW2 Peak** | 150 | 0.02 | 2400 | 0.0004 | 13.7 | 96615 | 0.02 | 52246.64 | 9.75 | 713.61 | 0.77 | 4.06 |
| | **FW3 Peak** | 200 | 0.03 | 2400 | 0.0003 | 26.2 | 138775 | 0.02 | 11464.63 | 7.84 | 299.90 | 0.83 | 3.27 |
| | **FW4 Peak** | 250 | 0.05 | 2400 | 0.00024 | 51.2 | 217124 | 0.02 | 3474.39 | 9.09 | 177.74 | 1.04 | 3.79 |
| | **FW4 Peak** | 300 | 0.07 | 2400 | 0.0002 | 51.2 | 180936 | 0.02 | 1400.33 | 3.66 | 71.64 | 0.72 | 1.53 |
| Residential block | **Demand Peak** | 150 | 0.02 | 2400 | 0.0004 | 6.5 | 45652 | 0.02 | 58943.99 | 2.46 | 380.42 | 0.37 | 1.02 |
| | **FW2 Peak** | 150 | 0.02 | 2400 | 0.0004 | 13.8 | 97550 | 0.02 | 52178.22 | 9.92 | 719.58 | 0.78 | 4.13 |
| | **FW3 Peak** | 200 | 0.03 | 2400 | 0.0003 | 26.3 | 139477 | 0.02 | 11457.26 | 7.92 | 301.22 | 0.84 | 3.30 |
| | **FW4 Peak** | 250 | 0.05 | 2400 | 0.00024 | 51.3 | 217685 | 0.02 | 3473.39 | 9.14 | 178.15 | 1.04 | 3.81 |
| | **FW4 Peak** | 300 | 0.07 | 2400 | 0.0002 | 51.3 | 181404 | 0.02 | 1399.85 | 3.68 | 71.80 | 0.73 | 1.53 |
| HW + Residential block | **Demand Peak** | 150 | 0.02 | 2400 | 0.0004 | 12.2 | 86628 | 0.02 | 53050.55 | 7.96 | 649.70 | 0.69 | 3.32 |
| | **FW2 Peak** | 150 | 0.02 | 2400 | 0.0004 | 14.9 | 105745 | 0.02 | 51621.37 | 11.54 | 771.71 | 0.85 | 4.81 |
| | **FW3 Peak** | 200 | 0.03 | 2400 | 0.0003 | 27.4 | 145623 | 0.02 | 11395.13 | 8.59 | 312.79 | 0.87 | 3.58 |
| | **FW4 Peak** | 250 | 0.05 | 2400 | 0.00024 | 52.4 | 222602 | 0.02 | 3464.88 | 9.53 | 181.73 | 1.07 | 3.97 |
| | **FW4 Peak** | 300 | 0.07 | 2400 | 0.0002 | 52.4 | 185502 | 0.02 | 1395.78 | 3.84 | 73.21 | 0.74 | 1.60 |
| | **Demand Peak** | 0 | 0.00 | 0 | #DIV/0! | 0.0 | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! |
| | **FW2 Peak** | 0 | 0.00 | 0 | #DIV/0! | 0.0 | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! |
| | **FW3 Peak** | 0 | 0.00 | 0 | #DIV/0! | 0.0 | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! |
| | **FW4 Peak** | 0 | 0.00 | 0 | #DIV/0! | 0.0 | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! |
| | **FW4 Peak** | 0 | 0.00 | 0 | #DIV/0! | 0.0 | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! | #DIV/0! |

Recommended HL/m is less than 5m/km under peak flow (concept for this assessment)
Recommended velocity is 1-3m/s under peak flow (concept for this assessment)
This assessment is for conceptual purposes only and should be confirmed in the WDC model which accounts for lcoal network conditons (pressure and local pipeline diameters and losses)
FW3 and FW4 may not be able to be provided by the network so the larger diameters may not be required, conversely pressure at the connection point could be high and F# or FW4 could be achieved with smaller diameters.

## Wastewater Pumpstation Concept Design

**Job No.** 60220144
**Computed:** C HARDY **Date:** 27/02/2018
**Checked:** **Date:**

**Project** Horotiu Farms Ltd Te Rapa
**Consultant** AECOM Ltd
**Date** 8/03/2018

**Calculation of Flows** NOT ALL SCENARIOS ARE ABLE TO BE ASSESSED AT THIS ATEG PENDING ADDITIONAL CATCHMENT INFORMATION

Catchment Data

Assumed Population Density = 42 persons/ha
Wastewater production = 200 L/person/day
Peaking Factor = Varies [Refer HCC ITS]

**Infiltration Allowance** 2,250 L/ha/day

**Surface Water Ingress** 16,500 L/ha/day

| Design Case | Catchment Area (Ha) | Equivalent population | Peaking Factor | ADF (L/day) | ADF (L/s) | PDF (L/day) | PDF (L/s) | PWWF (L/day) | PWWF (L/s) |
|---|---|---|---|---|---|---|---|---|---|
| HW | 18.4 | 770 | 3.1 | 195,400 | 2.3 | 518,800 | 6.0 | 822,400 | 9.5 |
| Residential | 20.5 | 858 | 3.0 | 217,701 | 2.5 | 560,853 | 6.5 | 899,103 | 10.4 |
| HW+Residential | 38.9 | 1628 | 2.9 | 413,101 | 4.8 | 1,031,696 | 11.9 | 1,673,546 | 19.4 |
| Ex. Industrial | 0 | 0 | 0 | 0 | 0.0 | 0 | 0.0 | 0 | 0.0 |
| Ex. Residential | 0 | 0 | 0 | 0 | 0.0 | 0 | 0.0 | 0 | 0.0 |
| Ex. Ind+Res | 0 | 0 | 0 | 0 | 0.0 | 0 | 0.0 | 0 | 0.0 |
| All | 0 | 0 | 0 | 0 | 0.0 | 0 | 0.0 | 0 | 0.0 |

| Peak Volume (m3/day) |
|---|
| 499,000 |
| 555,951 |
| 1,054,951 |
| - |
| - |
| - |
| - |

**Storage Volume**

*Provide a minimum of 6hrs of storage at Average Daily Flow before emergency overflow occurs.*

Equivgalent storage time 9 hours

| Design Case | ADF (m3/day) | Minimum Storage (m3) |
|---|---|---|
| HW | 195.4 | 74 |
| Residential | 217.7 | 82 |
| HW+Residential | 413.1 | 155 |
| Ex. Industrial | 0.0 | 0 |
| Ex. Residential | 0.0 | 0 |
| Ex. Ind+Res | 0.0 | 0 |
| All | 0.0 | 0 |

**Pump & Rising Main Sizing**
*The pumping range shall be selected to give between 1 and 15 starts per hour at peak daily flow*

*Design to ensure velocities in the rising main are between 1m/s and 3m/s under normal operation conditions and shall have a minimum diameter of 80mm*

Simple Pump Station Analysis

Input
Manipulate
Target cell
Design Output
Calculated cell

Velocity notes:
Single duty/design pump = 1.2 m/s optimal (0.9 m/s minimum)
Dual duty/design pumps = 1.5 m/s at PWWF (design) giving approx 1.2m/s when one pump operating at less than PWWF
Dual duty/design pumps = 1.5 m/s at PWWF (design) giving approx 1.2m/s when one pump operating at less than PWWF
Lift pump station no rising main - discharge pipework only = 2.5 m/s
Unless otherwise stated.

Other:
Overall pump effiency = 0.6

Recommended HL/m is less than 5m/km under peak flow

| Design Case | Rising Main Length (m) | Rising Main ID (m) | Nominal Capacity (m3/s) | Velocity (m/s) | Static head (m) | Roughness (m) | Relative roughness | Reynolds No. | f | Moody =1 | Friction Head (m) | Total Head (m) | Estimated Power Required (kW) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| HW+Residential | 530 | 0.15 | 0.019 | 1.09610314 | 10.0 | 0.0015 | 0.0100 | 164415 | 0.039 | 1.00 | 8.44 | 18.44 | 5.8 |
| Ex. Ind+Res | 10000 | 0.15 | 0.000 | 0 | 10.0 | 0.0015 | 0.0100 | 0 | 0.039 | #DIV/0! | 0.00 | 10.00 | 0.0 |
| All | 10000 | 0.15 | 0.000 | 0 | 10.0 | 0.0015 | 0.0100 | 0 | 0.039 | #DIV/0! | 0.00 | 10.00 | 0.0 |