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Reference: MDL000972

9 April 2021

Waipa District Council
Private Bag 2402
Te Awamutu 3840

Attention: Mark Batchelor

Sent via email: Mark.Batchelor@ckl.co.nz

Dear Mark

RE: 3Ms of Cambridge GP Limited – Response to section 92 Request for Further Information

As detailed in the letter sent to the Waipa District Council ("**WDC**") dated 26 March 2021, 3Ms of Cambridge GP Limited ("**3Ms**") has been seeking additional input from various technical consultants to address matters raised in the s92 letter from WDC. This letter provides a response to the further information requests that were not able to be responded to earlier.

Please note that these responses have been prepared by Beca, Harrison Grierson, McCaffrey Engineering, Stantec and Mitchell Daysh on behalf of 3Ms.

Stormwater Matters

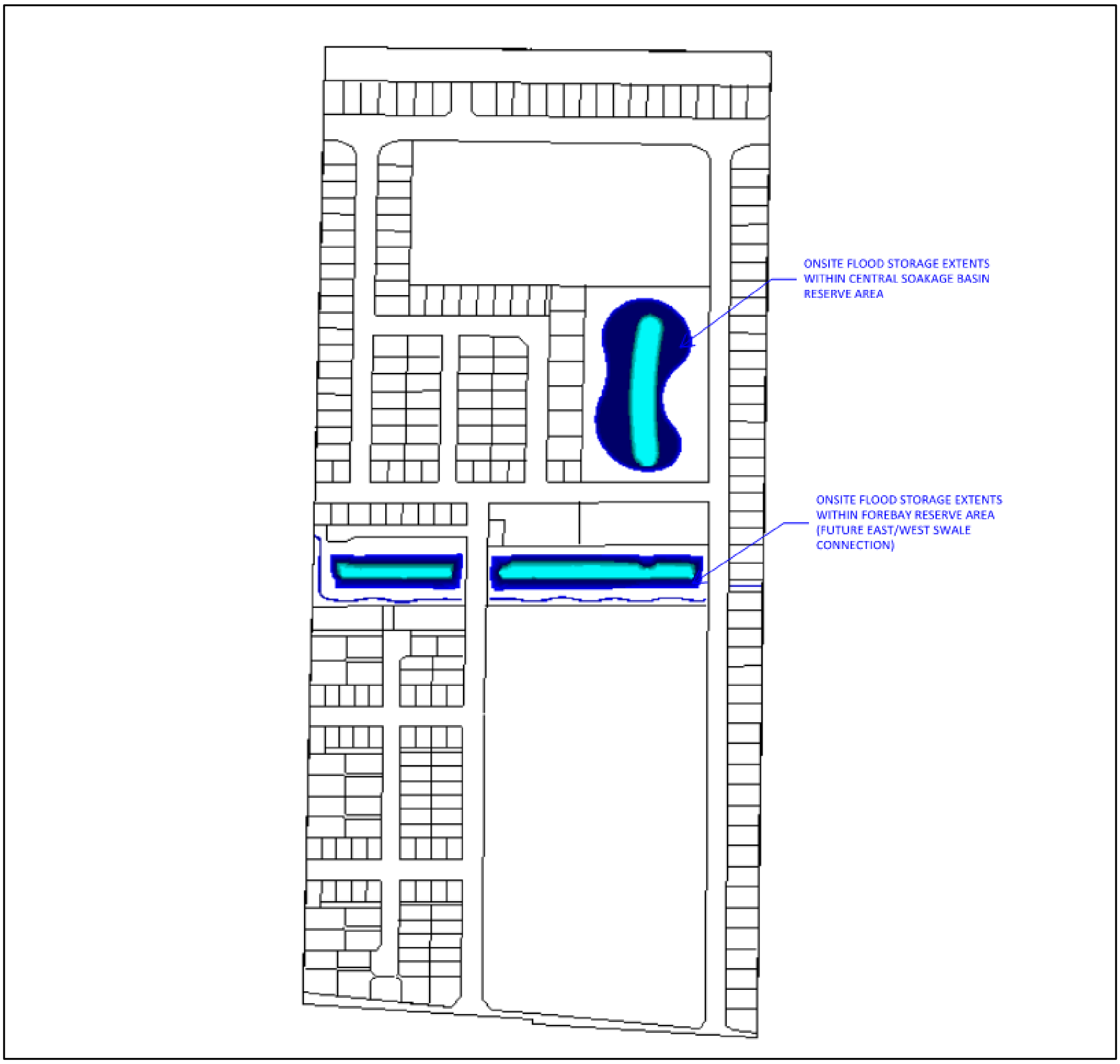
Question 1: *Please provide and demonstrate technical engineering assessment proving the onsite stormwater management will have sufficient capacity to protect the surrounding locality from potential for stormwater flooding.*

The stormwater solution provided within the 3Ms development site has been sized to provide flood storage to accommodate a 24hr/1% AEP storm event within proposed stormwater reserve areas, including more than 300mm freeboard to adjacent areas. The following table provides an assessment of runoff volumes generated by the development.

Storm Event	Storm Duration (h)	Total Runoff (m^3)	Level Without any Soakage (m RL)	Required Storage (m^3)	Level with Soakage (m RL)	Time to Clear (h)
ARI = 2	24	17,614	61.95	10,119	61.12	45
ARI = 10	24	27,786	62.52	18,524	62.01	72
ARI = 50	24	39,383	63.07	30,087	62.64	102

Storm Event	Storm Duration (h)	Total Runoff (m^3)	Level Without any Soakage (m RL)	Required Storage (m^3)	Level with Soakage (m RL)	Time to Clear (h)
ARI = 100	24	44,315	63.29	35,019	62.88	114
Overflow level = RL63.4m						

The following diagram shows the extent of flooding within the proposed reserve area during a 24hr/1% AEP storm event demonstrating there is sufficient capacity to manage all development runoff within the proposed stormwater system. Please note that the flood storage provided has adequate capacity to retain the full 24hr/1% AEP storm event in the event of a total blockage within the proposed soakage system.



Question 4: *Please advise whether the development is proposed to be connected to the stormwater network within the C2 growth cell, either temporarily or permanently. Please advise the timing of any such connections relative to development of the subdivision.*

The reason for this question is the east/west swale separates the land to the south of it from the on-site stormwater reserve and this swale appears on the plans to be wider than what is expected to be required for stormwater purposes. There is also connections form outside the application site to the balance of the C2 cell indicated on the application plans.

3Ms confirms that the proposed development, while initially designed to function as a 'self-contained system' is intended to be permanently connected to the stormwater network within the C2 growth in the future (as outlined in drawing 17001-C-0430). Stormwater from the site, including south of the development is conveyed through the east/west swale (which will act as a forebay) when it is then conveyed to the stormwater basin.

Timing for providing connection to the wide C2 growth cell stormwater network is as follows:

- The eastern (upstream) connection will be installed as part of the initial construction to ensure the required culvert pipe is installed below permanent roading infrastructure (i.e. Road 10). This approach will allow the upstream catchments to be connected to the system at any time.
- Infrastructure for the western (downstream) connection can be installed at any time allowing for east/west stormwater network connectivity through the 3MS site.

Exact timing for the completion of the above connections is subject to WDC delivery of the wider stormwater network.

The land required for the east-west swale is slightly larger than the previous WDC design for the following reasons:

- The batter slopes for the proposed forebay design are flatter than those previously proposed for this section of the east/west swale.
- The stormwater reserve includes additional land expected to be required for mitigation (NB: this was previously shown as general reserve areas for the previous east/west swale) - *refer to response to question 9 below for further details.*
- The invert level in the centre of forebay #1 has been lowered by 700mm to accommodate the upstream pipe networks.

It should be noted that the base of the forebays (east/west swale) are designed to provide pre-treatment prior to discharging runoff into the proposed central soakage basin area to the north. This treatment approach is consistent with the function of the previous design for this section of the east/west swale.

Question 7: *Please advise what effects proposed infiltration of stormwater through and below the iron pan within the infiltration basin will have on stability ground stability within the application site and outside its boundaries, and where or if it is expected to discharge and effects at that point.*

This should include but not be exclusive of any other matter your assessment may identify as relevant, confirmation that the soakage pond proposal will not result in additional ground flows to the C3 terraces and in turn cause erosion and slip and ground stability risk and effects.

An initial assessment of potential mounding (rise in groundwater level) due to stormwater infiltration was provided in the Beca (December 2020) report based on a simple Hantush equation and an anticipated design infiltration rate (100mm / hour). This assessment indicated that under the more typical design events (2-year and 10-year) the extent of mounding is expected to be no more than 80 metres from the centre of the basin, i.e. is limited to wholly within the development site and with groundwater levels adjacent the basin remaining at least 2 m bgl.

3Ms notes that as per the original assessment undertaken for Waipa (Beca, 2019), there will inevitably be additional groundwater discharge towards the C3 area as a result of the soakage basin, but this is considered necessary particularly in the longer term to offset the reduced upgradient groundwater flow to C3 that will arise from the increased impervious cover in C2 (and net reduction in direct rainfall recharge at the surface). However, this is considered to result in an overall net balance of the water budget as opposed to an increase in flow, and the calculated extent of mounding is not expected to result in a noticeable change in groundwater level or steepened flow gradient at the C3 terraces, and hence is not expected to result in any increase in instability.

It is noted that additional site testing has now been completed and has indicated a lower permeability than originally anticipated at the basin location. 3Ms is currently working through the implications for design infiltration rate as part of the detailed design and will be undertaken further (more detailed) assessment including modelling, to confirm that the above assessment remains valid. Whilst some localised increase in mounding can be anticipated as a result of the lower hydraulic conductivity, it is expected that the detailed design process can be used to manage effects to within that already assessed.

Question 9: *Please provide advice on the risk of instability along the side of the proposed stormwater pond and open swales within the C2 growth cell including but not limited to, lateral spread and bank erosion and how this risk will be avoided.*

Slope Stability/Lateral Spread

The initial assessment completed by WDC relating to the wide C2 growth cell stormwater network (within the 3MS site) indicates that, based on the recently measured groundwater levels, lateral spread risks associated within the proposed swale and central basin excavations are ‘less than 50mm lateral movement’ – refer to the plan attached in Appendix A. A detailed assessment of slope stability and liquefaction assessment will be completed as part of the detailed design phase.

Please note that additional land areas/offsets have been provided within the stormwater reserve areas adjacent to basin and forebay/swales to allow for potential mitigation of any risks that may arise through the detailed assessment results. Final mitigations (if any) will be confirmed as part of the final design.

<u>Bank Erosion</u>

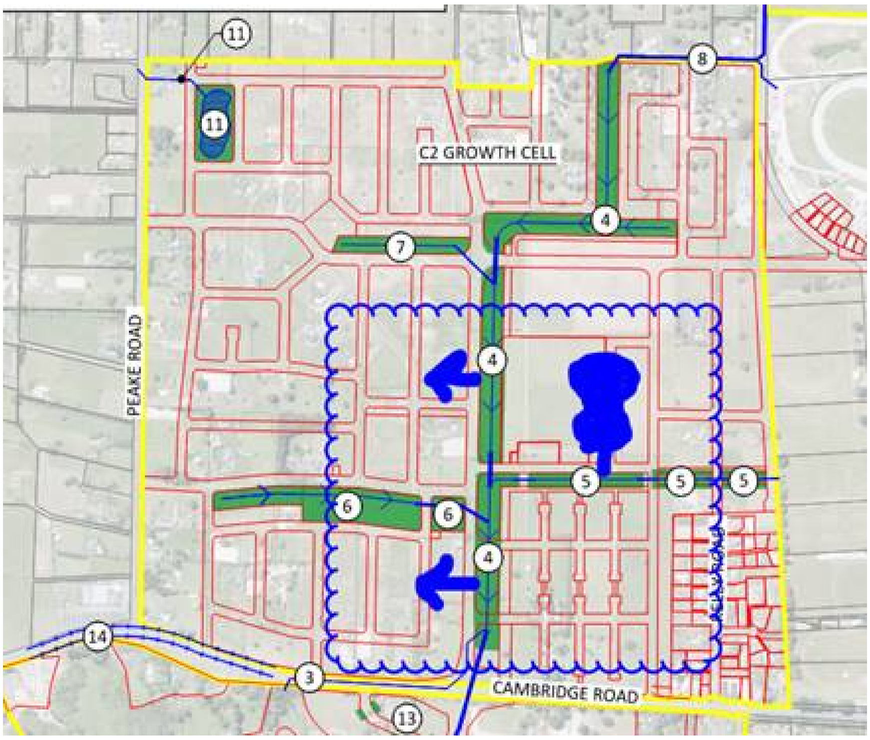
General velocities within the swale and stormwater basin are estimated at less than 0.3m/s. This indicates that standard vegetation planting will be sufficient to mitigate against potential bank erosion during operation as it is well below the allowable 1.5m/s allowed for in Table 8.1 of the WRC Stormwater Management Guideline. In areas of higher velocities (i.e. pipe outlets) scour protection will be used to prevent scour.

Question 10: *Please provide confirmation in writing from Waikato Regional Council that there are no issues with the stormwater discharge consent arising from the stormwater soakage pond and discharge proposals and particularly that consent for the discharges proposed has been obtained. Alternatively, if applicable, please describe how obtaining any consent for these discharges that may be required may be provided for.*

Representatives of 3Ms met with Waikato Regional Council ("**WRC**") on 24 February 2021.

Key notes from the meeting with Brian Richard and Megan Wood are as follows:

- No red flags were raised associated with 3Ms stormwater proposal subject to technical approval from WRC as part of the detailed design process enshrined in the consent held by WDC.
- WRC noted that a mounding assessment for the new location will be required, with 3Ms explaining that this was already being addressed. The results of the assessment will need to be included in WRC technical submission.
- As the WDC stormwater discharge permit lists all the key stormwater assets within a consent condition (see sketch below), WRC would like an application under s127 of the RMA to be lodged along with with the detailed design approval so this list of key assets can be updated to include the stormwater basin within the 3Ms developement. 3Ms understands they will need to pay for the costs associated with the s127 process, but WRC noted that the change would be straightforward.



3Ms will need to continue to engage with WDC regarding the s127 change to consent conditions as WDC is the consent holder and would need to therefore be the applicant of any such process. However, 3Ms can facilitate this.

Question 11: *Please provide the technical report prepared by BECA and referenced in the application that provided the advice your assessment of the hydrological effects of the stormwater soakage pond will have.*

See the attached assessment (Appendix B) but as noted above, further assessment is proposed to be undertaken as part of the detailed design process.

Question 12: *Please provide the stormwater pond soakage results confirming it is a viable option.*

As noted earlier, initial testing has now been completed at the basin location and has returned lower results than initially anticipated, but still within an acceptable range for soakage.

Testing was in the form of constant head tests, with 3 tests conducted in two piezometers (one test at the southern end and two tests at the northern end of basin). The piezometers are screened in the aquifer below the iron pan and into which the soakage is expected. The assessed in-situ hydraulic conductivity ranged from 1x10-5 m/s to 2.9x10-5 m/s; the latter being in the area of the proposed soakage array at northern end of basin. Taking an average of all tests (to account for variability across the site), this would be broadly equivalent to a raw (unfactored) rate of 78 mm/hour.

Further testing in the excavated basin is proposed to be undertaken shortly to confirm the in-situ hydraulic conductivity and assess any variability across the extent of the basin.

As already noted, 3Ms is currently working through implications for the design (factored) infiltration rate as part of the detailed design, whilst the hydraulic conductivity is lower than initially anticipated is in within the wider range of test results reported in the Cambridge area. A hydraulic conductivity of 1x10-5 m/s or higher is considered viable for soakage, for example Hamilton City Council expects soakage (with storage) to be considered where permeability is > 1x10-5 m/s.

Question 14: *Please advise how assurance of connection to and from the application site and adjoining land that may be proposed or required for stormwater management purposes until and after the on-site facilities are constructed.*

The existing open drain that provides connectivity through the 3Ms development site will remain in place at all times with a minor diversion proposed as part of the development works. This diversion will be completed as part of the initial bulk earthworks contract currently underway (and as authorised by a land use consent from WDC for the earthworks)

Future connections to the wider stormwater network within the C2 growth cell can be completed at any time without impacting the existing open drain (i.e. offline construction). Once completed it is proposed that the open drain be disestablished when the east/west and north south swales are commissioned and replaced with a walking/cycling connection in the same location.

Transportation and Roading

Questions 30-31 of the further information request letter are addressed Attachment C, which has been prepared by Stantec.

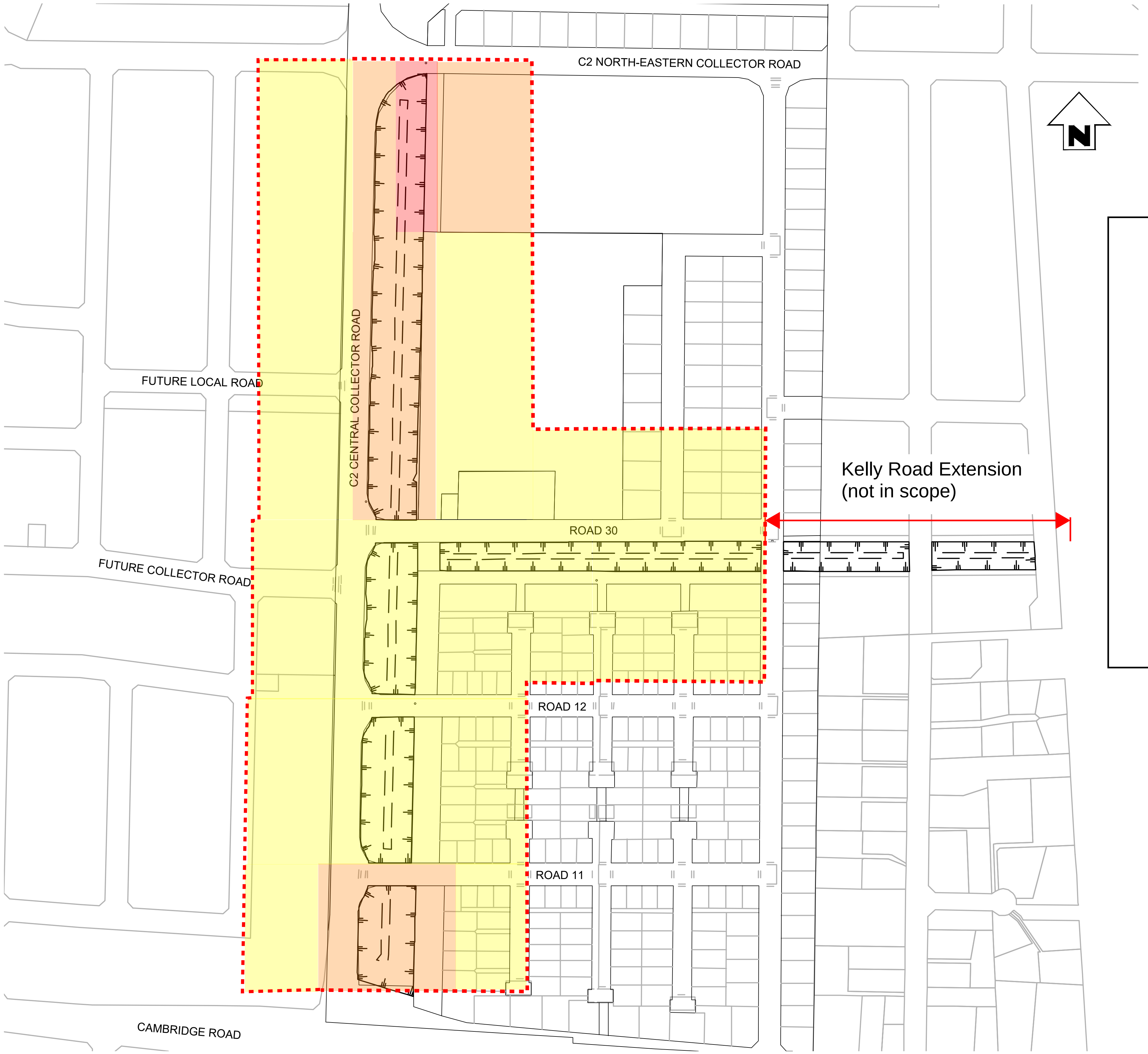
Please do not hesitate to contact me directly if any matters in this letter require further clarification.

Yours sincerely,

Abbie Fowler
Associate
Mitchell Daysh Ltd

Cc: Wayne Allan (Wayne.Allan@waipadc.govt.nz); Tony Quickfall (Tony.Quickfall@waipadc.govt.nz)

ATTACHMENT A – LATERAL SPREADING EFFECTS PLAN



40 0 40 80 120 160 200m

SCALE 1 : 4000

SITE PLAN

SCALE 1:4000

Legend

🟥	More than 100mm lateral movement
🟧	Between 50mm and 100mm lateral movement
🟨	Less than 50mm lateral movement
⋰	Not defined beyond here

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Disclaimer:
Areas and dimensions may be subject to scale error.
Scaling from this drawing is at the users risk.

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No.	Revision	By	Chk	Appd	Date
A	For review	JW			06/07/20

Drawing Originator: Beca

Original Scale (A1)	Design		
	Drawn		
Reduced Scale (A3)	Dsg Verifier		
1:4000	Dwg Check		
	* Refer to Revision 1 for Original Signature		

Client: Waipa District Council 3MS

Project: C2 Swale Assessment

Title: Lateral Spreading Effects with Deeper Groundwater

Discipline: GEOTECHNICAL

Drawing No.:

Rev.:

BLUEBEAM

DO NOT SCALE

IF IN DOUBT ASK.

ATTACHMENT B – TECHNICAL ASSESSMENT OF GROUNDWATER EFFECTS

3Ms Cambridge Subdivision

Technical Assessment of Groundwater Effects

Prepared for 3MS of Cambridge GP Ltd

Prepared by Beca Limited

1 December 2020

Creative people together transforming our world

Revision History

Revision Nº	Prepared By	Description	Date
0.1	James Botting and Sian France	Draft for Client comment	25/11/20
1.0	James Botting and Sian France	Final	1/12/20

Document Acceptance

Action	Name	Signed	Date
Prepared by	James Botting and Sian France		1/12/20
Approved by	Sian France		1/12/20
on behalf of	Beca Limited		

Contents

Appendices

1 Introduction

1.1 Background

Waipa District Council (WDC) have identified Cambridge as a growth area, with the population expected to almost double in the next 50 years (WDC Plan Change 7, 2017). WDC has been preparing for the increase in housing needs by developing frameworks (Structure Plans) for managing residential development in a series of identified Growth Cells in the Cambridge area (Figure 1).

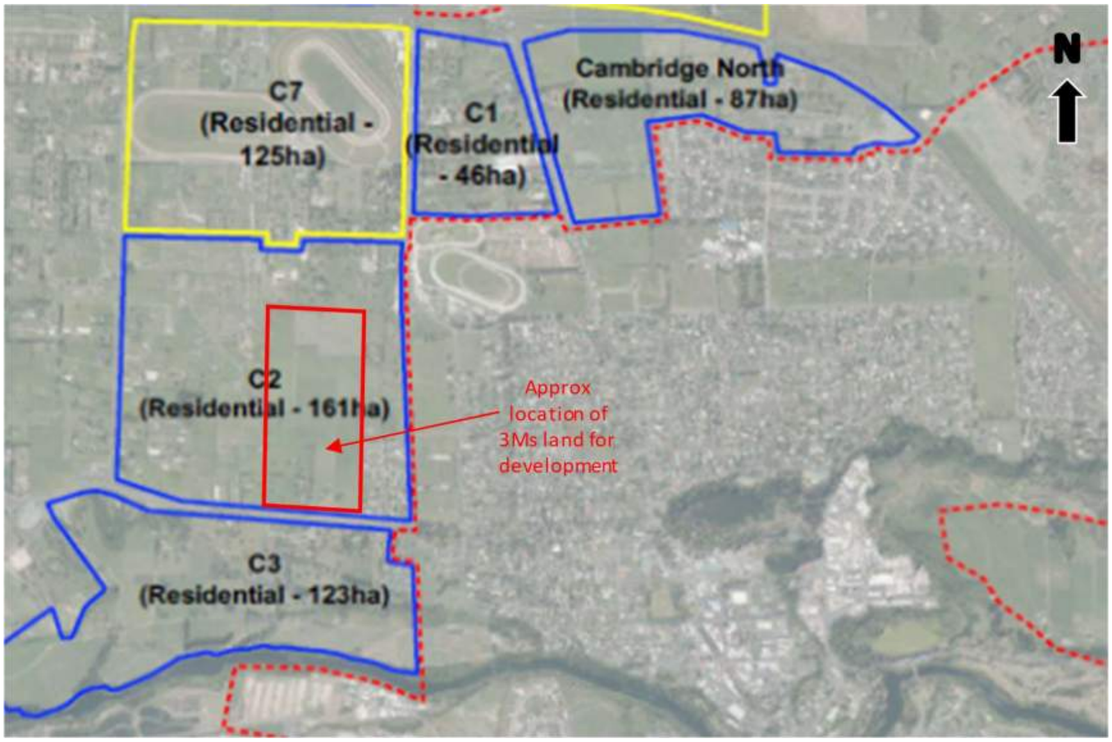


Figure 1: Excerpt from Appendix S1, Waipa District Plan (December 2019) showing planned growth cells for Cambridge and location of 3Ms land (subject of this report)

WDC are currently in the process of preparing consent applications for the major infrastructure in the C2 Growth Cell (herein referred to as “C2”).

3Ms of Cambridge Ltd (3Ms) have a landholding within C2 and would like to commence development this earthworks season, ahead of the current WDC programme. Accordingly, 3Ms have proposed a revised option, with staged development of the land to allow for residential subdivision works to be progressed ahead of WDC consenting.

To support this, 3Ms are seeking consents for the first stage of the development which includes:

- Waipa District Council (WDC) Subdivision for the entire development; and
- Waikato Regional Council (WRC) Construction Consents (entire site) including:
  - Clean fill importation;
  - Groundwater diversion (dewatering); and
  - Temporary water take.

Longer term, WDC are proposing to construct a central stormwater swale which will be used to collect and convey stormwater from C1 and C2 towards the Waikato River. The current Structure Plan also allows for some permanent soakage basins. However, as this long-term stormwater infrastructure will not be consented or constructed in time to support the initial development, 3Ms are proposing an interim stormwater solution which comprises a single stormwater soakage basin near the middle of the development (the “central stormwater reserve” shown in Figure 2).

3Ms will also partially construct the west-east component of the WDC stormwater swale which will essentially behave as forebays, providing pre-treatment and additional storage.

The soakage basin and forebays are all expected to extend below a perched near-surface groundwater level, hence the need for a dewatering consent.

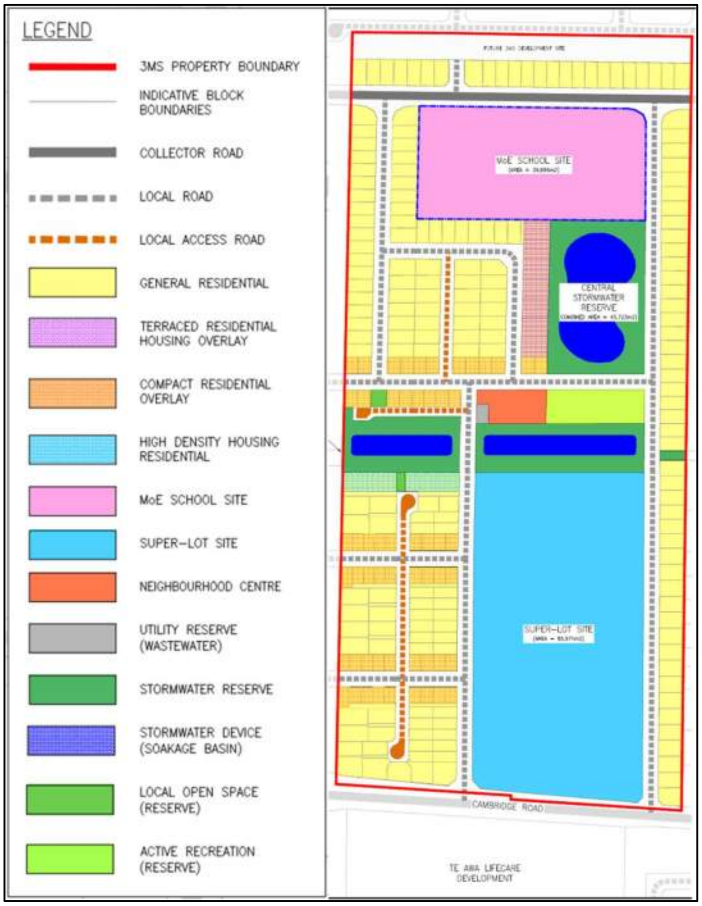


Figure 2: Excerpt from dwg 17001-SK-094 showing approximate development layout. Of relevance for this report is the stormwater reserve (in dark green) and deep excavations to form the basin and pre-treatment devices.

1.2 Scope of this report

Beca Ltd (Beca) have been engaged by 3Ms to undertake a dewatering assessment to support the regional consent application for groundwater take and diversion. This report presents the results of that assessment, specifically:

- An assessment of the potential for drawdown from the permanent diversion of a perched groundwater level into the stormwater reserve basin; and
- An assessment of the potential for adverse effects associated with the take and diversion of groundwater e.g. consolidation settlement, interference effects on shallow groundwater users or surface water bodies and the potential for the mobilisation of contaminants

The quantitative assessment of effects is based on a previously prepared 3D numerical groundwater model for WDC (Beca, 2019).

2 Proposed Works

2.1 Site Location and Description

A summary of key features pertinent to the assessment of effects and parties which might require assessment as potentially affected, is provided below. A description of groundwater conditions is provided in Section 3.

2.1.1 Topography

The 3Ms land parcel is located on a broad, relatively flat alluvial terrace at ~64 m RL.

To the south of Cambridge Road, the topography is marked by a series of four (4) distinct terraces at around 64 m RL, 58 m RL, 40 m to 47 m RL and 38 m RL.

The Waikato River sits approximately 20 m below the lowest terrace (at an estimated level of ~18 m RL, based on LiDAR) and is expected to control the regional groundwater level in the area.

2.1.2 Existing Land use and Buildings

The surrounding area is primarily rural with small properties (e.g. nurseries, lifestyle lots and farms) at low density, i.e. 3rd party owned buildings are generally relatively sparse, with the notable exceptions being:

- The eastern boundary of the 3Ms owned land, which borders the existing residential development on Kelly Road (namely 2 to 48 Kelly Road, and, 1891 to 1895 Cambridge Road are all located within 100 m of the boundary)
- 5 Hunter Lane and 59 Racecourse Road, located along the northern property boundary; and
- 694 Grasslands Drive Road and 1835A Cambridge Road located along the western property boundary.

2.1.3 Existing groundwater and surface water take

A review of the Waikato Regional Council (WRC) consent database undertaken in July 2020 indicated that there are no consented water takes within the 3Ms owned land.

There are also no consented groundwater takes within 1 km of the 3Ms owned land; however, a review of the borehole database suggests that there are several consented wells in the area. It is likely that some, if not all, of these wells will be taking groundwater as a Permitted Activity (in which case there will be no publicly available data but regardless the owners are legally entitled).

The nearest surface water takes are associated with the Waikato River, which is located at a much lower level and not considered to be directly connected to the shallow groundwater levels present on the upper river terraces and in the area of works.

2.1.4 Utilities

Only the current Town Boundary (to the east) has established, extensive infrastructure such as roads and utilities. There are no known piped utilities within the landholding itself however adjacent there is:

- Cambridge Road along the southern boundary;
- Grasslands Drive near the western boundary;
- Kelly Road near the eastern boundary;
- Water pipes along Cambridge Road; and
- Water, wastewater and stormwater lines in and servicing properties on Kelly Road.

2.1.5 Surface Water Bodies

Whilst there are several man-made drains and overland flow paths that run across the 3Ms property and in the surrounding area of Growth Cell 2, there are no naturalised streams.

The man-made drains were likely formed to deal with both surface run-off, but potentially also a shallow perched water level present in parts of this area (see Section 3 for further detail).

To south of Cambridge Road, there is a ~900 m length of unnamed stream which discharges to the Waikato River. It is likely that shallow groundwater contributes some component of baseflow to this stream; however, in the absence of flow gauging it is not possible to quantify the proportion.

2.2 Proposed works that might require a regional Groundwater Consent

The works which may require a groundwater consent include:

- Excavation, construction and operation of a central stormwater basin (up to 4 m deep).
  - The basin will be designed to have a soakage field beneath the invert level.
  - The basin will be designed to accommodate (soak and if necessary, store) a 100-year design event in the interim case. Once the full stormwater system is constructed the basin may only be used for soaking smaller events with overflow to the forebays (swales) or, may not be used at all, but would be kept for amenity purposes rather than infilled (i.e. the diversion is long term). We note this component of the activity is authorised by way of the consent held by WDC for the discharge of stormwater onto land, and to the Waikato River for the entire extent of the C1 and C2/C3 Growth Cells.
  - The basin will be unlined over most, it not all is depth and footprint and hence, where it is below the groundwater table, there will be a permanent groundwater diversion;
- Excavation of a stormwater swale / forebays (up to 4 m deep).
  - The swale will be unlined over most, if not all, of its depth and hence, where it is below the groundwater table, there will be a permanent groundwater diversion;

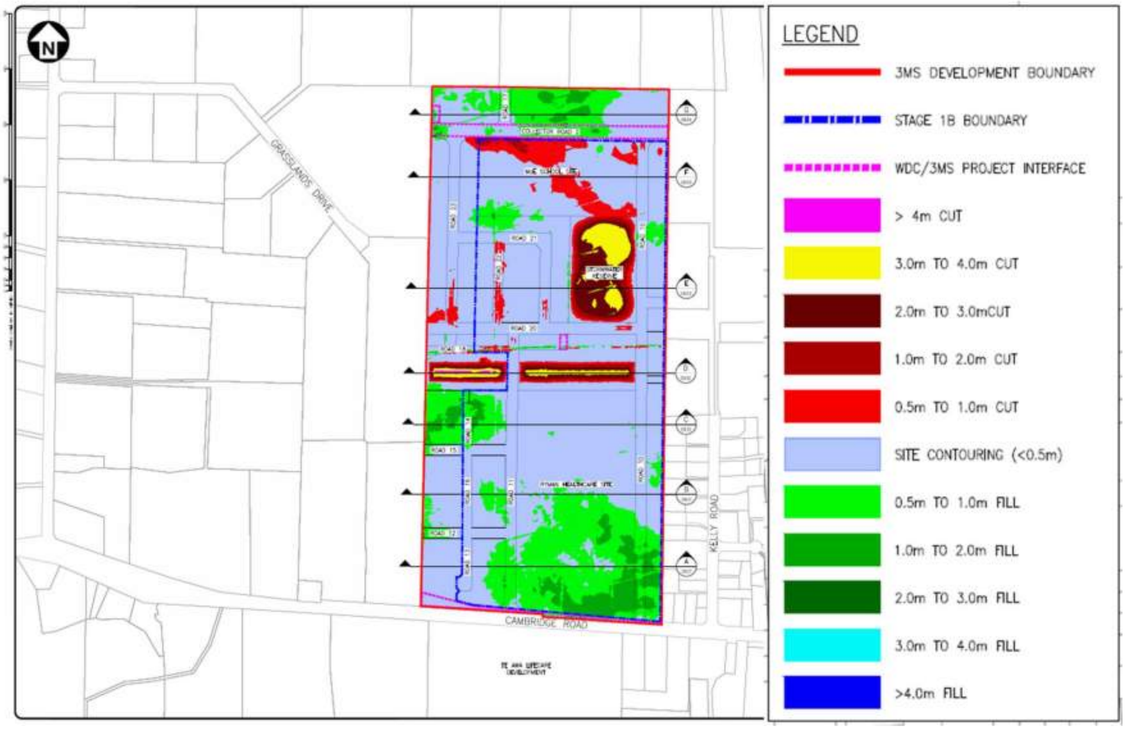


Figure 3: Excerpt from dwg 17001-C-0920 showing approximate final cut/ fill. Of relevance for this report is the stormwater reserve and deep excavations to form the basin and pre-treatment devices indicated in dark red and yellow.

As noted above the soakage component is already authorised by way of the consent held by WDC for the discharge of stormwater onto / into land, and to the Waikato River for the entire extent of the C1 and C2/C3 Growth Cells. Further, as the soakage will be discharging to an underlying aquifer it will not mitigate any drawdown effects of the upper perched level and so for simplicity is **not** included in this assessment

Short-term dewatering associated with the construction of stormwater culverts below proposed roads, water and wastewater networks is being considered separately by WDC.

Other works occurring in as part of Stage One e.g. filling, construction of collector roads etc. are not expected to influence groundwater.

3 Hydrogeological Setting

3.1 Conceptual Groundwater Model

Groundwater flow within the Waikato area is strongly influenced by the depositional history, which has created lateral and vertical variability in grain size (a mixture of pumiceous sand, silts, and gravels interbedded with clay/peats).

The Cambridge area is typically characterised by highly permeable coarse sand and gravels of the Hinuera Formation near the surface which are interlayered with lower permeability silty soils, creating a series of perched groundwater tables above the regional water table.

The regional groundwater table is expected to be controlled by the Waikato River, which based on LiDAR is at ~18 m RL. The groundwater level will rise with distance from the river but based on typical groundwater gradients would still be expected to be in the order of 20 to 30 m RL beneath the proposed areas of work. Whilst there is no specific monitoring of deep wells in this area to confirm this, this has been the observation from the Waikato Expressway – Hamilton Section project where there are multiple, nested piezometer arrangements in the vicinity of deeper incised rivers and streams.

Previous geotechnical and hydrogeological investigations in the Cambridge area have indicated at least three continuous perched aquifers may exist above the regional groundwater level. “Continuous” or extensive perched aquifers are inferred at between 55 m to 61 m RL, 45 m to 50 m RL and 35 m to 40 m RL (Figure 3).

Whilst there is not always a distinct low permeability horizon to explain the perching, it is likely the result of the alternating lower permeability (e.g. silts) and higher permeability layers (e.g. sands and gravels). Again, this is consistent with the groundwater system identified for Waikato Expressway – Hamilton Section.

Shallower, but small / discontinuous perched horizons are inferred (from select piezometers and CPT logs) near surface. The most notable of these discontinuous perched water tables is present through the centre of the 3Ms land along and extending towards Kelly Road, where there is evidence of an iron pan at approximately 2.5 m depth (approximately 61.5 m RL). Piezometers to the north and south of the 3Ms land have generally indicated only a deeper water level is present i.e. the perched iron pan aquifer is not considered to be laterally extensive in all directions. This is consistent with observations of groundwater level in hand augers, test pits and CTPs from more recent investigations (BTW, 2020) which confirms that the shallowest water level is typically deeper than 2 m bgl, and deepens to the north.

Recharge to the aquifer is via rainfall infiltration to the perched water tables, before slowly infiltrating into the deeper aquifers. The flow rate is likely to be very slow, both horizontally and vertically due to the flat topography and presence of lower permeability horizons.

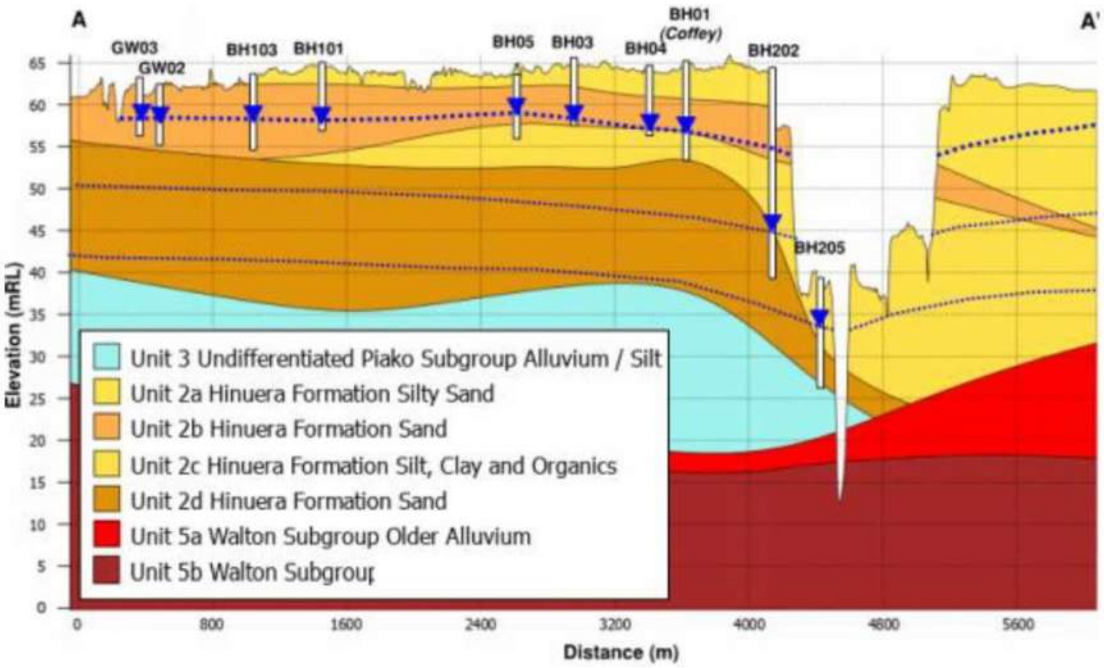


Figure 4: North-south cross section through the Cambridge C2 and C3 growth cells, including the 3Ms owned land. Steeply incised gully is the Waikato River which indicate the vertical exaggeration of the figure.

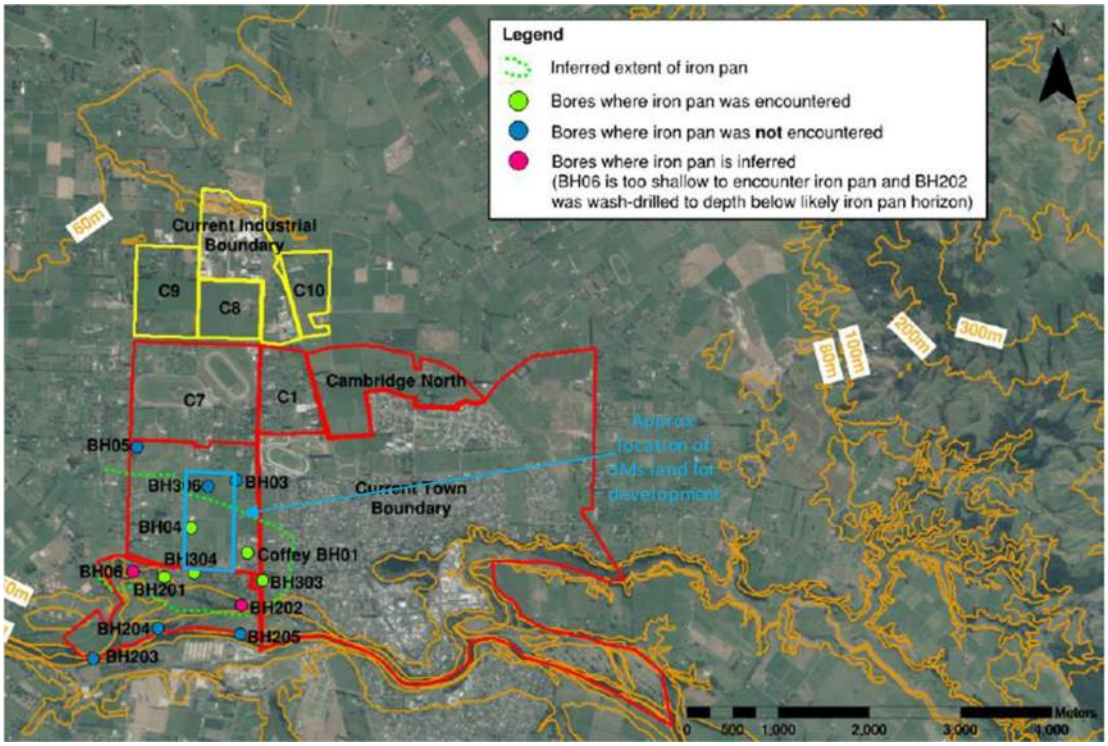


Figure 5: Inferred extent of iron pan and a possible perched aquifer.

3.2 Assumed Groundwater Level for Excavation

A series of site investigations have been previously undertaken in the broader C2 and C3 area to support structure planning.

There are six boreholes located in or close to the 3Ms landholding which are considered to provide the most pertinent information for this application. A summary of groundwater levels is provided in Table 1 (rows are approximately organised north (top of table) to south (bottom of table)). Refer Figure 5 for location.

Borehole logs and piezometer construction details are provided in Appendix A.

Table 1: Summary of available groundwater level data

Bore ID	**Screened Depth** m bgl [m RL]	**Duration of monitoring available**	**Lowest GWL** m bgl [m RL]	**Highest GWL** m bgl [m RL]	**Comments**
BH03	1.5 – 4.5 [60.5 – 63.5^{++}]	Jan'17 –May'20 (~1-year logger data)	Dry [-]	Dry [-]	Unsaturated to 58.8 m RL
	5.4 – 7.4 [57.6 – 59.6^{++}]	Jan'17 –Aug'17 (~7 months logger data)	7.3 [57.7^{++}]	6.2 [58.8^{++}]	
BH306	3.0 – 5.5 [58.5 – 61.0^{+}]	Mar'19 – May'20 (~1-year logger data)	5.1 [58.9^{+}]	3.1 [60.9^{+}]	
BH04	1.2 – 2.5 [61.5 – 62.8^{+}]	Jan'17 – Apr'20 (~3 years logger data)	2.2 [61.8^{+}]	0.5 [63.5^{+}]	Shallow perched on an Fe pan @ 61.5m RL
	4.5 – 7.5 [56.5 – 59.5^{+}]		Dry [-]	6.75 [57.2^{+}]	Unsaturated to 57.2 mRL
Coffey BH01	2.7 – 3.7 [60.3 – 61.3^{+}]	Single reading, September 2009	*no data*	0.9 [63.1^{+}]	Shallow perched on Fe pan
	5.7 – 6.5 [57.5 – 58.3^{+}]		*no data*	Dry [-]	Unsaturated to at least 56.3 mRL
	10.0 – 12.0 [52.0 – 54.0^{+}]		*no data*	7.7^{+} [56.3^{+}]	
BH304	3.6 – 6.6 [57.4 – 60.4^{+}]	Mar'19 – May'20 (~1-year logger data)	Dry [-]	Dry [-]	Unsaturated to 57.4 mRL
BH303	4.9 – 7.9 [57.6 – 59.6^{+++}]	Mar'19, Jun'20 (2x manual only)	Dry [-]	*no data*	Unsaturated to 57.6 mRL
	9.8 – 11.8 [57.6 – 59.6^{+++}]	Mar'19 – May'20 (~1-year logger data	9.1 [56.9^{+++}]	8.3 [57.6^{+++}]	

$^{+}$ Based on assumed ground level of 64 mRL

$^{++}$ Based on assumed ground level of 65 mRL

$^{+++}$ Based on assumed ground level of 66 mRL

🟦 Indicates a piezometer screened in the upper 3 m of the soil profile

Review of the above data and logs indicates that there is a ***shallow*** perched groundwater level present through the middle part of the 3Ms land, potentially extending as far as Kelly Road likely sitting on an iron pan at around 2.5 m depth. This shallow perched aquifer has a winter high groundwater level of ~63 - 63.5 m

RL, but near the centre of the 3Ms land may only be saturated over a thickness of a few hundred mm during summer months. This is based on the shallow piezometer at BH04 which has a summer low level of 61.8 m (just 0.3 m above the piezometer base / iron pan) and the adjacent deeper piezometer which is unsaturated to at least 7.5 m depth in summer (Figure 6). This is

- Similar to the pattern reported by Coffey (2009) for a multi-level piezometer BH01 in Kelly Road; and is
- Consistent with other piezometers BH303 and BH304 which indicate dry conditions over a depth range of 3.6 to 7.9 m bgl; and is
- Consistent with anecdotal reports from the previous landowners that deep excavations are dry.

A schematic of this shallowest perched level and underlying unsaturated ("dry") zone is provided in Figure 6. The deeper, though likely still perched groundwater water level is reported between 56 mRL to 61 mRL (note that it is this deeper, more consistent groundwater level that is shown in Figure 4).

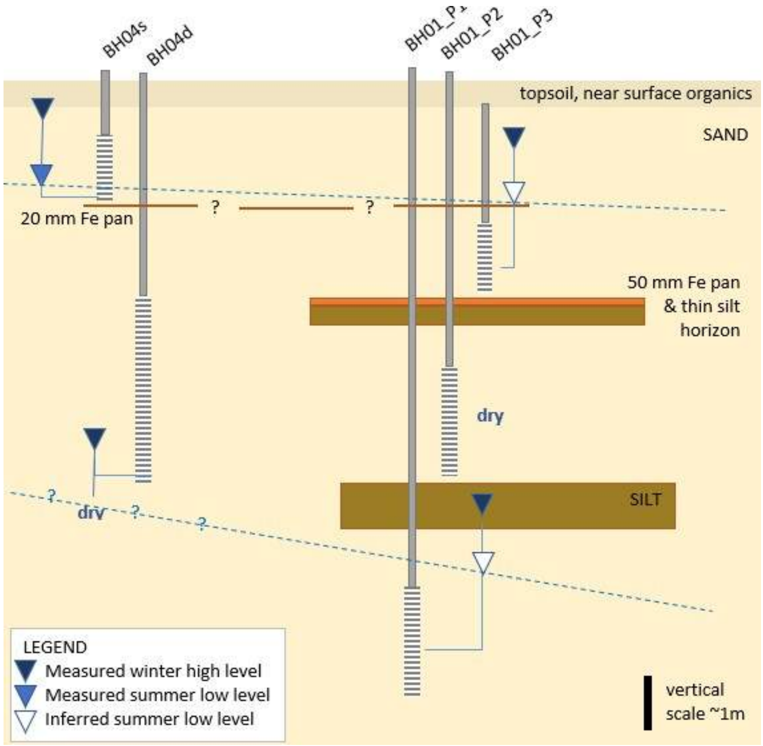


Figure 6: Schematic of iron pan and a possible perched aquifer extending from BH04 to Kelley Road.

Whilst there appears to be a correlation between the centre of the 3Ms site and Kelly Road, we note that a shallow piezometer (BH03) near the northern boundary is dry, suggesting that this shallowest perched level is not extensive over the wider area. Similarly, shallow piezometers along Cambridge Road are dry and bore logs suggest that any iron pan, where present, is much deeper, and, the recent BTW (2020) investigations suggest that any shallow perched aquifer also deepens to the north.

Hence the proposed 4 m deep excavations for stormwater basin and forebays are likely to puncture through the shallowest perched level and discharge into the unsaturated zone below.

Based on the above information a summer water level for the shallowest perched aquifer of 2.2 m bgl (61.8 mRL) has been adopted. This groundwater level is expected to deepen to the east towards Kelly Road and based on the conceptual section above, the saturated thickness of the perched aquifer over summer might be < 0.5 m at BH04 but increasing to 2 m at Kelly Road.

4 Assessment of Groundwater Drawdown

4.1 Existing numerical groundwater model

A steady state numerical groundwater model was developed in the three-dimensional (3D) finite-difference modelling package Groundwater Vistas. A full description of the model (design and construction boundary conditions and calibration, extracted from Beca, 2019) is provided in Appendix B and for brevity is not repeated here.

The original model was developed for WDC (Beca, 2019) to provide a high-level overview of the potential viability and effects arising from the broader Growth Cells development; particularly, the long term swale operation, reduced perviousness at the surface and the beneficial use of soakage to partially offset any adverse effects. We note that such works / effects are not the subject of this current application. However, the underlying model remains applicable and has been modified, where necessary, to specifically allow for the assessment of the 3Ms soakage basin.

Because of the greater depth of the proposed WDC central swale system and in order to consider soakage to the underlying more widespread aquifer below 58 mRL, the model was specifically calibrated to this more extensive and deeper water table. As this water level is > 4 m bgl, inclusion of the proposed 3Ms basin excavation in the model would result in no drawdown.

Where the perched level on the iron pan is not extensive than it may be that there is no drawdown, or that drawdown is limited to the 3Ms land only. However, in order to test an upper bound extent of drawdown assuming an extensive shallow aquifer at 2 m bgl, the basin has been artificially deepened in the model to achieve a maximum drawdown of 2 m.

The below sub-sections set out key changes which have been made to this model for the current assessment and the key commentary regarding the model calibration and remaining uncertainty or limitations

4.2 Changes made to model for this assessment

The following changes have been made to the groundwater model:

- The mesh was refined (from 20 m x 20 m cells to 10 m x 10 m cells) to allow for a more discretised simulation of the proposed works; and
- The extent of No-Flow cells south of Cambridge Road was reduced (as part of a separate package of work to WDC) to eventually allow for assessment of a proposed road cut through this area. This required moving the DRAIN boundary condition (assigned to the model's southern extent) further south;

No changes have been made to hydraulic conductivity, rainfall recharge, surface water bodies or calibration targets. The changes were first added to the existing steady state model and re-run to confirm that they did not result in any changes to the overall model calibration.

4.3 Model calibration

There are 90 groundwater level observations available for calibration which were weighted to reflect the data quality (refer **Appendix B**). The calibration was undertaken in steady state i.e. reflecting long term average groundwater conditions. The scaled RMS error based on the groundwater levels is 4.3%, for a weighted calibration of the 90 data points.

Most of the calculated heads are within 1 m of the observed; however, there are three high confidence level targets, not within the area of works considered here, which have residuals indicating the calculated heads are either greater than or less than the observed by between 1 m and 2 m but overall, the model is considered to suitably simulate the overall direction and flow of groundwater in the deeper, extensive perched aquifer.

For assessing the potential maximum drawdown of the shallower, perched iron pan groundwater level we have assumed a similar gradient and overall flow direction (towards the river) exists. As outlined in Section 3, the perched level may in fact be less extensive and likely dips towards the south and is non-existent to the north and hence assuming the same, extensive flat gradient as in the underlying aquifer is expected to overestimate the extent of drawdown. The results provide an upper bound for assessing effects and identifying potential mitigation measures, in the unlikely event they are needed.

4.4 Residual uncertainty / limitations

The primary limitation of the groundwater model is that it cannot simulate the multiple perched groundwater levels at the site (Figures 4 and 6). This is due to numerical difficulties in simulating and converging alternating saturated and un-saturated conditions; however, this a limitation of most professional applications of 3D software.

The implications for the model and subsequent assessment of effects, is that the model assumes a fully saturated profile from the calibrated groundwater level down to the base of the model. Based on deeper bores in the area and discussions with the landowners during previous drilling campaigns, it is generally expected that there is a significant unsaturated zone with depth i.e. the perched aquifers are of limited thickness.

Similarly, the model cannot simulate the discrete perched aquifers near surface (without the introduction of very high recharge rates and / or very low permeability layers, which would again make the model numerically unstable). Whilst these perched horizons are expected to be discrete over most areas, there is a more extensive perched aquifer overlying the iron pan in the 3Ms land.

There is no stream flow data for the unnamed stream or for the discharge of springs in the terraces south of Cambridge Road; hence the mass balance, in terms of the calculated absolute values of baseflow to these features must be viewed with some caution. However, the model can be used to provide an indication of the proportionate change in groundwater contribution to the surface water features.

5 Assessment of Effects

5.1 Changes in Groundwater Level

The stormwater basin and forebay / swale excavations immediately south, herein collectively referred to as “the excavations” will be excavated to a maximum depth of approximately 4 m. The shallowest ***continuous*** perched aquifer is expected to be some 6 m to 8 m below ground level, and hence the excavations are well above this and no drawdown or groundwater inflows from this aquifer are expected.

The shallower **perched iron pan aquifer** is inferred from two borehole locations and is not expected to be laterally continuous in all directions, but as the actual extent is not fully known it is considered prudent to consider the potential effects if it is extensive. To account for this the model simulation has considered an artificially deep excavation to simulate the potential drawdown effect of excavations up to 2 m below a perched aquifer (i.e. the excavations were artificially lowered until they were a maximum depth of 2 m below the saturated water table considered in the model).

The drawdown results for this upper bound case (Figure 7 and Figure 8) indicate a maximum drawdown of 2 m immediately adjacent the stormwater basin (where groundwater level is closest to the surface), reducing to generally less than 1.25 m at the site boundaries. Due to the high permeability of the soils, measurable drawdown (taken as 0.25 m) under this upper bound case, extends some distance, ranging 800 m to 1400 m from the excavations.

This drawdown estimate is likely to be overly conservative as the simulation indicates drawdown occurring beyond the iron pan extent, which is unlikely. Figure 7 and Figure 8 show the drawdown contours in the areas outside the known iron pan extent as dashed to note the much lower risk of measurable drawdown eventuating in these areas.

This model is considered to provide an upper bound of effects as it assumes an aquifer of effectively infinite vertical and lateral extent, whereas:

- The borehole logs and groundwater level monitoring indicate a perched water table (above the iron pan) of discrete extent, limited to the 3Ms site and extending towards Kelly Road only; and
- Based on the geology encountered at BH04 (western boundary of 3Ms land), the saturated thickness of the perched aquifer is likely to be only 1.5 m i.e. the swale invert breaks through the base and into an underlying unsaturated zone such that the maximum drawdown would on average only be 1.5 m, and in summer might be less than 0.3 m and hence the consolidation risk is significantly lowered.
- The model assumes the perched iron pan aquifer has a consistent water table depth of ~2 m, when in fact there is evidence that the iron pan lowers to 2.5 m depth (~61.5 m RL) towards Kelly Road and towards Cambridge Road (where present) it is even deeper, well below the swale level.

Some risk remains that drawdown from the swale may result in permanent discharge of the perched iron pan aquifer which would be beneficial in terms of the liquefaction risk for the development. However, it could result in consolidation settlement of any compressible near-surface soils in the area (see Section 5.2).

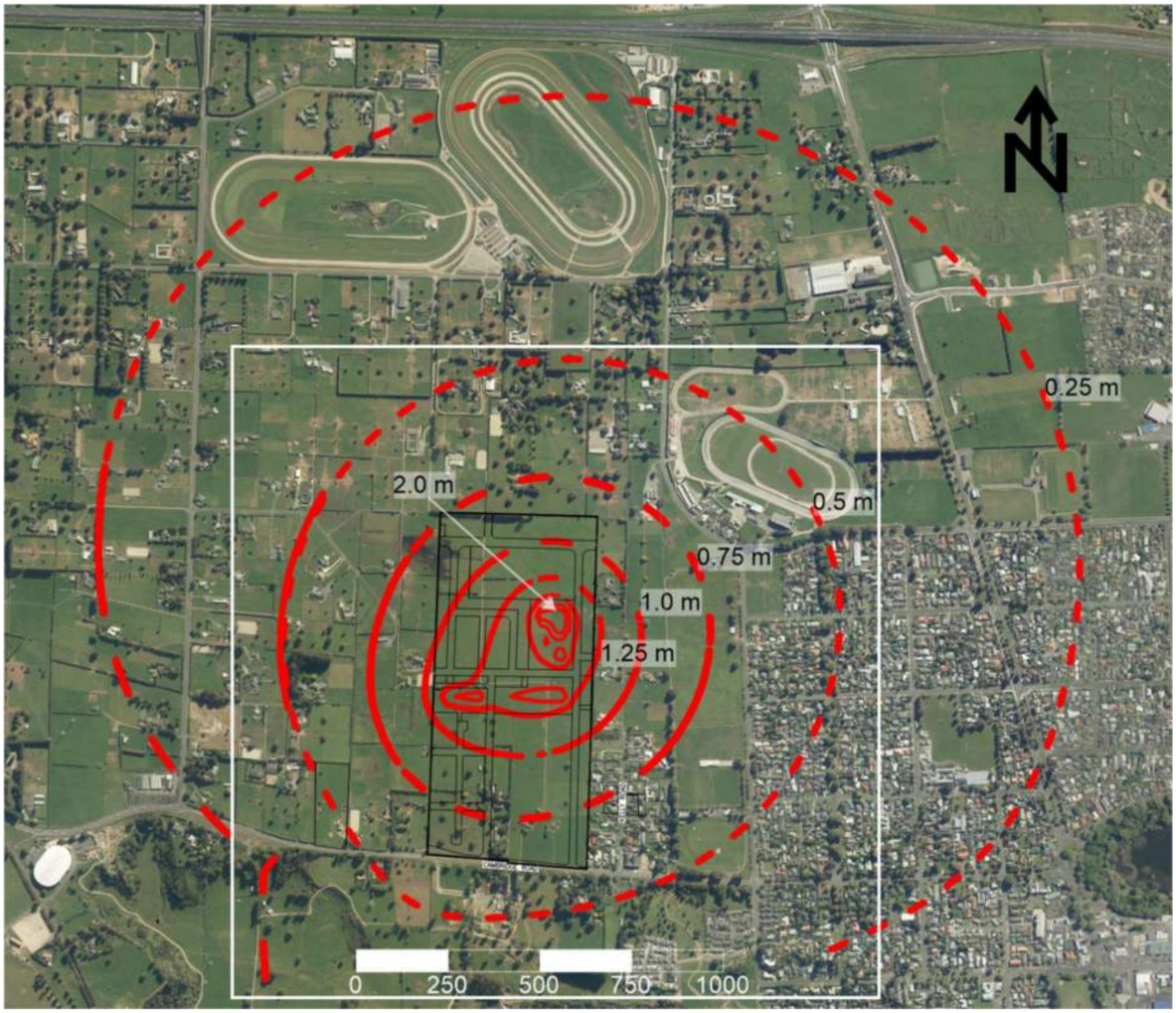


Figure 7: Full extent of calculated drawdown contours (0.25 m intervals) from long term operation of soakage basin and stormwater swales. Drawdown contours are dashed beyond the known extent of the iron pan to note the much lower risk of measurable drawdown eventuating in these areas. Excavations demarcated by blue rectangular polygons in Figure 8 (extent of Figure 8 indicated by white rectangle).

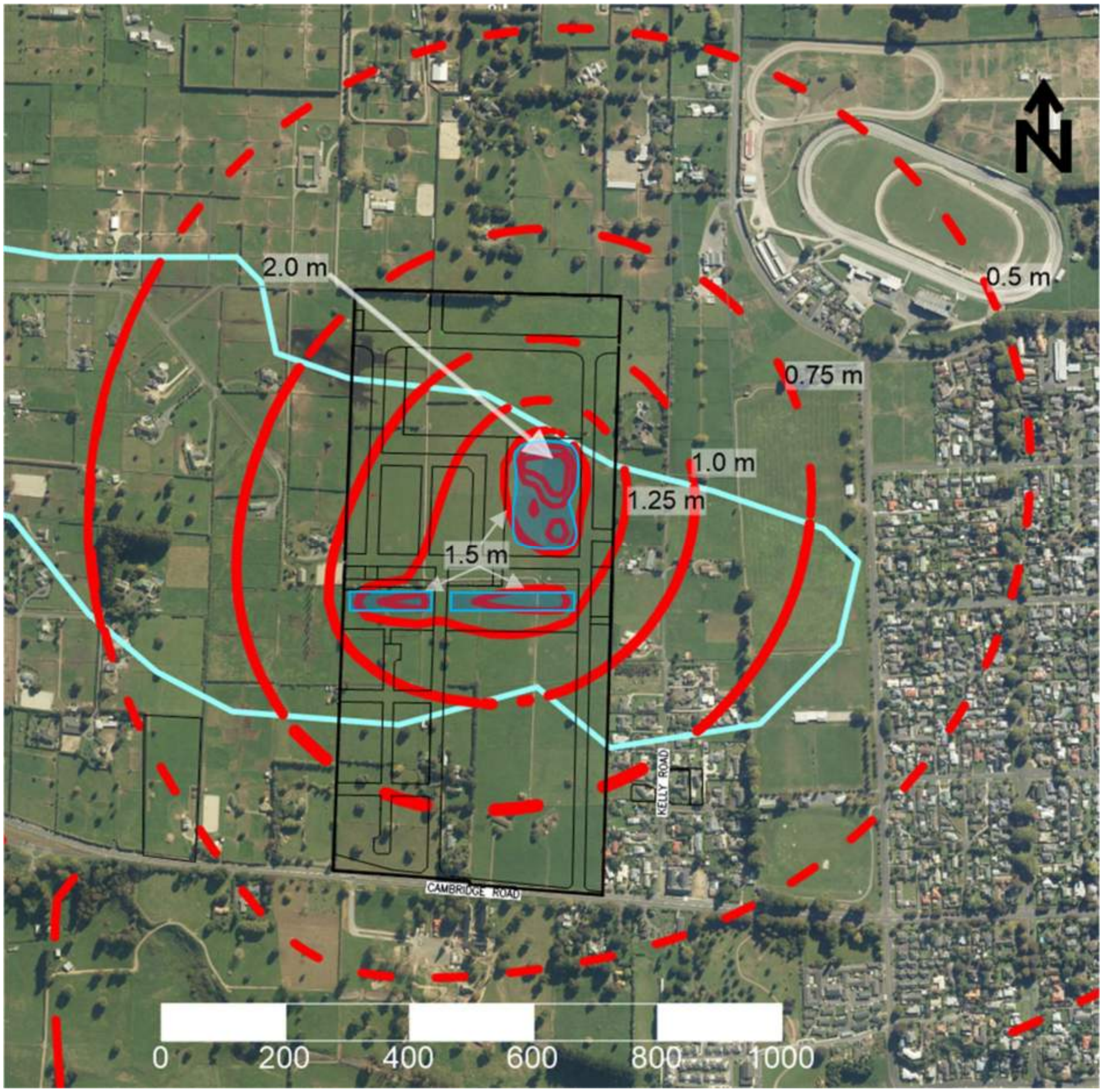


Figure 8: Close up of calculated drawdown contours (0.25 m intervals) from long term operation of soakage basin and stormwater swales. Drawdown contours are dashed beyond the known extent of the iron pan (outlined in light blue and consistent with areas identified by BTW as having a deeper, if any perched level) to note the much lower risk of measurable drawdown eventuating in these areas. Excavations demarcated by blue rectangular polygons.

5.2 Potential for Consolidation Settlement as a Result of Drawdown

Where the groundwater level in silty and clayey soils is drawn down below the naturally occurring low groundwater level there is the potential for some longer-term consolidation settlement of the ground to occur. Depending on the nature of the soils, such consolidation can continue for many years after construction is completed.

Given the relatively large hydraulic conductivities of the sandier soils (in which most drawdown will occur) and assuming that the basin is constructed in advance of subsequent development, it is likely that a significant proportion of drawdown will have occurred and the groundwater level largely stabilised, before any other private or public development of the site begins.

Generally, sandy soils (that are much less susceptible to settlement) dominate the area, and where silty and clayey soils occur, they tend to be thin interbeds within the sandier materials. As the thickness of the layer to consolidate is relatively small, settlement could also be expected to occur relatively quickly. There remains some risk, that if drawdown is transmitted along more sandy horizons, it could be felt by compressible layers at distance resulting in some settlement further away over time; however, as set out in the following sections, the settlement is likely to be very small and unlikely to result in damage.

The linear-based method of settlement calculation has been used to provide a quick and conservative estimate of potential settlement in the following sections, using an assessed coefficient of volume compressibility (m_v) and the calculated drawdowns relative to expected historical low water levels (i.e. summer conditions).

Settlement has been calculated based on:

$$S = m_V \times \Delta P \times t$$

Where S = settlement (mm);

m_V = volume compressibility with dimensions m^2/MN (a range of values between 0.1 and 0.5 was tested);

ΔP = change in pressure (maximum drawdown at base of layer in metres, converted to kN/m^2; and

t = thickness of compressible layers (values of 1 m and 2 m were tested in line with ground models developed by Beca, 2019 based on geotechnical testing and consistent with maximum saturated thickness of the aquifer).

Immediately adjacent to the excavations, where drawdowns of up to 2 m are calculated, a maximum settlement of up to 20 mm could occur (assuming a 2 m thick compressible layer with m_v = 0.5 m^2/MN). This is an upper-bound case assuming the maximum thickness and a high soil compressibility i.e. assumes 2 m of soft silts or clays. Further, such settlement even should it occur is likely to be limited to the area immediately surrounding the basin, adjacent road and green corridors; consequently, it is wholly limited to the development site boundaries.

A maximum drawdown of at the site boundary of 1.25 m is generally calculated. In the unlikely event that such drawdown did occur, this could result in 6 to 12 mm of consolidation settlement (upper bound assuming mv = 0.5 m^2/MN and 1 m to 2 m compressible thickness). This level of total settlement is unlikely to result in any damage to buildings or structures, and as the drawdown and settlement are likely to occur over a broad area with a relatively flat gradient, the risk of differential settlement (which has the potential to do the greatest damage) is considered to be low. However, the upper bound at 12 mm is slightly greater than usual first pass filters for risk of damage (being 10 mm) and so some further investigation and potentially some limited monitoring is recommended at the site boundary to better constrain and track any risk.

In the area of existing (denser) residential buildings, to the east of the 3Ms land on Kelly Road and along Cambridge Road, and in the vicinity of the race track, drawdown is calculated to be less than 1.0 m and expected to result in less than 10 mm of consolidation settlement. In these areas, the potential for damage to buildings or services as a result of the settlement is considered negligible.

Monitoring of groundwater levels before, during and after construction of the basin would be prudent to confirm that the actual drawdowns are within the range described above; particularly, in areas where any private development precedes the excavations. As noted above this should include a series of shallow monitoring wells in the areas of existing buildings to confirm if there is an extensive perched aquifer, in which case some limited survey monitoring may also be warranted (Section 6).

5.3 Groundwater Inflows to Excavations

Assuming a laterally extensive perched iron pan aquifer, modelling suggests that long term groundwater inflows to the basin would be of the order of ~320 m^3/d.

In practice this would not be pumped out of the ground in the long term, but rather would be discharged to the underlying aquifer. Some “take” might occur during initial excavation phases before the iron pan is broken through.

Again, we note that in this scenario the model adopts an aquifer of effectively infinite extent that may yield more groundwater than is realistic long term (i.e. the perched aquifer may be more finite in extent or potentially ephemeral); hence, this estimate is conservative.

5.4 Potential for Impacts on Existing Groundwater Users

As discussed in Section 2.1.3, there are no consented groundwater takes within 1 km of the proposed works; however, a review of the borehole database suggests that there are a number of groundwater wells within and surrounding the Growth area. It is likely that some, if not all, of these wells will be taking groundwater as a Permitted Activity (in which case there will be no publicly available data but regardless the owner is legally entitled).

In the upper bound scenario where a perched iron pan aquifer results in drawdown beyond the development area, results of modelling suggest some of these wells may experience up to 0.75 m of drawdown (Figure 9). However, the bores closest to the site are generally deeper than 20 m, and hence, are expected to be abstracting water from one of the much deeper aquifers, and are therefore, unlikely to be affected by drawdown from the proposed works.

The nearest shallowest bores likely to be taking water (2 No. screened between ~6 m depth and ~14 m depth) are shown within the calculated 0.5 to 0.75 m drawdown contours, however all bores are located within areas where available data suggests the iron pan aquifer is not present. Hence actual drawdown is expected to be much less.

Regardless, we note that the wells are shallow and will have limited available drawdown. They may already be subject to seasonal “drying” and in the unlikely event that drawdown at the upper bound presented here does occur, there is some small risk that this may result in noticeable additional pumping effort or a change in the ability of those users to abstract groundwater (if not monitoring bores and if being used for groundwater abstraction). A summary of the potential effects on all shallow bores within the zone of > 0.5 m drawdown in provided in Table 2.

It is recommended that the consent holder visit these bores prior to works commencing to confirm if they are operational and record the nature of pumping equipment, groundwater level etc., if possible. If it is determined that they could be affected, then it would be prudent to undertake some monitoring of levels either in these bores (where access allows) or via a purpose-built shallow monitoring well nearby.

If these wells do experience any interruption to their supply, then mitigation measures could include either to supplement the owner’s supply up to the permitted rate of 15 m^3 a day (where the effect is short term) or deepen the pump or well (for a long term effect).

Overall, any effects on private well users are likely to be less than minor and can be managed through the monitoring and mitigation measures outlined in Section 6.

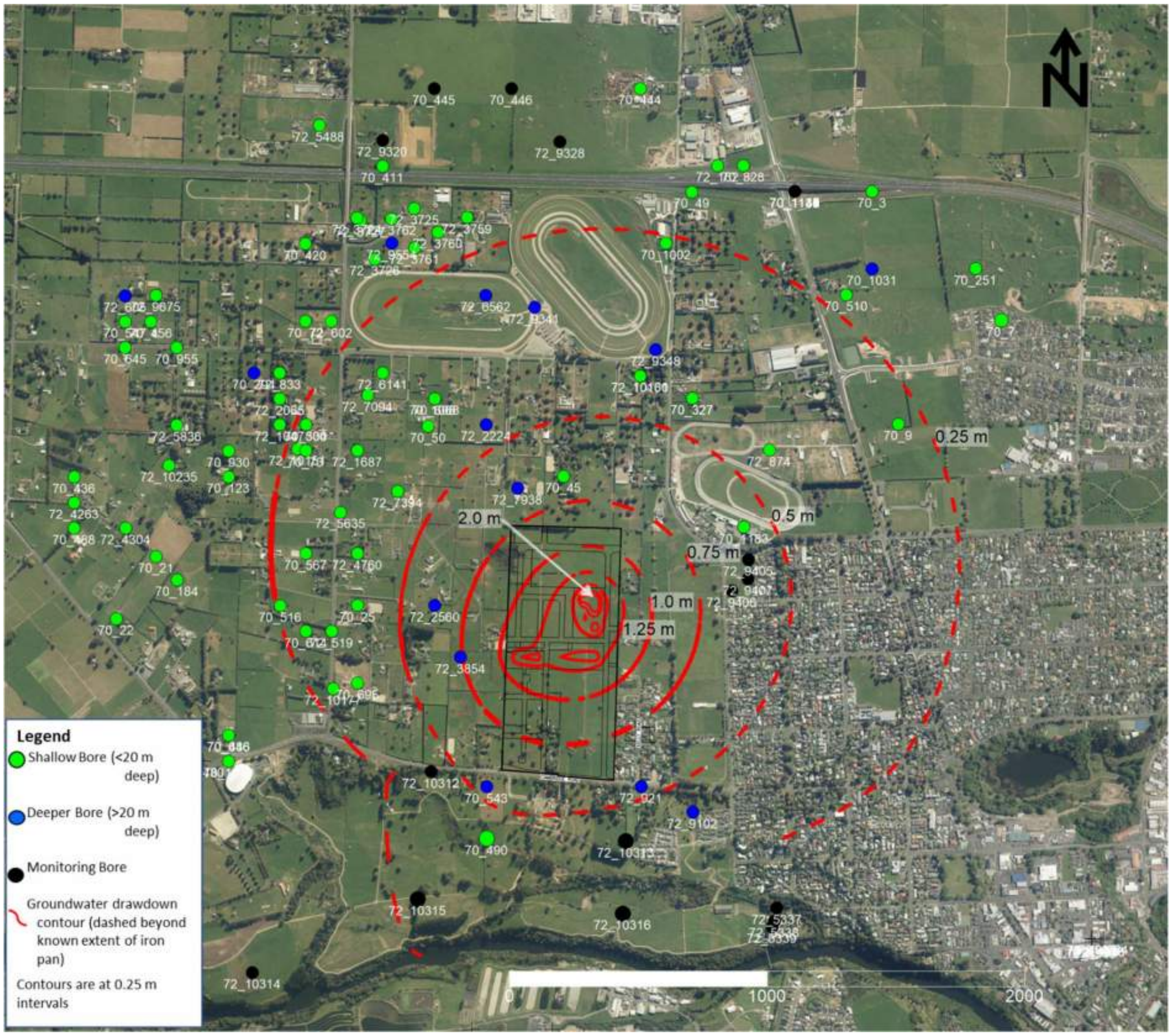


Figure 9: Groundwater Bores on WRC Database within 1 km of predicted extent of drawdown effects from long term dewatering by excavations – upper bound **artificially deep swale scenario**. Drawdown contours are dashed beyond the known extent of the iron pan to note uncertainty / lower risk of drawdown.

Table 2: Expected upper bound drawdown at neighbouring shallow private wells

WRC ID	Easting (NZTM)	Northing (NZTM)	Maximum Calculated Drawdown (m)	Comments
70_45	1815376	5804790	0.75	The well is approximately 450 m from the basin and WRC records report this well to be 7.88 m deep. Monitoring of BH03 at the north-western corner of Consenting Zone 4 between December 2016 and July 2018 recorded groundwater levels between 6 m and 7.27 m below ground level in the deep piezometer (screened depth of 5.40 m - 7.40 m) and absent from the shallow piezometer (screened depth of 1.55 m - 4.55 m). This suggests that seasonal periods of little to no groundwater (i.e. <0.5 m) may already occur and that there is little drawdown available should the maximum predicted drawdown occur. Should this well be in use, there is some risk that permanent dewatering may impact supply. There is no known take consent associated with this well and WRC report the casing is 903 mm in diameter which suggests it is a historical bore and may no longer be used. A site visit should be made by the consent holder to confirm, if possible.
70_1183	1816077	5804591	0.6	The well is approximately 600 m from the basin and WRC reports this well to be 14.3 m deep and screened between 10.9 m and 14.3 m depth. Well 70_1183 is expected to be abstracting water from a deeper aquifer and is unlikely to be affected by drawdown from the proposed works. There is no known take consent associated with this well however this well likely remains in use at its Cambridge Raceway location. A site visit should be made by the consent holder to confirm, if possible.
72_9405	1816094	5804466	0.6	These wells are between approximately 490 m to 570 m from the basin and WRC reports all wells to be 6 m deep and screened between 3.5 m and 6 m depth. Based on the reported construction details (50 mm diameter PVC casing installed December 2016), identification of this site as a historical refuse fill site (Section 5.6), and subsequent subdivision of this parcel based on WRC GIS, it is likely these are groundwater monitoring wells for geotechnical and/or contaminated site investigation purposes. A site visit should be made by the consent holder to confirm, if possible.
72_9407	1816093	5804391	0.6	
72_9406	1816030	5804345	0.7	

5.5 Effects on Surface Water Bodies

As mentioned in Section 2.1.5, whilst there are several man-made drains and overland flow paths in the surrounding area, the nearest naturalised stream is to the south of Cambridge Road.

As described earlier, the model mass balance can be used to consider proportionate changes in the volume of groundwater which is flowing towards the unnamed stream. The model simulation suggests that the diversion of groundwater from the perched iron pan aquifer towards the permanent basin, might result in a 5% reduction in the volume of groundwater flow which discharges to the stream.

Whilst there is no existing stream monitoring to quantify the range of naturally occurring flows, a 5% change in groundwater contribution would likely be unnoticeable against the background range of flows. Further the take itself is not consumptive in the sense that it is not removed from the system, but rather will discharge into the underlying aquifer and still report to the stream i.e. there is likely to be no net change.

Overall, any effects on surface water bodies are expected to be less than minor.

5.6 Potential for Contaminant Migration or Mobilisation

Contaminated sites (or sites appearing on the hazardous activities and industries list, HAIL) recorded in the WRC database and located within 2 km of the proposed works are shown on Figure 10. The contaminated sites are generally located in the rural industrial area to the west and pastural area to the southeast (both of which are hydraulically down gradient from the basin) and are typically described as areas of pesticide bulk storage or use.

The only known contaminated sites hydraulically up-gradient of the proposed works are:

- A historical refuse fill site, located 470 m from the proposed works, that from aerial imagery appears to have been capped between 1967 and 1971 (RetroLens); and
- A historical sawmill located 760 m from the proposed works, (which is noted on WRC record as ‘entered in error’ and which is absent in aerial images).
- An agricultural / industrial retail site located greater than 1 km from the proposed works (fertiliser manufacture or bulk storage).

Overall, the risk of contaminant migration or mobilisation as a result of the basin is considered low.

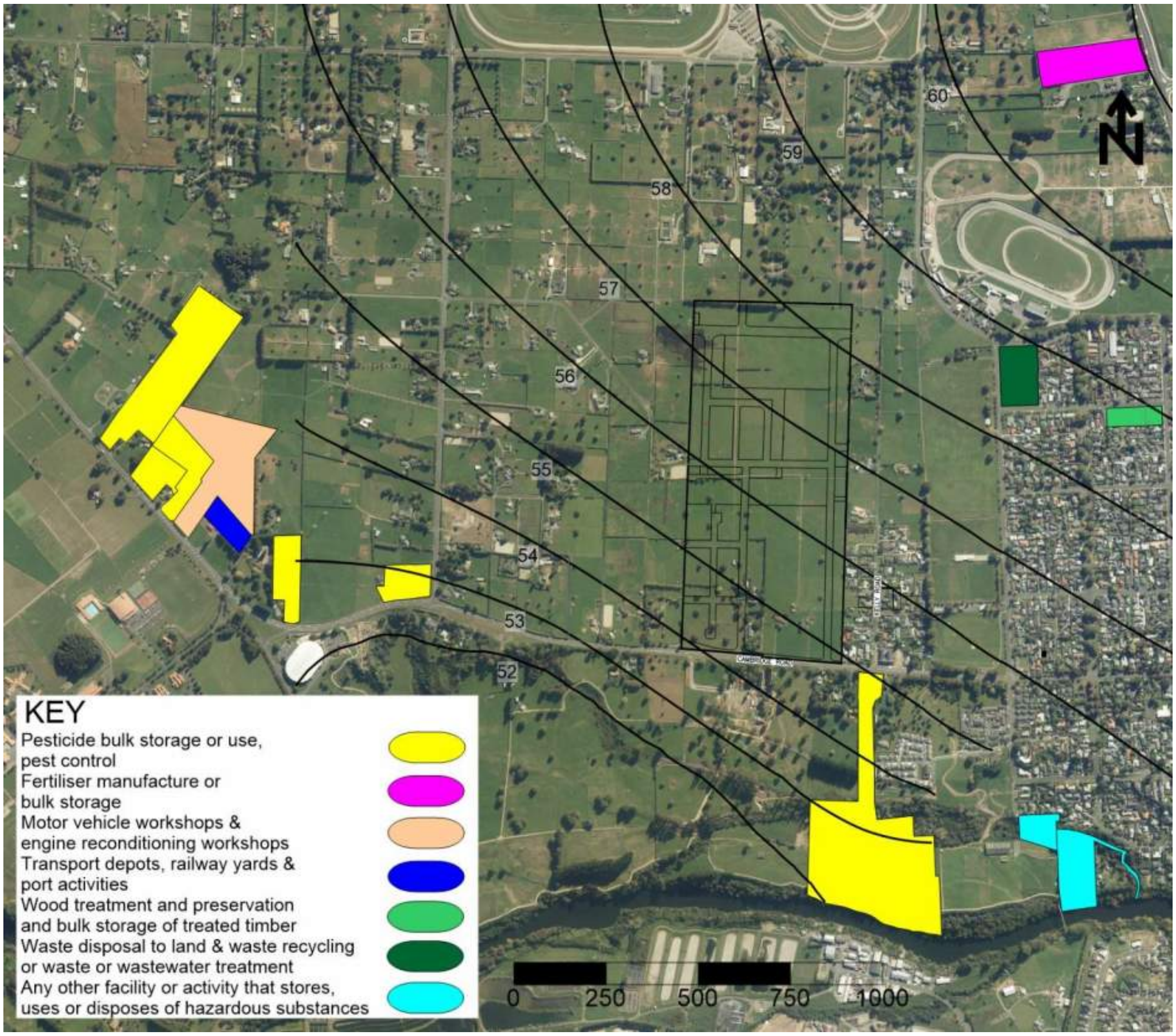


Figure 10: Contaminated sites (or sites appearing on Hazardous Activities and Industries List) within 2 km of proposed works (Source WRC, November 2020). Modelled base case average groundwater level contours are shown in black.

5.7 Basin Mounding Assessment

Whilst the operation of the soakage basin itself is a provided for, an initial assessment of the operation has been undertaken to confirm basin sizing and risk of mounding under expected operational conditions.

The assessment used the Hantush (1967) solution for the growth and decay of groundwater mounds in response to uniform infiltration over a given time period.

A full description of the key assumptions and limitations, as well as calculation inputs is provided in Appendix C. However, the most critical assumptions / limitations are:

- The **site-specific design infiltration rate** which has not yet been confirmed. A design infiltration rate of 100 mm/hour has been adopted. This is comparable to the factored rate adopted for the BIL site in Hautapu recently and is lower than the average rate reported in the SMP (WDC, 2019) but we note that this was based on the average of a range of different test methods and locations. Site specific testing at the basin location is required to confirm the rate; and
- The Hantush solution is a simplified method which **determines the relative height of mounding below a basin but cannot account for any stored volume in the basin itself**. This is significant as the stored volume in the central stormwater basin is calculated to be ~30,000 m^3 with an additional 19,600 m^3 in the

forebays / swales (Harrison Grierson, 2020). This more than exceeds the total runoff volume of the smaller design rainfall events, but also would be sufficient to fully store the 100-year event, such that even if the mound height rises to the invert level (IL) of the basin, any additional stormwater volume is readily accommodated in the basin and the mound height will be less than calculated (Figure 9).

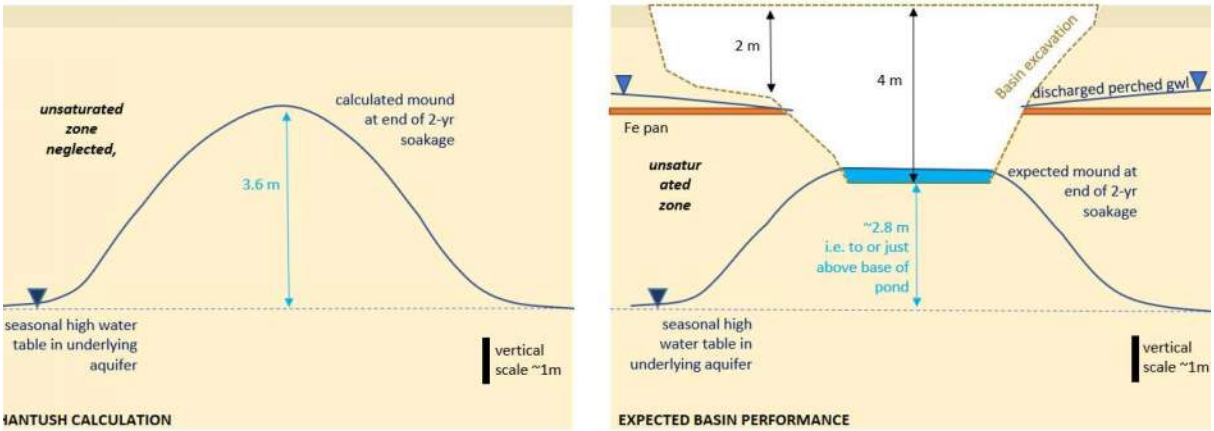


Figure 11: Schematic of Hantush calculation vs expected basin operation

The calculations suggest that any short-term mounding associated with the 2-year and 10-year design events can be readily accommodated within the unsaturated zone and by storage within the basin, such that the groundwater level adjacent the basin remains more than 2 m below the ground surface.

For a larger 100-year event the groundwater mound will rise to a higher level within the basin but will remain fully contained i.e. no surface breakout. In the unlikely event that the water level in the basin reaches the overflow level, then the swales / forebays provide additional storage to allow a slower release of soakage to ground.

We note that the assessment is not based on site specific testing over the footprint of the basin. As noted earlier there is some variability across the area and site-specific testing of the saturated hydraulic conductivity in the lower aquifer is critical to confirming the assumptions presented above. This should comprise at least two tests over the footprint. Testing should be a 4-hour duration constant rate test in a piezometer screened below the deeper groundwater level, or, if undertaken in the unsaturated zone should include a suitable period of pre-soak or multiple (repeat) tests until steady state conditions are reached.

6 Monitoring

Groundwater level monitoring is recommended to confirm that the magnitude and extent of any groundwater drawdown which does occur, is within the upper bound assessment presented in this report. It is anticipated that monitoring would be formalised in a Groundwater Monitoring and Mitigation Plan, that would be submitted to WRC ahead of the start of dewatering.

6.1 Monitoring of Groundwater Drawdown

Drawdown will result in a depression of the groundwater level that will extend outwards from the activity, declining in magnitude with distance from that activity. Drawdown as a result of the long-term diversion of groundwater into the basin will be permanent and as there are two private shallow wells, and some existing residential buildings within the calculated upper bound zone of influence, some limited monitoring is prudent.

It is proposed that groundwater monitoring of purpose-built monitoring wells be carried out to provide early warning if drawdown occurs and / or is likely to extend beyond the site. Proposed monitoring is shown in Figure 12.

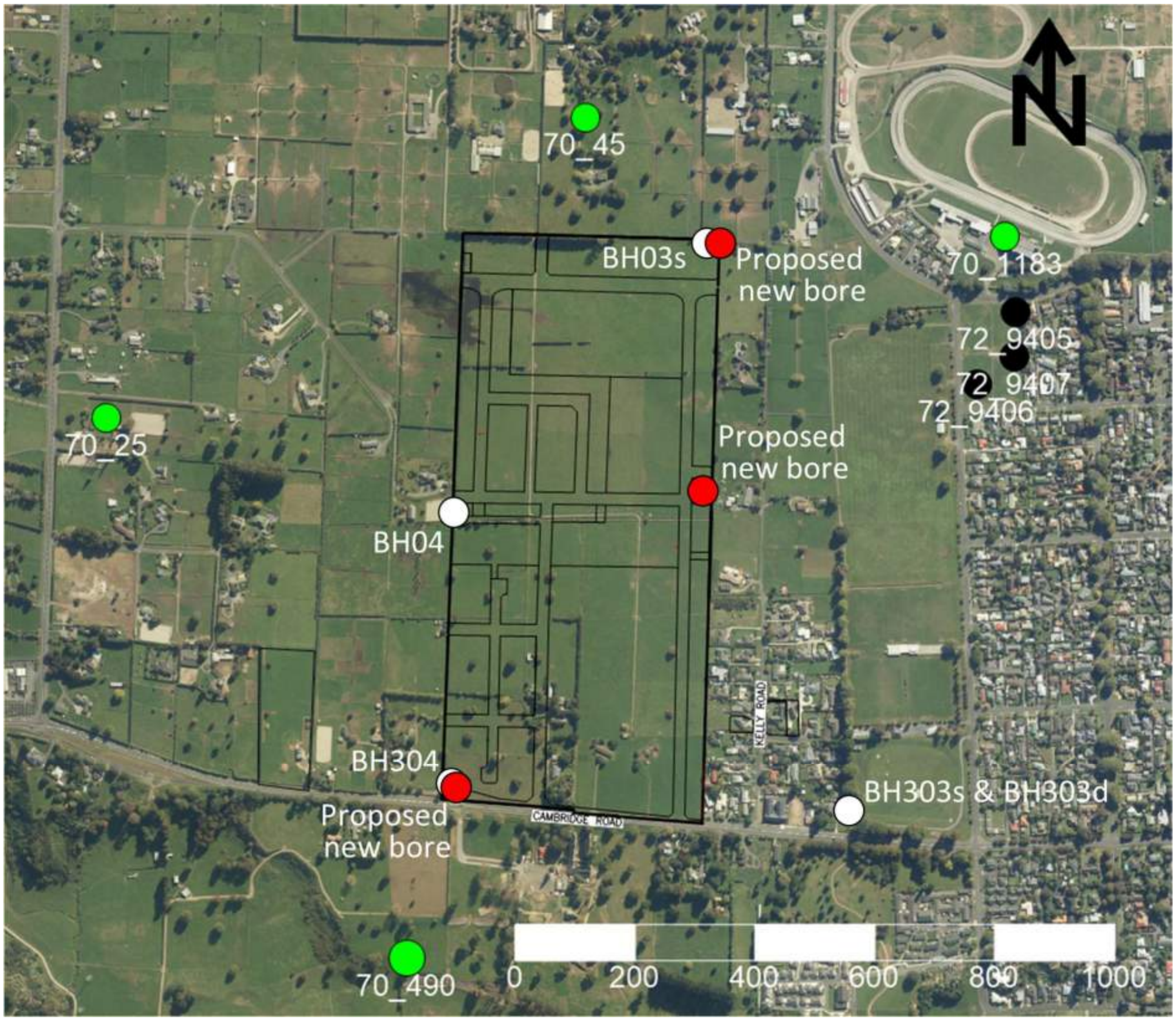


Figure 12: Location of existing private wells (green) and proposed monitoring bores (white existing and red new) for groundwater monitoring.

Existing monitoring wells BH03 and BH04 (both nested shallow and deep pairs) are located on the north and west boundary respectively; monitoring well BH303 (again a paired set) is located approximately 250 metres east of site toward Kelly Road (Figure 12).

Additional dedicated groundwater monitoring wells are proposed to be drilled at targeted locations adjacent to the site boundary to provide coverage where there is a data gap and / or key buildings are present. Alternatively, if the Kelly Road piezometer still exists and is accessible this could be used in lieu of the new bore shown on the eastern boundary.

Manual monitoring on a monthly basis from minimum one month (but preferably longer) before construction commences until active construction is completed, is considered prudent to confirm the extent of any extensive perched aquifer, and if present to confirm that any drawdown does not exceed calculated predictions and to enable prompt supply of water should the bore owner's supply be impacted. Where initial monitoring indicates that there is no perched level than the need for monitoring could be reviewed and potentially removed.

It is proposed that the consent holder liaise with the landowner of the nearest potentially affected private wells (Well ID 70_45 and 70_490) at least one (1) month prior to the commencement of active dewatering to confirm the nature of any take, and then again at least one (1) week prior to works commencing to advise them of the nature of the works and provide an emergency contact (name and all hours phone number) to the bore owner, so that they can notify if they consider that the well is impacted during this time.

The consent holder is to record the date and time of the reported impact and record the action taken in response (e.g. supply of water to the owner or, longer term, deepening of the pump or well).

6.2 Drawdown Induced Ground Settlement

Generally, the predicted drawdown is sufficiently small that it is unlikely to result in adverse ground settlement. This is consistent with our experience at Greenhill Park Subdivision in similar soils, excavation and dewatering depths, where drawdown did not result in any ground settlement which could be attributed to the works.

Considering the groundwater monitoring proposed above, which should serve as an early signal that conditions for potential groundwater induced settlement have been reached, no settlement monitoring is proposed. This should be further reviewed after drilling and initial monitoring of the new piezometers i.e. should they show an extensive perched aquifer and / or compressible soils then some limited ground monitoring at the boundary and pre-construction and post-construction condition surveys may be warranted in the short term.

7 Summary & Conclusions

7.1 Summary of Environmental Effects

Analyses suggest that for the upper bound scenario, where an extensive perched iron pan aquifer exists beneath the site, that the calculated drawdown would still be unlikely to result in significant or damaging consolidation settlement beyond the development area (where there are existing buildings). Any consolidation settlement that does occur within the development area, is likely to occur before private development is completed (subject to construction of the stormwater swale preceding development). Regardless, developers should consider building some flexibility into permanent works such that they could accommodate any small settlements that do occur.

Similarly, the analyses suggest that impacts on any private groundwater wells are likely to be less than minor, but some limited groundwater level monitoring during and following construction would be prudent to confirm that the magnitude and extent of drawdown is as expected or to allow refinement of the analyses, in the unlikely event that it is greater. Groundwater level monitoring will also enable prompt supply of water should the closest shallow private bore owner’s supply be impacted (should the bores still be in use).

7.2 Further Recommendations

The results described in this report are based on the site data collected to date and the indicative basin design available at the time of reporting. Should significant changes be made to the design (i.e. changes to depth or extent, etc.) the modelling will need to be re-run to consider the impact this will have on the calculated effects.

Similarly, the modelling should be reviewed immediately prior to construction of the swale to allow consideration of the results from the proposed new bores. This is particularly important noting that the borehole data will aid in constraining the extent of the perched iron pan aquifer.

Water level monitoring should be carried out during, and for a short time following, construction (at selected sites) to confirm that the extent of changes in groundwater level are within the expected range. We also recommend that observation monitoring of the excavations is undertaken during construction to help manage potential risks associated with groundwater inflows and slope stability.

Site specific testing of the permeability to inform design infiltration rate should be undertaken to inform detailed design of the basin.

8 Applicability Statement

This report has been prepared by Beca on the specific instructions of our Client. It is solely for our Client’s use for the purpose for which it is intended in accordance with the agreed scope of work. Any use or reliance by any person contrary to the above, to which Beca has not given its prior written consent, is at that person's own risk.

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

9 References

Beca, 2019. Cambridge C1 and C2/C3 Infrastructure: Groundwater Model and Assessment. Prepared for Waipa District Council 29 May 2019.

BTW Company, 2020. 3MS Site - Cambridge (Part of C2 Growth Cell), Geotechnical Suitability Report for 3MS of Cambridge 13 November 2020

Beca

BOREHOLE No: **BH 03 deep**

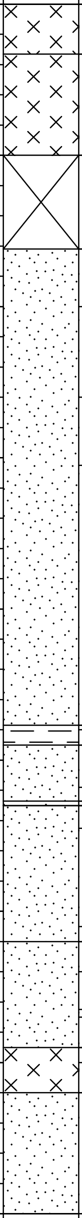
MACHINE BOREHOLE LOG

SHEET 1 of 1

PROJECT: Cambridge West
SITE LOCATION: West Cambridge
JOB NUMBER: 4284909
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,804,574 m
E 1,815,592 m
BOREHOLE LOCATION: 1881 Cambridge road. North east corner of farm
R L: 65 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: 5

DEPTH (m)	FLUID LOSS	DAILY WATER LEVEL	CORE RECOVERY	METHOD	CASING	RQD	SV	τ (kPa)	SPT 'N'	SAMPLES	R L (m)
0 – 0.5			80 %	OB			140+	205+		C	65
0.5 – 1.0			100 %	OB			UTP			C	
1.0 – 1.5			0 %	OB					1 1 2 2 2 2 N=8	SPTLS	64
1.5 – 1.95			100 %	SPT						C	
1.95 – 2.5			100 %	TT						C	63
2.5 – 3.5			60 %	TT						C	62
3.5 – 3.95			44 %	SPT					2 2 2 2 1 1 N=6	SPTLS	
3.95 – 4.5			100 %	TT						C	61
4.5 – 4.95			100 %	SPT					1 0 0 1 1 1 N=3	SPTLS	
4.95 – 5.5			100 %	OB						C	60
5.5 – 6.0			80 %	OB						C	59
6.0 – 6.5			100 %	SPT					1 2 2 3 4 4 N=13	SPTLS	
6.5 – 7.0			100 %	OB						C	58
7.0 – 7.5			100 %	OB						C	
7.5 – 8.0		16/12/2016 25/01/2017	100 %	SPT					1 2 1 2 1 2 N=6	SPTLS	57
9											56



SOIL / ROCK DESCRIPTION

"Soft", SILT, some organics, minor clay, trace fine to coarse sand; dark brown-black; moist low plasticity. Organics: amorphous, rootlets. [Topsoil]

"Hard" SILT, minor fine sand, minor clay, trace organics; orange-brown; moist, low plasticity. Organics: rootlets.

Core loss

Loose fine to coarse SAND, some fine to medium gravel, minor silt; orange-grey with white staining; moist, non plastic. Gravel: Subangular to subrounded greywacke, pumice.
2.0m, grey.

2.6m, thin bed of medium GRAVEL, subrounded. Pumice.

3.05m, orange-grey.

"Stiff" CLAY, some silt, trace fine sand; grey-white; moist, high plasticity. MNo flecks.
4.8m, orange.

"Loose" fine to coarse SAND, some silt, trace fine gravel; orange-grey; moist, non plastic. Pumiceous
5.18m, thin bed of silty CLAY, trace fine sand.
5.28m, thin bed of silty CLAY, trace fine sand.

"Loose", fine to medium SAND, trace silt; grey mottled orange; moist, non plastic. Pumiceous. Subrounded volcanic glass.
5.45m, moderately thin bed of silt clay.
6.1m, moderately thin bed of silt clay.

Medium dense fine to coarse SAND; grey mottled orange; moist, non plastic. Sand: Subangular to subrounded, quartzose.
6.4m, fine to coarse sand, pumiceous, quartzose.
6.65m, trace fine gravel: SW greywacke.
6.7m, thin bed of "soft" silty CLAY.

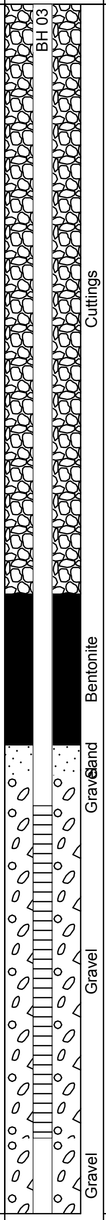
"Hard" SILT, some fine to coarse sand; dark grey; moist, low plasticity.
7.0m, saturated.

Loose medium to coarse SAND, trace silt, trace fine to medium gravel; grey mottled orange; saturated, non plastic. Sand: subangular, quartzose. Gravel: subrounded, greywacke.

END OF LOG @ 8 m

GEOLOGICAL UNIT: **Hinuera Formation**

MONITORED WATER LEVEL



DATE STARTED: 16/12/16
DATE FINISHED: 16/12/16
LOGGED BY: MP/AM
SHEAR VANE No: Geo1249
DRILLED BY: Pro-Drill
EQUIPMENT: Fraste SL.G
DRILL METHOD: OB/SPT/TT
DRILL FLUID: Polymer
DIAMETER/INCLINATION: -/ 90°

COMMENTS:
Terminated at target depth.
Groundwater encountered at 7.5m bgl.
Elevation estimated from BD 34 Topo50 map.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

BECA LIB 1.07.3.GLB Log BECA MACHINE BOREHOLE CAMBRIDGE WEST.GPJ <<DrawingFile>> 27/01/2017 09:03 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Beca 1.07.3 2015-07-31 Prj: Beca 1.07 2014-12-16

BOX: 1 **DEPTH: 0.0 to 4.4 m**

BOX: 2 **DEPTH: 4.4 to 7.5 m**

SPT @ 1.5 m

SPT @ 3.5 m

SPT @ 4.5 m

Beca

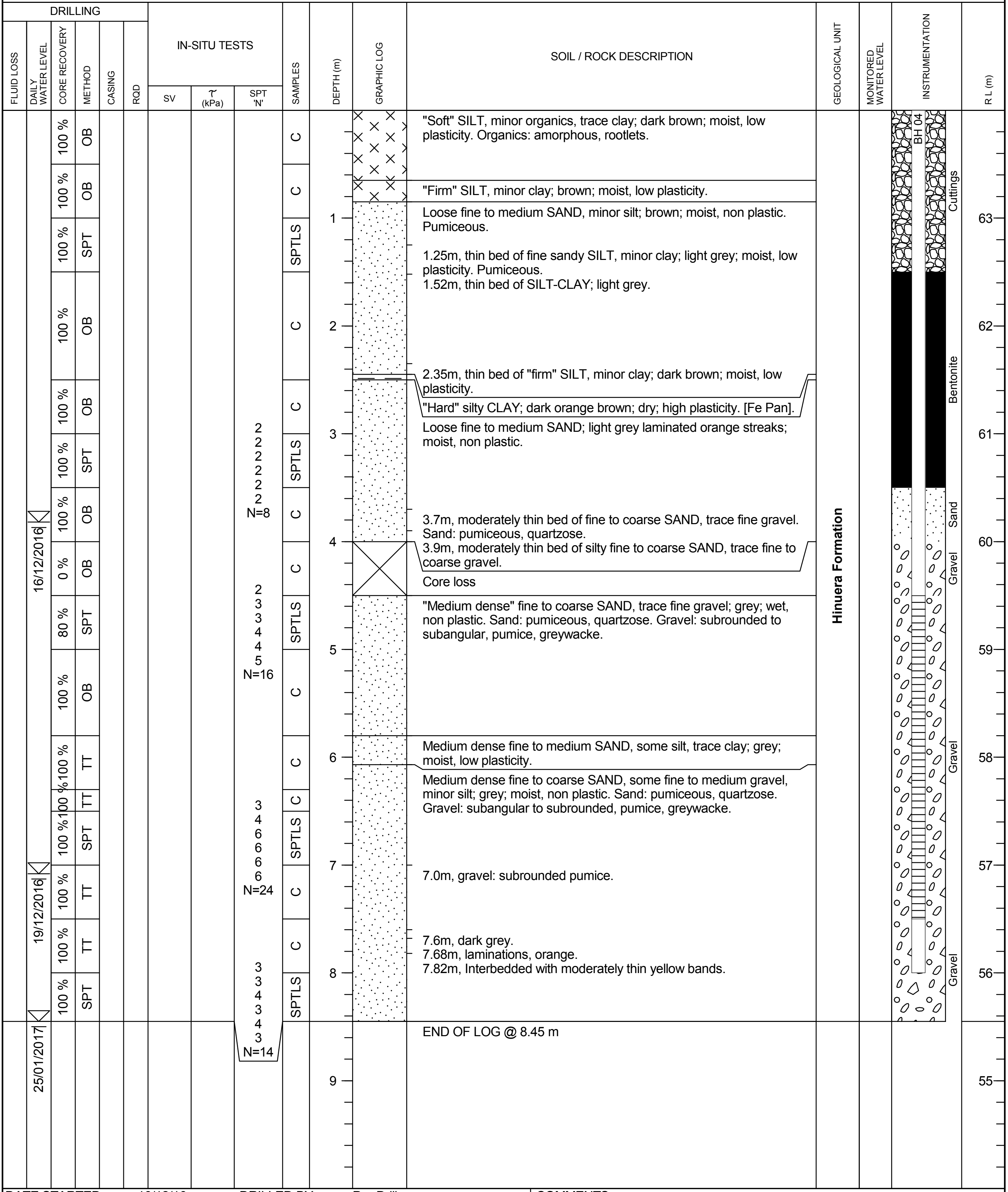
BOREHOLE No: BH 04 deep

MACHINE BOREHOLE LOG

SHEET 1 of 1

PROJECT: Cambridge West
JOB NUMBER: 4284909
SITE LOCATION: West Cambridge
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,804,128 m
E 1,815,167 m
BOREHOLE LOCATION: 1881 Cambridge road. Along western fence line, middle of farm
R L: 64 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: 5



CORE RECOVERY	METHOD	SPT 'N'	SAMPLES
100 %	OB		C
100 %	OB		C
100 %	SPT		SPTLS
100 %	OB		C
100 %	OB		C
100 %	SPT	2 2 2 2 2 2 N=8	SPTLS
100 %	OB		C
0 %	OB		C
80 %	SPT	2 3 3 4 4 5 N=16	SPTLS
100 %	OB		C
100 %	TT		C
100 %	TT		C
100 %	SPT	3 4 6 6 6 6 N=24	SPTLS
100 %	TT		C
100 %	TT		C
100 %	SPT	3 3 4 3 4 3 N=14	SPTLS

"Soft" SILT, minor organics, trace clay; dark brown; moist, low plasticity. Organics: amorphous, rootlets.

"Firm" SILT, minor clay; brown; moist, low plasticity.

Loose fine to medium SAND, minor silt; brown; moist, non plastic. Pumiceous.

1.25m, thin bed of fine sandy SILT, minor clay; light grey; moist, low plasticity. Pumiceous.
1.52m, thin bed of SILT-CLAY; light grey.

2.35m, thin bed of "firm" SILT, minor clay; dark brown; moist, low plasticity.

"Hard" silty CLAY; dark orange brown; dry; high plasticity. [Fe Pan].

Loose fine to medium SAND; light grey laminated orange streaks; moist, non plastic.

3.7m, moderately thin bed of fine to coarse SAND, trace fine gravel. Sand: pumiceous, quartzose.
3.9m, moderately thin bed of silty fine to coarse SAND, trace fine to coarse gravel.

Core loss

"Medium dense" fine to coarse SAND, trace fine gravel; grey; wet, non plastic. Sand: pumiceous, quartzose. Gravel: subrounded to subangular, pumice, greywacke.

Medium dense fine to medium SAND, some silt, trace clay; grey; moist, low plasticity.

Medium dense fine to coarse SAND, some fine to medium gravel, minor silt; grey; moist, non plastic. Sand: pumiceous, quartzose. Gravel: subangular to subrounded, pumice, greywacke.

7.0m, gravel: subrounded pumice.

7.6m, dark grey.
7.68m, laminations, orange.
7.82m, Interbedded with moderately thin yellow bands.

END OF LOG @ 8.45 m

DATE STARTED:	16/12/16	DRILLED BY:	Pro-Drill
DATE FINISHED:	19/12/16	EQUIPMENT:	Fraste SL.G
LOGGED BY:	MP/AM	DRILL METHOD:	OB/SPT/TT
SHEAR VANE No:	Geo1249	DRILL FLUID:	Polymer
		DIAMETER/INCLINATION:	-/ 90°

COMMENTS:
Terminated at target depth.
Groundwater encountered at 3.82m bgl.
Elevation estimated from BD 34 Topo50 map.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

BECA LIB 1.07.3.GLB Log BECA MACHINE BOREHOLE CAMBRIDGE WEST.GPJ <<DrawingFile>> 27/01/2017 09:03 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Beca 1.07.3 2015-07-31 Prj: Beca 1.07 2014-12-16

BOX: 1 **DEPTH: 0.0 to 2.4 m**

NB: box trampled by stock over weekend

BOX: 2 **DEPTH: 2.4 to 5.8 m**

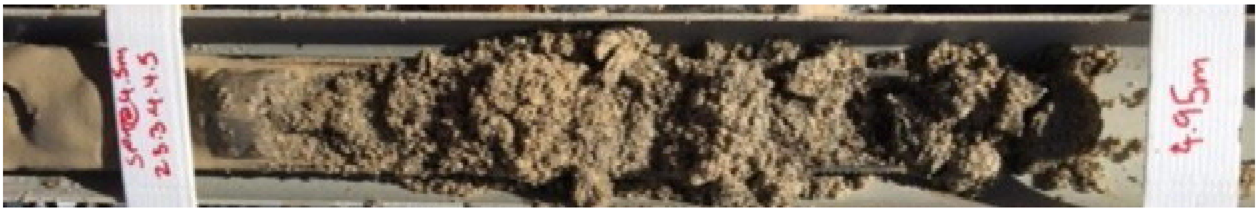
Cambridge C1 - C3 Structure Plan

BOX: 3

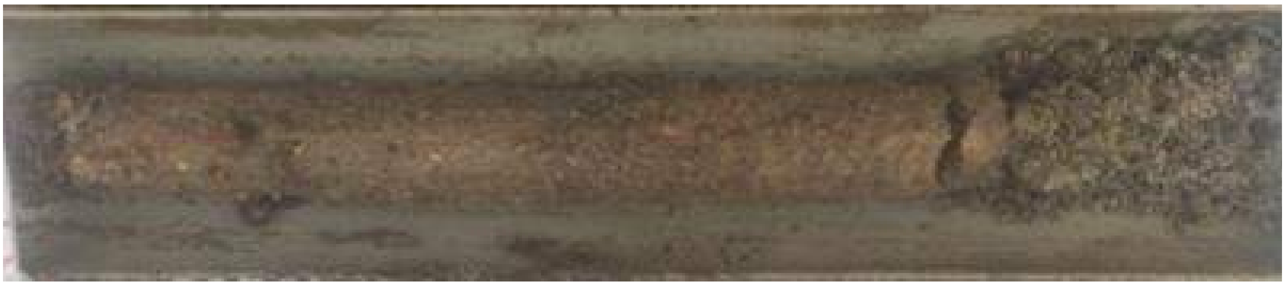
DEPTH: 5.80 to 8.45 m

SPT @ 1.5 m

SPT @ 3.0 m



SPT @ 4.5 m



SPT @ 6.5 m

SPT @ 8.0 m

BOREHOLE No: BH303 Deep

MACHINE BOREHOLE LOG

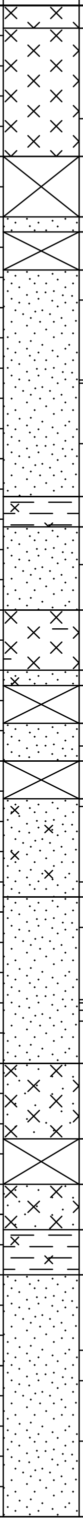
SHEET 1 of 2

PROJECT: Cambridge C1-C3
JOB NUMBER: 3208540
SITE LOCATION: C2 - North of 1871 Cambridge Road to 1 Hamilton Road.
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,803,625 m
E 1,815,814 m
BOREHOLE LOCATION: Refer GI Plan.
R L: 67 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: ±5m

FLUID LOSS	DAILY WATER LEVEL	CORE RECOVERY	METHOD	CASING	RQD	SV	τ (kPa)	SPT 'N'	SAMPLES
		70 %	OB						C
		50 %	OB						C
		20 %	OB						C
		50 %	OB						C
		60 %	OB						C
		80 %	OB						C
		74 %	OB						C
		100 %	OB						C C
		90 %	OB						C
		50 %	OB						C
		60 %	OB						C
		60 %	OB						C
		80 %	OB						C
		100 %	OB						C
		80 %	OB						C
		40 %	OB						C
		95 %	OB						C
		75 %	OB						C

DEPTH (m)	R L (m)
1	66
2	65
3	64
4	63
5	62
6	61
7	60
8	59
9	58



SOIL / ROCK DESCRIPTION

'Soft' clayey SILT, minor clay and organics; dark brown; moist, low plasticity. Organics: rootlets [Topsoil].

'Firm' fine sandy SILT, some clay; brown; dry, high plasticity.
0.75m, fine to coarse sand.
1.0m to 1.4m, no recovery.

'Loose' fine to medium SAND; brown speckled black and orange; wet, non plastic. FeO staining.
1.5m to 1.75m, no recovery.

'Loose' fine to coarse SAND; brown speckled black and orange; saturated, non plastic.
1.95m, moist.
2.48m, very thin (10mm) layer of clayey silt.
2.5m, fine to medium SAND
2.9m, fine SAND, some silt.

'Firm' silty CLAY; greyish brown mottled orange and brown; moist, high plasticity.

'Loose' fine to coarse SAND; grey speckled black, white and orange; moist, non plastic. FeO stained.
3.7m, trace fine gravel: subrounded to rounded, SW-UW, pumice.

'Firm' clayey SILT, trace coarse gravel; grey streaked orange; moist, high plasticity. Gravel: subrounded to rounded, SW-UW, pumice.

'Loose' fine silty SAND; grey, moist, non plastic.
4.5m -4.75m, no recovery.

'Loose fine SAND; grey; moist, non plastic. Very thinly (10mm) bedded.
4.9m, some silt.
5.0m - 5.25m, no recovery.

'Loose' silty fine SAND; grey; moist, non plastic.
5.3m, moderately thin (100mm) layer of clayey silt.
5.8m, very thin (10mm) layer of clayey silt.

'Loose' fine to medium SAND; grey; moist, non plastic.
6.1m, fine SAND.
6.58m, thin (20mm) orange band. FeO stain.
6.6m, thin (50mm) layer of clayey silt.
6.65m, moderately thin (70mm) dark grey band.
6.2m, some silt.

'Loose' fine sandy SILT; grey; moist, non plastic.
7.5m to 7.8m, no recovery.

'Loose' fine sandy SILT; grey; moist, non plastic.

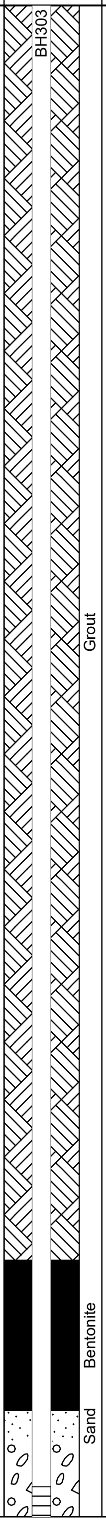
'Firm' silty CLAY; grey; moist, high plasticity.

'Loose' silty fine to medium SAND; grey banded orange; moist, non plastic. Thinly (20mm) interbedded with fine sandy SILT.
8.9m, very thin (6mm) weakly cemented layer. Iron pan.
9.1m, thin (50mm) layer of medium to coarse gravel; cemented sand, iron pan?. Saturated.
9.5m, moist.

GEOLOGICAL UNIT: **Hinuera Formation**

MONITORED WATER LEVEL

INSTRUMENTATION



DATE STARTED: 21/3/19
DATE FINISHED: 21/3/19
LOGGED BY: JMW
SHEAR VANE No: n/a
DRILLED BY: Pro-drill (Auck) Ltd
EQUIPMENT: SLG 02
DRILL METHOD: OB
DRILL FLUID: Water/polymer
DIAMETER/INCLINATION: -/ 90°

COMMENTS:
Borehole terminated at target depth. No strength testing undertaken, borehole cored for indicative permeability to inform piezometer install.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

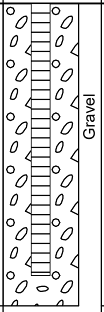
BOREHOLE No: BH303 Deep

MACHINE BOREHOLE LOG

SHEET 2 of 2

PROJECT: Cambridge C1-C3
JOB NUMBER: 3208540
SITE LOCATION: C2 - North of 1871 Cambridge Road to 1 Hamilton Road.
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,803,625 m
E 1,815,814 m
BOREHOLE LOCATION: Refer GI Plan.
R L: 67 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: ±5m

DRILLING						IN-SITU TESTS										
FLUID LOSS	DAILY WATER LEVEL	CORE RECOVERY	METHOD	CASING	RQD	SV	τ (kPa)	SPT 'N'	SAMPLES	DEPTH (m)	GRAPHIC LOG	SOIL / ROCK DESCRIPTION	GEOLOGICAL UNIT	MONITORED WATER LEVEL	INSTRUMENTATION	R L (m)
		90 %	OB						C			'Loose' silty fine to medium SAND; grey banded orange; moist, non plastic. Thinly (20mm) interbedded with fine sandy SILT.	**Hinuera Formation**			
										11		11.1m, moderately thin (150mm) layer of clayey silt.				56
		90 %	OB						C							
										12		END OF LOG @ 12 m				55
										13						54
										14						53
										15						52
										16						51
										17						50
										18						49
										19						48

DATE STARTED: 21/3/19
DATE FINISHED: 21/3/19
LOGGED BY: JMW
SHEAR VANE No: n/a

DRILLED BY: Pro-drill (Auck) Ltd
EQUIPMENT: SLG 02
DRILL METHOD: OB
DRILL FLUID: Water/polymer
DIAMETER/INCLINATION: -/ 90°

COMMENTS:
Borehole terminated at target depth. No strength testing undertaken, borehole cored for indicative permeability to inform piezometer install.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

Cambridge C1- C3

BOX: 1

DEPTH: 0.0m to 3.7m

BOX: 2

DEPTH: 3.7m to 6.5m

BH303

3208540

Machine Borehole Photos

BOX: 3 **DEPTH: 6.5m to 9.0m**

BOX: 4 **DEPTH: 9.0m to 11.5m**

Cambridge C1- C3

BOX: 5 **DEPTH: 11.5 to 12.0m EOH**

Beca

MACHINE BOREHOLE LOG

BOREHOLE No: **BH303 Shallow**

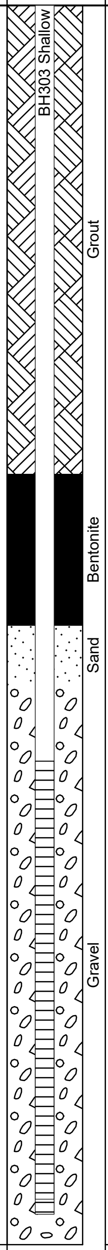
SHEET 1 of 1

PROJECT: Cambridge C1-C3
JOB NUMBER: 3208540
SITE LOCATION: C2 - North of 1871 Cambridge Road to 1 Hamilton Road.
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,803,623 m
E 1,815,813 m
BOREHOLE LOCATION: Refer GI Plan.
R L: 67 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: ±5m

DRILLING						IN-SITU TESTS										
FLUID LOSS	DAILY WATER LEVEL	CORE RECOVERY	METHOD	CASING	RQD	SV	τ (kPa)	SPT 'N'	SAMPLES	DEPTH (m)	GRAPHIC LOG	SOIL / ROCK DESCRIPTION	GEOLOGICAL UNIT	MONITORED WATER LEVEL	INSTRUMENTATION	R L (m)
			WB							0.0 – 8.2		0.0 - 8.2m, borehole wash drilled for piezometer installation.				
										8.2		END OF LOG @ 8.2 m				

DEPTH (m)	R L (m)
1	66
2	65
3	64
4	63
5	62
6	61
7	60
8	59
9	58



DATE STARTED: 22/3/19
DATE FINISHED: 22/3/19
LOGGED BY: JMW
SHEAR VANE No: n/a

DRILLED BY: Pro-drill (Auck) Ltd
EQUIPMENT: SLG 02
DRILL METHOD: WB
DRILL FLUID: Water
DIAMETER/INCLINATION: -/ 90°

COMMENTS:
Borehole washdrilled for shallow piezometer install

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

BOREHOLE No: BH304

MACHINE BOREHOLE LOG

SHEET 1 of 2

PROJECT: Cambridge C1-C3
JOB NUMBER: 3208540
SITE LOCATION: C2 - North of 1871 Cambridge Road to 1 Hamilton Road.
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,803,687 m
E 1,815,141 m
BOREHOLE LOCATION: Refer GI Plan.
R L: 64 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: ±5m

DRILLING: CORE RECOVERY	METHOD	SAMPLES	DEPTH (m)
100 %	OB	C	0.0 – 0.5
100 %	OB	C	0.5 – 1.0
80 %	OB	C	1.0 – 1.5
0 %	OB	C	1.5 – 2.0
60 %	OB	C	2.0 – 2.5
100 %	OB	C	2.5 – 3.0
40 %	OB	C	3.0 – 3.5
100 %	OB	C	3.5 – 4.0
90 %	OB	C	4.0 – 4.5
100 %	OB	C	4.5 – 5.0
60 %	OB	C	5.0 – 5.5
600 %	OB	C	5.5 – 6.0
75 %	OB	C	6.0 – 6.5
100 %	OB	CC	6.5 – 7.0
60 %	OB	C	7.0 – 7.5
0 %	OB	C	7.5 – 8.0
80 %	OB	C	8.0 – 8.5
60 %	OB	C	8.5 – 9.0
100 %	OB	C	9.0 – 10.0

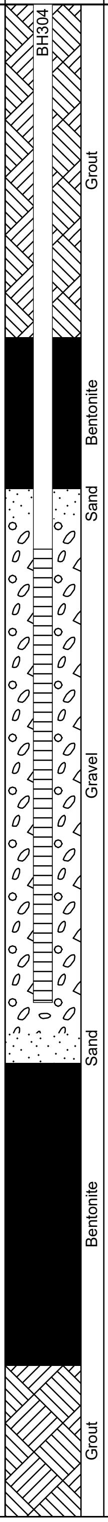
DAILY WATER LEVEL: 12/03/2019

IN-SITU TESTS: SV, τ (kPa), SPT 'N'

GEOLOGICAL UNIT: Hinuera Formation

DEPTH (m)	SOIL / ROCK DESCRIPTION
	'Firm' SILT, some organics, minor clay, minor fine sand; dark brown; moist, low plasticity. Organics: rootlets and grass [Topsoil].
	'Firm' clayey SILT, some fine sand; dark brown; moist, high plasticity.
1	'Loose' silty fine SAND; light brown; moist, non plastic. 0.9m, trace organics; decomposing wood.
	'Loose' fine to medium SAND; brownish grey, speckled black and white; moist, non plastic. 1.45m, fine SAND; grey
	1.5m - 2.0m, no recovery.
2	2.0m - 2.3m, no recovery.
	'Loose' fine to coarse SAND; grey; wet, non plastic. Pumiceous. 2.31m, moderately thin (80mm) layer of silty fine SAND; saturated. 2.53m, moderately thin (100mm) layer of silty fine SAND; wet.
3	'Firm' clayey SILT; grey; moist, high plasticity. 2.66m, trace organics; dark brown. Organics: decomposing wood. 2.7m, thin layer (30mm) of silty fine SAND; grey mottled dark grey [Tephra?]
	'Loose' fine sandy SILT; brown; moist, non plastic.
	3.0m - 3.3m, no recovery.
4	'Loose' fine to medium SAND; brown; wet; non plastic. 3.5m, some silt. 3.53m, moderately thin (160mm) layer of SILT, some clay. 3.9m, fine to coarse SAND; grey speckled black and white. Pumiceous.
	'Loose' silty fine SAND; grey; moist, non plastic.
5	5.0m to 5.2m, no recovery.
	'Loose' silty fine SAND; grey; moist, non plastic. 5.35, very thin (10mm) layer of clayey SILT; dark brown; high plasticity.
	5.5m to 5.7m, no recovery.
6	'Loose' silty fine SAND; grey; moist, non plastic. 5.8m, very thin (15mm) layer of fine to medium pumice, UW, subrounded.
7	'Loose' medium to coarse SAND; dark grey speckled black and white; moist, non plastic. Moderately thinly interbedded with fine to medium SAND, some silt, minor clay; black; low plasticity (MnO, Tephra?) 6.5m, trace fine gravel; subrounded, UW-SW, pumice. 6.6m, thinly (20mm) banded orange (FeO) and grey. 7.2m, orange speckled white, brown, black and grey. 7.3m, thin (15mm) cemented red layer; iron pan. 7.32m, fine to medium SAND; red. 7.4m, fine to coarse SAND. 7.45m, fine to medium SAND, minor silt; reddish brown.
8	7.5m - 8.0m, no recovery.
	'Medium dense', fine to medium SAND, minor silt; reddish brown; moist, non plastic. 8.2m, red. 8.3m, thin (15mm) cemented red layer; iron pan. 8.35m, grey specked black and brown. 8.4m, banded blackish brown and brown.
9	'Loose' silty fine SAND; yellowish brown; moist, non plastic. 9.0m, moderately thin (100mm) dark grey layer.
	'Firm' clayey SILT; brownish grey banded orange; moist, high plasticity. 9.4, grey banded brown, streaked black.
	'Medium dense' fine to coarse SAND, trace fine gravel; orange

R L (m)	DEPTH (m)
63	1
62	2
61	3
60	4
59	5
58	6
57	7
56	8
55	9



DATE STARTED:	12/3/19	DRILLED BY:	Pro-drill (Auck) Ltd
DATE FINISHED:	12/3/19	EQUIPMENT:	Tractor Rig
LOGGED BY:	JMW	DRILL METHOD:	OB
SHEAR VANE No:	n/a	DRILL FLUID:	Water/polymer
		DIAMETER/INCLINATION:	-/ 90°

COMMENTS:
Borehole terminated at target depth. No strength testing undertaken, borehole cored for indicative permeability to inform piezometer install.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

BOREHOLE No: BH304

MACHINE BOREHOLE LOG

SHEET 2 of 2

PROJECT: Cambridge C1-C3
JOB NUMBER: 3208540
SITE LOCATION: C2 - North of 1871 Cambridge Road to 1 Hamilton Road.
CLIENT: Waipa District Council

CIRCUIT: NZTM
COORDINATES: N 5,803,687 m
E 1,815,141 m
BOREHOLE LOCATION: Refer GI Plan.
R L: 64 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: ±5m

FLUID LOSS	DAILY WATER LEVEL	CORE RECOVERY	METHOD	CASING	RQD	SV	τ (kPa)	SPT 'N'	SAMPLES	DEPTH (m)	GRAPHIC LOG	SOIL / ROCK DESCRIPTION	GEOLOGICAL UNIT	MONITORED WATER LEVEL	INSTRUMENTATION	R L (m)
												brown, banded black; moist, non plastic. Gravel: subrounded, SW, pumice.				
												'Firm' clayey SILT; brownish grey banded orange; moist, high plasticity.				
												END OF LOG @ 10 m				
										11						53
										12						52
										13						51
										14						50
										15						49
										16						48
										17						47
										18						46
										19						45

DATE STARTED: 12/3/19
DATE FINISHED: 12/3/19
LOGGED BY: JMW
SHEAR VANE No: n/a
DRILLED BY: Pro-drill (Auck) Ltd
EQUIPMENT: Tractor Rig
DRILL METHOD: OB
DRILL FLUID: Water/polymer
DIAMETER/INCLINATION: -/ 90°

COMMENTS:
Borehole terminated at target depth. No strength testing undertaken, borehole cored for indicative permeability to inform piezometer install.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

BOX: 1 **DEPTH: to 3.0m**

BOX: 2 **DEPTH: 3.0m to 6.5m**

BH304

BOX: 3 **DEPTH: 6.5m to 9.4m**

BOX: 4 **DEPTH: 9.4m to 10.0m EOH**

BOREHOLE No: **BH306**

MACHINE BOREHOLE LOG

SHEET 1 of 1

PROJECT: Cambridge C1-C3	JOB NUMBER: 3208540
SITE LOCATION: C2 - North of 1871 Cambridge Road to 1 Hamilton Road.	CLIENT: Waipa District Council

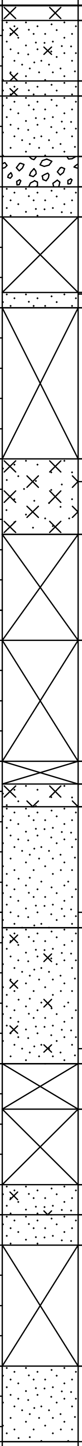
CIRCUIT: NZTM
COORDINATES: N 5,804,538 m
E 1,815,280 m
BOREHOLE LOCATION: Refer GI Plan.
R L: 64 m
DATUM: MSL
COORDINATE ORIGIN: hhGPS
ACCURACY: ±5m

DRILLING: FLUID LOSS | DAILY WATER LEVEL | CORE RECOVERY | METHOD | CASING | RQD
IN-SITU TESTS: SV | τ (kPa) | SPT 'N'
SAMPLES | DEPTH (m) | GRAPHIC LOG | SOIL / ROCK DESCRIPTION | GEOLOGICAL UNIT | MONITORED WATER LEVEL | INSTRUMENTATION | R L (m)

DAILY WATER LEVEL: 11/03/2019

CORE RECOVERY	METHOD	SAMPLES
60 %	OB	C
70 %	OB	C
80 %	OB	C
20 %	OB	C
0 %	TT	C
0 %	TT	C
100 %	TT	C
0 %	TT	C
0 %	TT	C
70 %	OB	C
100 %	OB	C
60 %	OB	C
0 %	OB	C
0 %	OB	C
100 %	OB	CC
0 %	TT	CC
50 %	TT	CC

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9



SOIL / ROCK DESCRIPTION

'Firm' SILT, some organics, minor clay, minor fine sand; dark brown; moist, low plasticity. Organics: rootlets and grass [Topsoil].

'Loose' silty fine SAND; orange brown; dry, non plastic.

'Loose' silty clayey fine to coarse SAND, some fine to medium gravel; orange brown; moist, high plasticity.

'Loose' fine to coarse SAND, some fine to medium gravel; brown; moist, non plastic.

'Loose' fine to coarse GRAVEL, some coarse sand; orange, red, white and black; wet, non plastic. Gravel: subrounded to subangular, SW, pumice and volcanics.

'Loose; fine to coarse SAND, some fine to coarse gravel; grey; moist, non plastic. Gravel: subrounded to subangular, SW, pumice and volcanics.

1.4m - 1.9m, no recovery.

'Loose; fine to coarse SAND, some fine to coarse gravel; grey; moist, non plastic. Gravel: subrounded to subangular, SW, pumice and volcanics.
1.9m, reddish brown speckled orange; iron staining.

2.0m - 3.0m, no recovery.

'Loose' fine sandy SILT; grey; moist, non plastic.
3.15m, moderately thin (150mm) layer of fine to coarse SAND, minor fine to coarse gravel; grey speckled orange and white; moist, non plastic

3.5m - 4.2m, no recovery.

4.2m - 5m, no recovery.

5.0m - 5.15m, no recovery.

'Loose' fine sandy SILT, trace organics; grey streaked brown; moist, non plastic. Organics: decomposing wood.

'Loose' fine to medium SAND; grey banded orange and dark grey; moist, non plastic. Sand: pumiceous.
6.0m, orange band (iron staining).

'Loose' silty fine SAND; grey; moist, non plastic. Moderately thinly (100mm) interbedded with thin beds (10mm) of clayey SILT; grey; moist, high plasticity [Tephra].
6.9m, black.

7.0m - 7.3m, no recovery.

7.3m - 7.8m, no recovery.

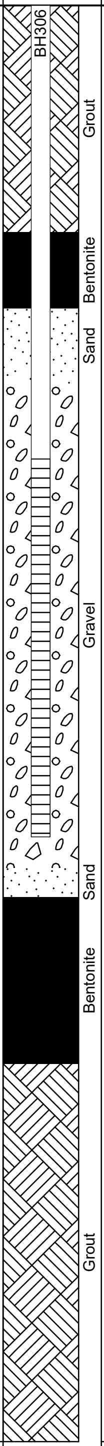
'Loose' silty fine SAND; grey banded dark grey and orange; moist, non plastic. Thinly (10mm) bedded [Tephra].

'Loose' fine to coarse SAND, minor fine to medium gravel; grey speckled dark grey and orange; moist, non plastic
8.2m to 9.0m, no recovery.

'Loose' fine to coarse SAND, minor fine to medium gravel; grey speckled dark grey and orange; moist, non plastic

END OF LOG @ 9.5 m

GEOLOGICAL UNIT: **Hinuera Formation**



R L (m): 63, 62, 61, 60, 59, 58, 57, 56, 55

DATE STARTED:	11/3/19	DRILLED BY:	Pro-drill (Auck) Ltd
DATE FINISHED:	11/3/19	EQUIPMENT:	SLG 02
LOGGED BY:	JMW	DRILL METHOD:	OB/TT
SHEAR VANE No:	n/a	DRILL FLUID:	Water/polymer
		DIAMETER/INCLINATION:	-/ 90°

COMMENTS:
Borehole terminated at target depth. No strength testing undertaken, borehole cored for indicative permeability to inform piezometer install.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

A4 Scale 1:50

Cambridge C1- C3

BOX: 1 **DEPTH: 0.0 to 6.3 m**

BOX: 2 **DEPTH: 6.3 to 9.5m EOH**

BH306

Machine Borehole Photos

3208540

coffey geotechnics

Engineering Log - Machine Auger Borehole

Machine Borehole No.	**MH01**
Sheet	1 of 3
Project No:	**GENZTAUC14196**
Client:	**CAMBRIDGE LAND DEVELOPMENTS LIMITED**
Principal:	
Project:	**31 KELLY ROAD CAMBRIDGE**
Machine Borehole Location:	**Refer to site plan**
Date started:	**26.8.2009**
Date completed:	**26.8.2009**
Logged by:	**MM**
Checked by:	**MM**

Drill model & mounting: Easting: m Slope: -90° R.L. Surface: m Vane No:

Hole diameter: 100 mm Drilling fluid: Water Northing: m Bearing: Datum:

drilling information | **material substance** | **rock mass defects**

stratigraphy	method	support	water	notes samples, tests, etc	RL	depth metres	material (Soil - Soil type; colour, structure. Grading; bedding; plasticity, sensitivity. Secondary and minor components. Rock - Colour, fabric, rock type; discontinuities, additional information.)	moisture condition	recovery %	defect description (number, type, orientation, shape, roughness, aperture, infill description (refer to defect description explanation sheet))
Fill	TT	N	Refer Piezometer Readings				TOPSOIL; dark brown	M	100	
						0.5	SILT, with a mixture of clay and occasional medium grained sand; dark brown to orange/brown	W		
Hinuera Formation						1.0	Fine to medium SAND, trace silt; light grey (pumiceous) occasional rootlets up to 15mm thick interbedded silty clay lenses up to 30mm thick, bluish grey		90	
						1.5	Fine grained silty SAND; cream and grey with interbedded silt lenses 15mm thick - 30mm to 100mm spacings			
						2.0	Very fine silty SAND; light bluish grey			20mm thick iron pan @ 2.3m
						2.5	SAND, coarse grained with some fine angular gravels; orange and reddish brown		60	Partial core loss, casing put down to 3.0m after core loss
				Piezometer 3 Screen		3.0	Coarse grained SAND, with fine pumice gravels		45	0.5m core loss 3.0 to 3.9m
						3.5				
							Silty CLAY; light grey, plastic, trace fine sands	D	90	
						4.0	Silty CLAY; dark brown (lense) to 3.95m	W		50mm thick iron pan @ 3.95m
							SILT; cemented, orange brown, (iron pan)			
						4.5	Very fine SAND, with some silt and interbedded silt lenses (15 to 20mm			

Other columns (not filled in): graphic log core recovery; classification symbol; consistency/ density index; weathering alteration; estimated strength (EW VW W MS S VS ES); vane shear (remoulded /peak) kPa (25 50 75 100 125 150 175); RQD %; defect spacing mm (30 100 300 1000 3000); defect description: particular, general.

method
- AD auger drilling
- OB open barrel
- TT triple tube
- W washbore

support
- N nil
- C casing

vane shear (kPa)
- ● remoulded
- × peak
- ≫× peak greater than 200kPa
- **UTP** unable to penetrate

classification symbols and soil description

based on Field Description of Soil and Rock, New Zealand Geotechnical Society Inc 2005

notes, samples, tests
- U_{50} undisturbed sample 50mm diameter
- U_{63} undisturbed sample 63mm diameter
- D disturbed sample
- N* SPT - sample recovered
- Nc SPT with solid cone
- Bs bulk sample
- E environmental sample

water
- ▼ 10/1/98 water level on date shown
- ► water inflow
- ◁ partial drill fluid loss
- ◀ complete drill fluid loss

moisture
- D dry
- M moist
- W wet
- S saturated

consistency/ density index
- VS very soft
- S soft
- F firm
- St stiff
- VSt very stiff
- H hard
- VL very loose
- L loose
- MD medium dense
- D dense
- VD very dense

weathering
- UW unweathered
- SW slightly weathered
- MW moderately weathered
- HW highly weathered
- CW completely weathered
- RS residual soil

rock mass strength
- EW extremely weak
- VW very weak
- W weak
- MS moderately strong
- S strong
- VS very strong
- ES extremely strong

MACHINE BOREHOLE 14196 MBH 270809.GPJ COFFEY.GDT 7.9.09

Form GEO 5.3 Rev.6

coffey geotechnics

Engineering Log - Machine Auger Borehole

Machine Borehole No.	**MH01**
Sheet	2 of 3
Project No:	**GENZTAUC14196**
Client:	**CAMBRIDGE LAND DEVELOPMENTS LIMITED**
Date started:	**26.8.2009**
Principal:	
Date completed:	**26.8.2009**
Project:	**31 KELLY ROAD CAMBRIDGE**
Logged by:	**MM**
Machine Borehole Location:	**Refer to site plan**
Checked by:	**MM**

Drill model & mounting:
Easting: m
Slope: -90°
R.L. Surface: m
Vane No:
Hole diameter: 100 mm
Drilling fluid: Water
Northing: m
Bearing:
Datum:

drilling information / material substance / rock mass defects

stratigraphy: **Hinuera Formation**
method: TT
support: N
water: Refer Piezometer Readings
moisture condition: W
notes, samples, tests, etc: Piezometer 2 Screen

Depth markings (metres): 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0

material description
thick), light grey
Silty CLAY; light grey, no plasticity, with limonite streaks *(continued)*
Medium to coarse grained (pumiceous SAND, with some silt, and interbedded silt lenses
SILT, trace fine sand; greyish cream
Interbedded SILT, and very fine SAND lenses; very closely spaced, bluish grey with orange staining
Coarse SAND; bluish grey, with dark blue lithic grains
Fine to medium SAND, with interbedded silt lenses closely spaced (15mm) to (50mm)
SILT; light grey, trace fine sand, friable, saturated
Coarse grained SAND; light bluish grey, with trace silt

recovery %: 90, 90, 0, 73, 60

defect description:
- 200mm core loss 5.8 to 6.0m
- 0.4m core loss 6.0 to 7.5m
- 0.5m core loss 7.5 to 9.0m

method

AD	auger drilling
OB	open barrel
TT	triple tube
W	washbore

support

N	nil
C	casing

vane shear (kPa)

●	remoulded
×	peak
>>×	peak greater than 200kPa
UTP	unable to penetrate

classification symbols and soil description

based on Field Description of Soil and Rock, New Zealand Geotechnical Society Inc 2005

notes, samples, tests

U_{50}	undisturbed sample 50mm diameter
U_{63}	undisturbed sample 63mm diameter
D	disturbed sample
N*	SPT - sample recovered
Nc	SPT with solid cone
Bs	bulk sample
E	environmental sample

water

▼	10/1/98 water level on date shown
►	water inflow
◁	partial drill fluid loss
◀	complete drill fluid loss

moisture

D	dry
M	moist
W	wet
S	saturated

consistency/ density index

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

weathering

UW	unweathered
SW	slightly weathered
MW	moderately weathered
HW	highly weathered
CW	completely weathered
RS	residual soil

rock mass strength

EW	extremely weak
VW	very weak
W	weak
MS	moderately strong
S	strong
VS	very strong
ES	extremely strong

coffey geotechnics

Engineering Log - Machine Auger Borehole

Machine Borehole No.	*MH01*
Sheet	3 of 3
Project No:	*GENZTAUC14196*
Client:	*CAMBRIDGE LAND DEVELOPMENTS LIMITED*
Date started:	*26.8.2009*
Principal:	
Date completed:	*26.8.2009*
Project:	*31 KELLY ROAD CAMBRIDGE*
Logged by:	*MM*
Machine Borehole Location:	*Refer to site plan*
Checked by:	*MM*

Drill model & mounting:	Easting: m	Slope: -90°	R.L. Surface: m	Vane No:	
Hole diameter: 100 mm	Drilling fluid: Water	Northing: m	Bearing:	Datum:	

drilling information							material substance									rock mass defects			
stratigraphy	method	support	water	**notes** samples, tests, etc	RL	depth metres	graphic log core recovery	classification symbol	**material** Soil - Soil type; colour, structure. Grading; bedding; plasticity, sensitivity. Secondary and minor components. Rock - Colour, fabric, rock type; discontinuities, additional information.	moisture condition	consistency/ density index	weathering alteration	estimated strength EW VW W MS S VS ES	vane shear (remoulded /peak) kPa 25 50 75 100 125 150 175	recovery %	RQD %	defect spacing mm 30 100 300 1000 3000	**defect description** number, type, orientation, shape, roughness, aperture, infill description (refer to defect description explanation sheet) particular	general
Hinuera Formation	TT	N	Refer Piezometer Readings						SILT; grey, trace very fine sand, no plasticity	W					95				
						9.5			Fine to medium SAND, with some silt; bluish grey						90				
				Piezometer 1 Screen		10.0													
						10.5			SILT; bluish grey, with trace fine sand some clay						90				
						11.0 11.5 12.0			Medium to coarse SAND, trace silt coarse sand, silt						85				
						12.5			MH01 terminated at 12.5 metres.										
						13.0 13.5													

method

AD	auger drilling
OB	open barrel
TT	triple tube
W	washbore

support

N	nil
C	casing

vane shear (kPa)

●	remoulded
×	peak
≫×	peak greater than 200kPa
UTP	unable to penetrate

classification symbols and soil description

based on Field Description of Soil and Rock, New Zealand Geotechnical Society Inc 2005

notes, samples, tests

U_{50}	undisturbed sample 50mm diameter
U_{63}	undisturbed sample 63mm diameter
D	disturbed sample
N*	SPT - sample recovered
Nc	SPT with solid cone
Bs	bulk sample
E	environmental sample

water

▼	10/1/98 water level on date shown
►	water inflow
◁	partial drill fluid loss
◀	complete drill fluid loss

moisture

D	dry
M	moist
W	wet
S	saturated

consistency/ density index

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

weathering

UW	unweathered
SW	slightly weathered
MW	moderately weathered
HW	highly weathered
CW	completely weathered
RS	residual soil

rock mass strength

EW	extremely weak
VW	very weak
W	weak
MS	moderately strong
S	strong
VS	very strong
ES	extremely strong

B

Appendix B – 3D Groundwater Model Build

9 Groundwater model (from Beca, 2019)

The data collected during the 2019 investigations has been used to update the groundwater model with the following information:

- A better understanding of the iron pan extent and thus its effect on groundwater levels in the C2 area.
- Additional hydraulic conductivity values to inform the aquifer parameters.
- Additional groundwater level data in Hautapu (C10 area) and C2 to calibrate the model.

Changes to the model are noted in the following sections, where applicable.

9.1 Model design and construction

9.1.1 Model code

The Cambridge groundwater model was developed in the three-dimensional (3D) finite-difference modelling package Groundwater Vistas (version 7.17 build 15), a software package developed by Environmental Simulations Inc. which uses the MODFLOW2000 computer code.

MODFLOW was developed by the United States Geological Survey (USGS) and is considered an international standard for simulating and predicting groundwater conditions and groundwater/surface-water interactions. MODFLOW2000 is the third major release of MODFLOW by the USGS and this version is considered robust and is therefore recommended for this model.

9.1.2 Domain and mesh geometry

The model domain comprises the Cambridge growth cells north of the Waikato River and is approximately 30 km2, with a north-south grid alignment. The model is subdivided into 325 rows (east-west) and 325 columns (north-south) to create square grid cells of 20 x 20 m, with 250,072 active cells.

9.1.3 Layer type and properties

The model consists of four layers representing a simplified hydrogeological profile underlying the site. The model layers were defined based on the geological model (Section 8) and the known aquifer parameters of the geological units. A summary of the model layers and the geological units and hydraulic conductivity values they represent is provided in Table 8.

Table 3 Hydrogeological layers and assumed hydraulic conductivity values used in groundwater model

Layer	Geology units	hydraulic conductivity (m/d)		Description
		Horizontal	Vertical	
1	2a/2b	4.5	1.0	Unit 2a is a thin (less than 4 m) low permeable silt layer overlying the relatively more permeable silty fine to coarse sand characteristic of Unit 2b. The Unit 2b sands are likely to be interlayered by silt and clay lenses. The initial horizontal hydraulic conductivity is assumed to be the average of all the hydraulic tests conducted in the area and the vertical hydraulic conductivity is estimated to be 1m/d (anisotropy of ~0.2).
2	2c	0.4	0.2	Low energy layer at approximately 10 to 15 m depth. Unit consists of interbedded silt and sand with clay lenses, and therefore, likely to be of low permeability.

Layer	Geology units	hydraulic conductivity (m/d)		Description
		Horizontal	Vertical	
				The initial hydraulic conductivity is assumed based on hydraulic testing undertaken as part of the Waikato Expressway project (Beca, 2016).
3	2d	5.0	2.5	Unit consists of clean sands and pumiceous gravelly coarse sands; and is therefore likely to have relatively high permeability. The initial hydraulic conductivity is assumed based on hydraulic testing undertaken as part of the Waikato Expressway project (Beca, 2016).
4	3, 5a, and 5b	0.2	0.04	Unit 3 is alluvial sand and silt deposits, whereas Unit 5a and Unit 5b are weathered to slightly weathered volcanics (tephra, ignimbrites). These units are likely to have low hydraulic conductivity. Initial aquifer parameters are assumed based on hydraulic testing undertaken as part of the Waikato Expressway project (Beca, 2016).

All layers have been assigned as unconfined. The connectivity between units (i.e. leakance) is calculated in the MODFLOW2000 package using the vertical hydraulic conductivity.

9.1.4 Stresses and boundary conditions

9.1.4.1 Model boundary conditions

Throughflow to the site (groundwater flow entering and exiting the system from a distant source) is modelled using the general head boundary (GHB). This boundary condition operates by allowing prescribed head values (assigned along the boundary) to vary with the calculated heads. The extent to which the GHB head values vary depends on the conductance assigned to the boundary cells.

The heads along the site boundary to the north, east and west have been assigned based on the estimated groundwater contour levels previously discussed in Section 5.2.1. The GHB conductance parameter, initially set to 10 m^2/d, was modified as part of the model calibration (discussed in Section 9.2).

Along the southern border, groundwater seeps out of the steep riverbanks of the Waikato River and therefore is better simulated using a drain boundary condition. This boundary condition operates by removing water from the system by maintaining the head value prescribed when the adjacent groundwater level is higher, while also allowing surface water heads to fall below the specified heads during drier periods without the boundary continuing to recharge the system. Therefore, unlike the GHB, the drain boundary will only allow groundwater to seep from the Waikato riverbanks and will prevent groundwater from entering the system through the riverbank (which is conceptually unrealistic).

Figure 26 shows the model boundary conditions. We note that the drain boundary is set between 53mRL and 67mRL along the southern border to simulate seepage of the shallowest aquifer (the aquifer of interest). The head prescribed in the drain boundary is therefore at a higher elevation than a significant portion of the C3 growth cell (and the Current Town Boundary to a lesser extent). Consequently, the lower section of C3, which is connected to the lower aquifer systems, is excluded from the groundwater model.

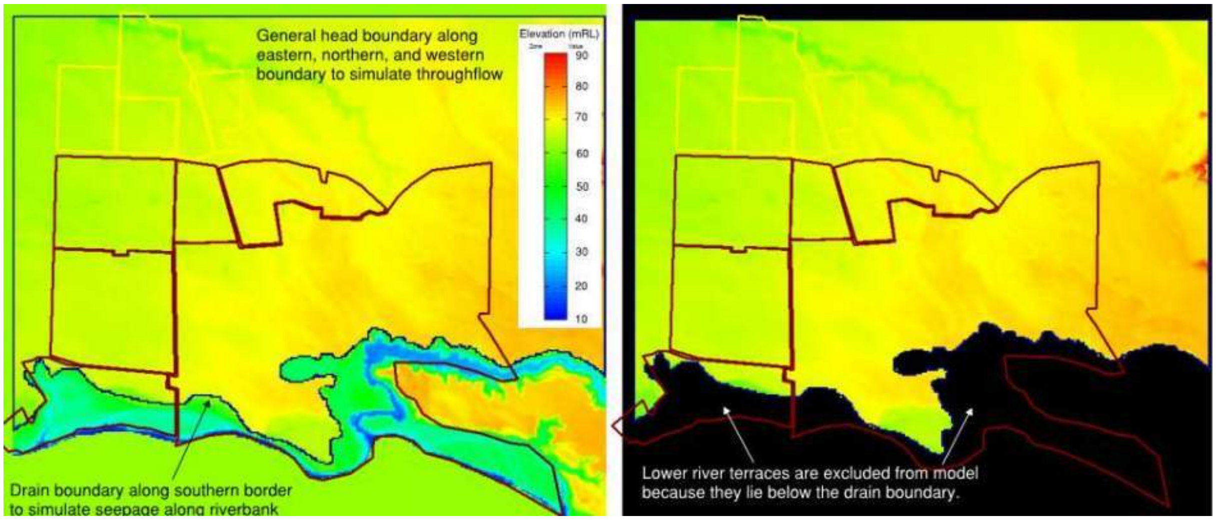


Figure 13 Model screenshots showing location of GHB relative to topography

9.1.4.2 Surface water

Discharge of groundwater to surface water drains and the Mangaone Stream was simulated using the drain boundary which (as discussed in Section 9.1.4.2) maintains the head value prescribed for the water body when the adjacent groundwater level is higher, while allowing surface water heads to fall below the specified head during drier periods (without the boundary continuing to recharge the system i.e. the drain and stream cells can be dry). The drain boundary construction is defined in Table 9.

Table 4 Drain boundaries used in the model

Water body	Description
Mangaone Stream	The heads along the Mangaone Stream are assumed to be at the elevation described by the LiDAR data. The stream is assumed to be 2 m wide based on site observations, and the streambed is assumed to be 1 m in thickness with an initial conductance of 10 m^2/d.
Drainage network	Because the drainage network polyline (sourced from NIWA and the WRC website) does not accurately follow the drain paths, the LiDAR elevation is not considered to be accurate. Therefore, the heads along the drainage network are assumed to be 1 m below the LiDAR elevation. The drainage width is assumed to be 2 m wide and the drainage bed thickness is assumed to be 1m with an initial conductance of 10 m^2/d (as per the Mangaone Stream).

The digitised drains and the drainage network polylines are shown in Figure 27.

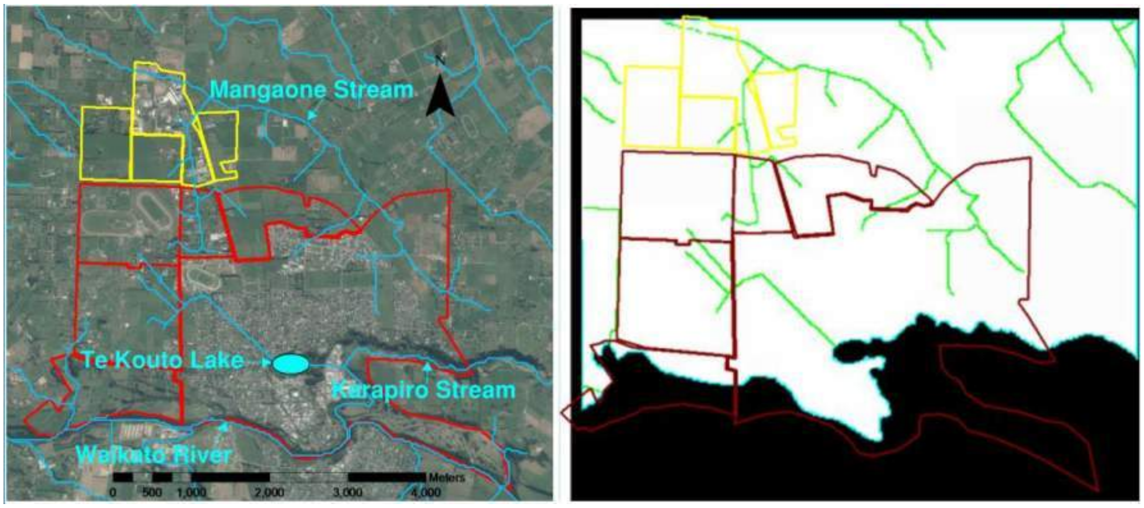


Figure 14 Digitised drains and drainage network polylines used in the model (shown on right hand side in green)

9.1.4.3 Recharge

An average recharge was applied to the top surface of the model. Three recharge zones were defined across the model domain: the rural area, the developed area, and the iron pan area. Recharge to these areas are defined in Table 10.

Table 5 Recharge zones used in the model

Recharge area	Description
Developed	Rainfall is generally prevented from recharging the aquifer in these locations due to infrastructure (e.g. roads, roofs, stormwater system). Recharge is therefore assumed to be 3% to 6% of the average annual rainfall (35mm/yr to 70mm/yr or 0.0001m/d to 0.0002mm/d) to account for impervious cover.
Rural	Estimate of rainfall recharge to the aquifer based on the water table fluctuation method detailed in Section 5.2.2. Recharge is therefore assumed to range between 15% and 30% of the average annual rainfall (175mm/yr to 350mm/yr or 0.0005m/d to 0.001m/d).
Iron pan	There is evidence that the iron pan creates a perched aquifer above the shallow aquifer represented in the model. Long-term monitoring has indicated that the perched aquifer is perennial with a water table which fluctuates seasonally from 0.5m to 1.7m above the iron pan (Section 6.1.3); therefore, it is assumed that rainfall infiltration is prevented from fully recharging the aquifer of interested over the extent of the iron pan. Recharge is assumed to be approximately the same as recharge to the developed area (i.e. 3% to 6%).

Figure 28 shows the areas selected as developed and rural.

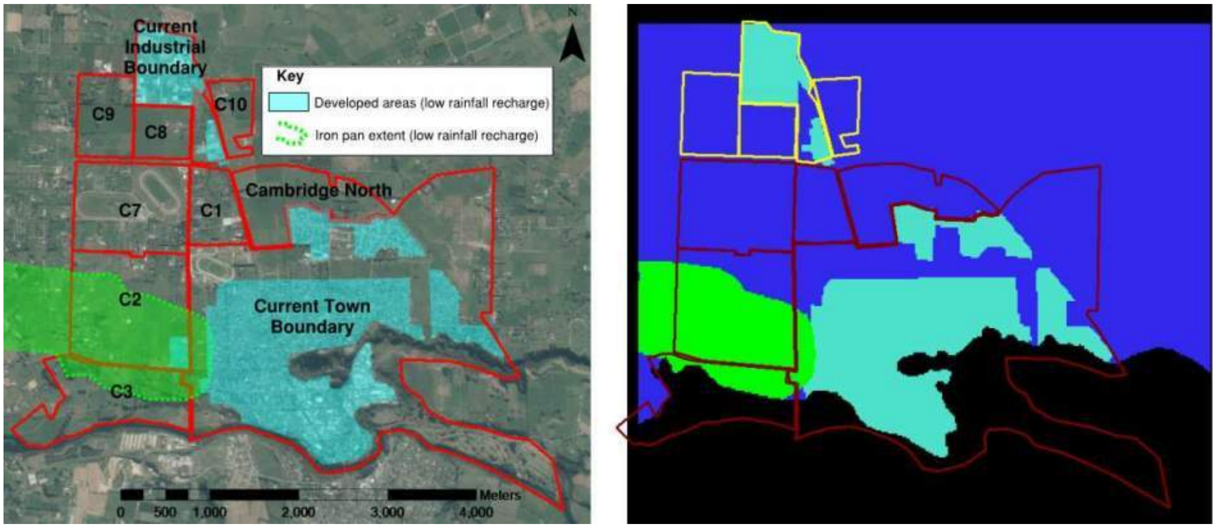


Figure 15 Areas of the model selected as developed (aqua), iron pan extent (green) and rural (remaining area blue)

9.2 Calibration

9.2.1 Calibration approach

The intended purpose of the model is to estimate the effect on groundwater levels due to changes caused by development (e.g. reduced regional recharge, soakage from devices, construction of swales). Therefore, the groundwater levels were the primary focus of the calibration. Often calibrating to a single parameter (such as groundwater head values) can lead to a non-unique result; consequently, the Mangaone Stream baseflow estimate was also used as a secondary calibration.

An iterative trial and error approach was used to calibrate, whereby model parameters were varied between model runs and calibration results were evaluated. The model was calibrated by manually adjusting the parameters listed in Table 11.

Table 6 Parameters used for model calibration

Parameter	Description
Recharge	The rainfall recharge to the rural areas was permitted to vary between the range set by the recharge estimate calculation presented in Table 9 (Section 9.1.4.3).
Hydraulic conductivity	The model layers were initially assigned the hydraulic conductivity values presented in Table 4 (Section 5.2.3). During calibration, the values were permitted to vary within the range set by the in-situ testing results.
Drain conductance	The streambed conductance for Mangaone Stream and the drainage network was initially prescribed a value of 10 m^2/d and was varied during calibration.
GHB conductance	The GHB conductance initially prescribed a value of 10 m^2/d and was varied during calibration.

9.2.2 Calibration to groundwater levels

There are 90 groundwater level observations available for calibration which were weighted to reflect the data quality. Figure 29 shows the groundwater level distribution and confidence level and Table 12 describes the weighting, which is based on how well the data reflects the average annual groundwater level. The weighting varies from 1 representing data providing the highest confidence level to 0.1 for the data providing the lowest confidence.

The 2019 site investigation has provided additional groundwater level information, as follows:

- The Hautapu water levels (BH02pz, BH03pz, BH06pz) have been recorded between Oct 2018 and Apr 2019, and therefore the confidence levels of these target heads have changed from low to high.
- There is one additional bore (BH306) which has only one dip measurement (i.e. low confidence level).

Table 7 Weighting of groundwater level observations

Weighting	Confidence level	Description	Number of data
1	High	Data with continuous records for approximately a year (in some cases longer).	10
0.5	Medium	Data with manual dip records for a minimum of a year.	22
0.25	Low	Data with a single manual reading from bore logs or reports where information provided indicates water level was static (i.e. the water level recorded is taken below ground level, was measured after development of the piezometer, and not as part of hydraulic testing).	7
0.1	Very low	Data (procured from the Waikato Regional Council records) with a single manual reading where there is no information to provide confidence that the water level reading represents static conditions (i.e. whether it was recorded during drilling or after development, if it was recorded during a pumping test, if the recorded level is measured from ground level, or there is incomplete screened interval details).	51

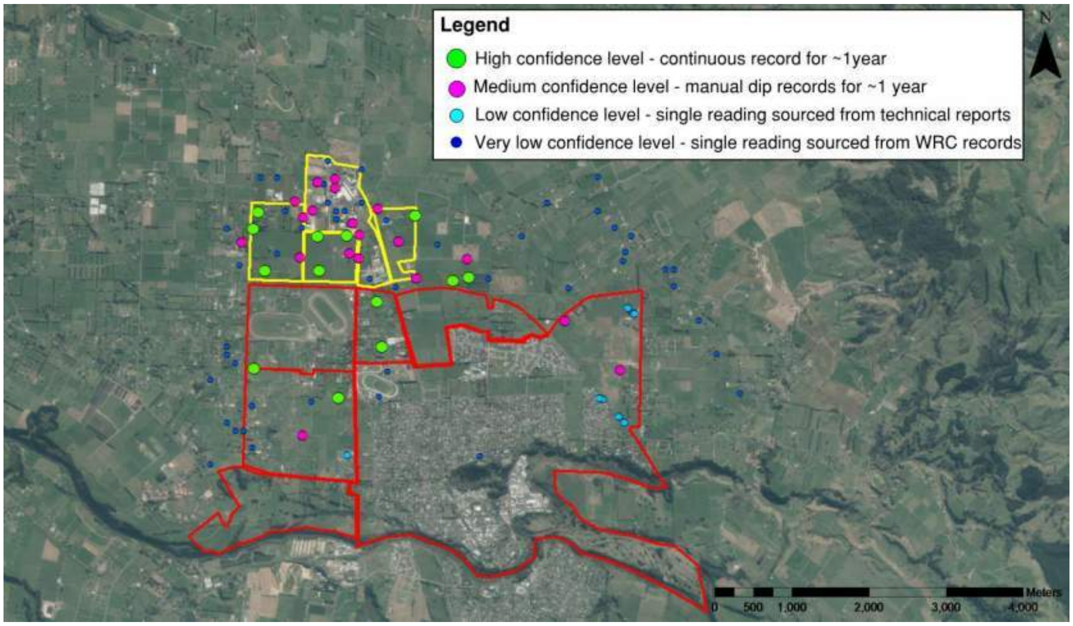


Figure 16 Distribution of target heads with confidence level

The calibration was undertaken in steady state i.e. reflecting long term average groundwater conditions. Figure 30 shows a scatter plot of all the measured head data compared to the corresponding modelled values. The scaled RMS error based on the groundwater levels is 4.3%, for a weighted calibration of the 90 data points.

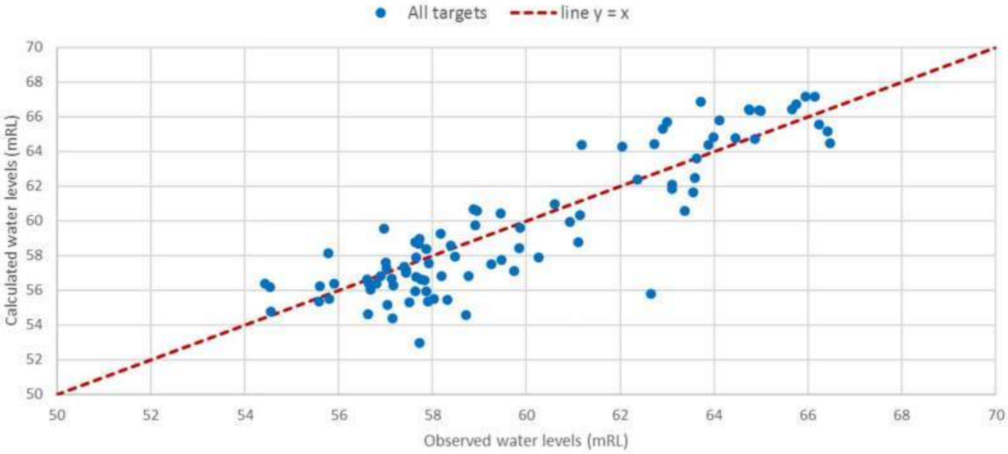


Figure 17 Scatter plot of all measured head data compared to corresponding modelled values

The residuals (difference between observed and calculated head values) versus the observed values are plotted in Figure 31, with the residuals colour coded according to their confidence level (the high confidence level observations are in red).

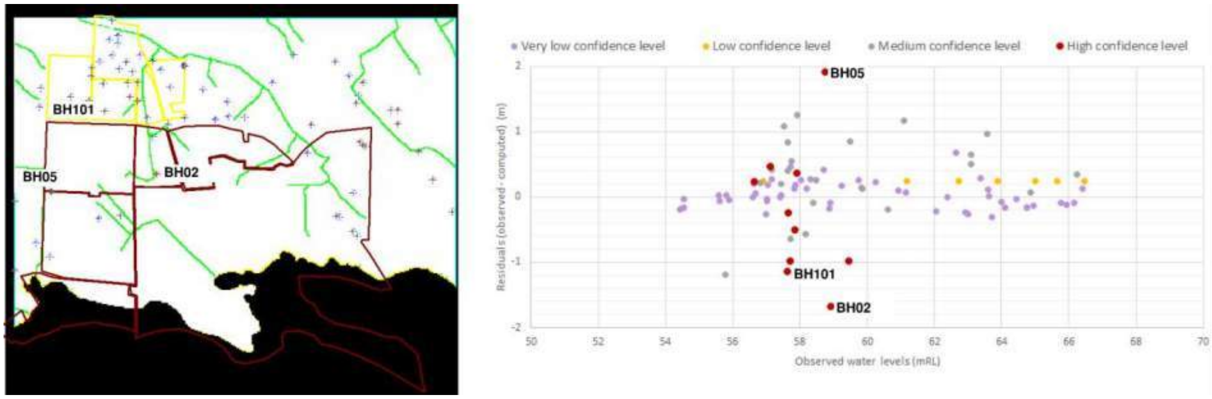


Figure 18 Residuals (difference between observed and calculated head values)

Most of the calculated heads are within 1m of the observed; however, there are three high confidence level targets which have residuals indicating the calculated heads are either greater than or less than the observed by between 1m and 2m. Two of those targets are within the residential growth areas (BH05 in C7 and BH02 in C1) and the remainder are located in the industrial area to the north (BH101 in C8).

The observed average groundwater level at BH05 is approximately 2m higher than the model prediction (58.8mRL observed compared to the modelled 56.9mRL) whereas the observed groundwater levels at BH02 and BH101 are between 1m and 2m less than the model prediction (BH02 water level is observed at 59.2mRL compared to the modelled 60.6mRL and the BH101 observed water level is at 57.6mRL compared to the modelled 58.8mRL). The reason for this is likely due to local hydrogeological variations (which cannot be efficiently simulated in a model of this scale) as evidenced by the long-term monitoring at BH01 to BH05 (Figure 18 in Section 5.2.1) which indicates the water level at BH05 is more responsive to rainfall than the other bores.

The distribution of the residuals is shown on the map in Figure 32.

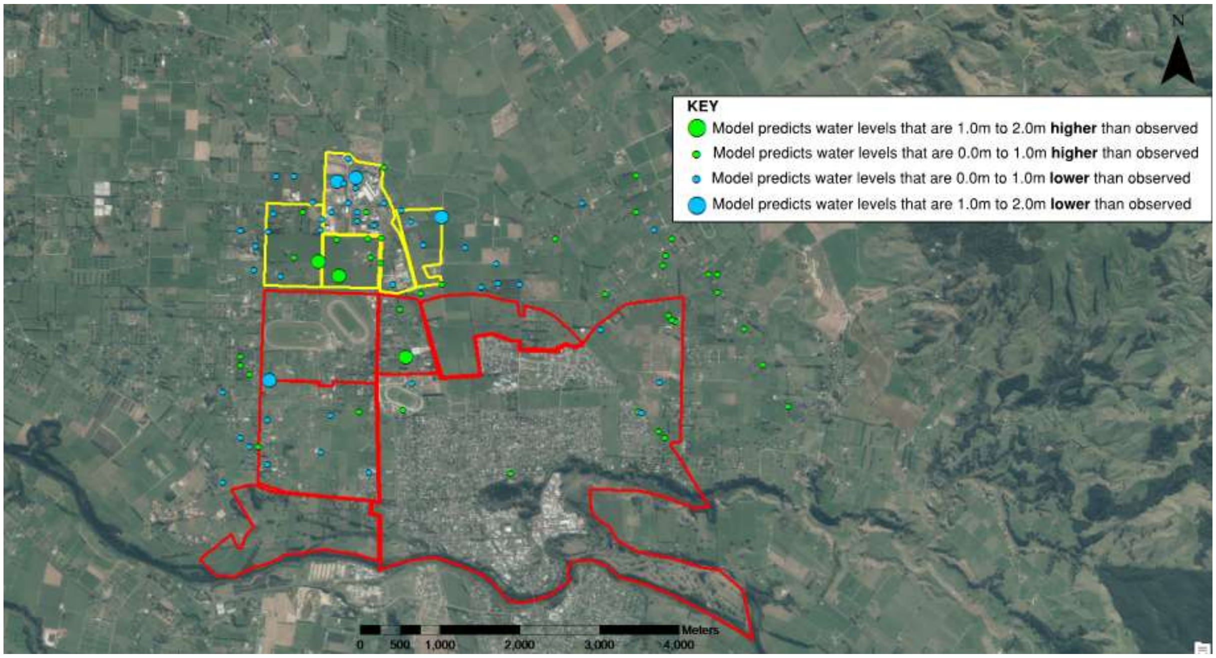


Figure 19 Distribution of residuals (green represent areas where the model over-predicts the groundwater levels and blue is where the model under-predicts)

The variability in groundwater levels (indicating the complexity in the geological depositional history) within the model are evident by the distribution of the observed levels that are either over-estimated or under-estimated by the model (e.g. there is no area where the model consistently under or over predicts). The uncertainty in the model's ability to predict the groundwater levels will affect the accuracy of any simulations used to inform the soakage capacity of the growth cells (as discussed in Section 10).

9.2.3 Calibration to flow data

The model was also calibrated to the baseflow estimate along the Mangaone Stream. The resulting calibration indicates a flow of 0.044m^3/s, an 8% to 12% difference to the assumed average baseflow (0.04m^3/s to 0.05m^3/s). We note that due to the paucity of data, calculating the scaled RMS error is not statistically significant, and therefore, the percent difference is used instead.

9.3 Calibrated model results

9.3.1 Calibrated model parameters

The resulting calibrated model parameters are shown in Table 13.

Rainfall recharge in the growth cells (except where the iron pan is located) is estimated to be 0.0006m/d (approximately 20% of average annual rainfall), which is at the lower end of the expected range (20% to 40%). However, since the recharge estimate is within the calculated range (14% and 27% as shown in Section 5.2.2) it is considered reasonable. Also, conceptually the perched layers above the aquifer likely intercept some of the rainfall, which infiltrates into the ground, reducing the amount of recharge reaching the aquifer.

Recharge to the shallow aquifer over the iron pan extent has been calibrated to be 0.0001m/d (approximately 3% of average annual rainfall), similar to the developed area which was a assigned a recharge of 0.00016m/d (approximately 5% of average rainfall).

The hydraulic parameters for the calibrated model are reasonable and within the range of values calculated from the permeability and soakage testing at the site. While storage values are not relevant to steady state models, an estimate of the specific storage was inferred from the calibrated rainfall recharge (based on the water table fluctuation method discussed in Section 5.2.2).

Table 8 Calibrated model parameters

Unit	Recharge (m/d)	Horizontal K (m/d)	Vertical K (m/d)	Specific Storage
2a/2b	0.0006	4.5	1	0.15
2c	-	0.75	0.25	-
2d	-	10	5	-
3/5a/5b	-	0.2	0.04	-

9.3.2 Head distribution

The simulated distribution of heads is illustrated in Figure 23 and shows contours which approximate the groundwater contours estimated from the available data. As noted in the calibration discussion (Section 9.2), the model generally simulates the groundwater levels; however, there are local variabilities which are not captured in the model. This uncertainty in the model affects the confidence level of the predictive scenario simulations and is discussed further in Section 10.

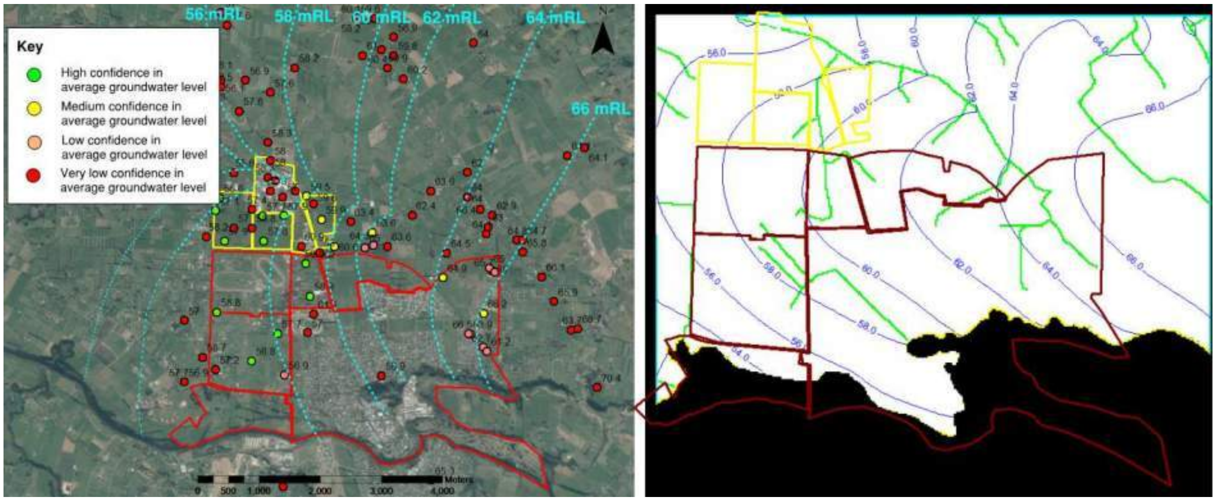


Figure 20 Approximate groundwater level contours estimated from the available data

9.3.3 Groundwater flow budget

The simulated groundwater flow budget for the model calibration in steady state is presented in Table 14. Rainfall recharge is the primary source of flow into the site, accounting for 92% of the inflow budget. Approximately 40% of the groundwater is discharged to the drains, with the Mangaone Stream being the primary watercourse interacting with the aquifer. While the aquifer is not directly connected to the Waikato River, approximately 30% of the groundwater flows towards the Waikato River (also Karapiro Stream and Te Kouto Lake) where it discharges as seepage along its riverbanks.

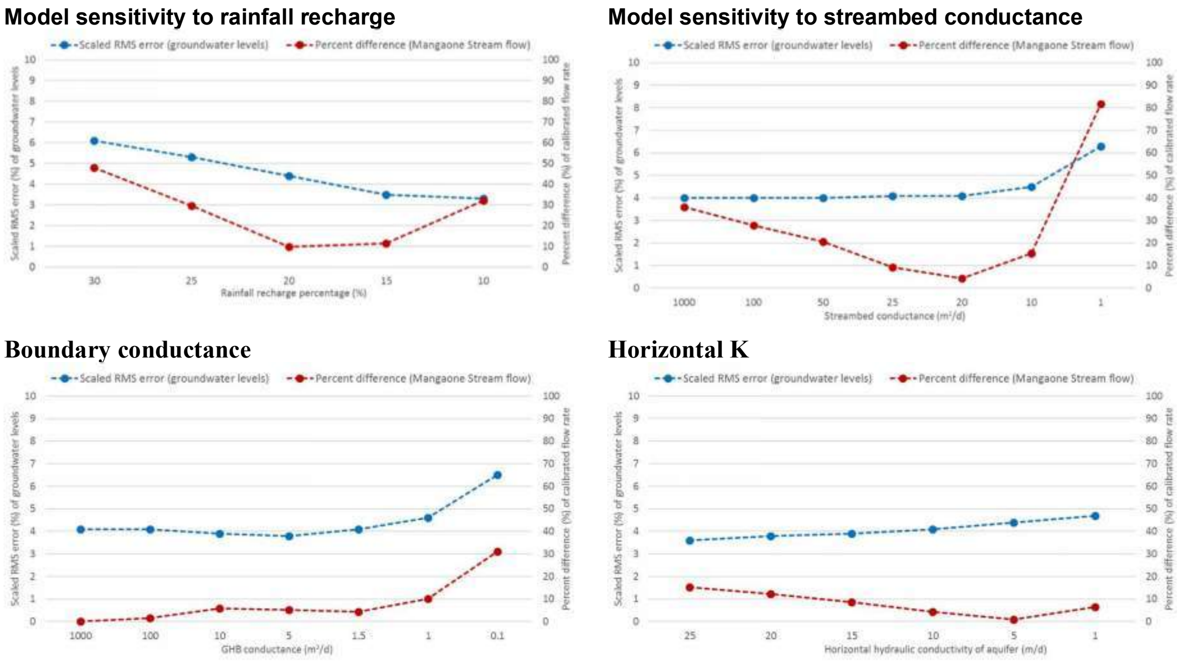
Table 9 Simulated groundwater flow budget for the model calibration in steady state

Flow component	Flow IN (m^3/d)	Flow OUT (m^3/d)	Net flux (m^3/d)	Budget proportion
Rainfall recharge	11,680	-	11,680	88% (IN)
Evapotranspiration	-	1,670	-1,670	13% (OUT)
Discharge to surface water bodies	-	4,962	-4,962	37% (OUT)
Aquifer throughflow	1,620	2,404	-784	12% (IN); 18% (OUT)
River bank seepage	-	4,264	-4,264	32% (OUT)
Total	13,300	13,300	0.02	-

9.4 Sensitivity analysis

The sensitivity of the model calibration performance to various parameters was established during the calibration process, with the following general comments regarding parameter sensitivity:

- The model is most sensitive to changes in rainfall recharge, followed by stream bed conductance. Figure 30 shows that the best calibration is achieved when recharge is between 15% and 20% of rainfall, while the Mangaone Stream is simulated with a streambed conductance between 20 and 25 m^2/d.
- The aquifer parameters and GHB conductance have less of an impact on model calibration as a whole (Figure 34); however, refinement of the water levels within certain growth cells are affected by these parameters.


Model sensitivity to rainfall recharge

Model sensitivity to streambed conductance

Boundary conductance

Horizontal K

Figure 21 Results of sensitivity analysis

C

Appendix C – Basin Mounding Assessment

Background

Hantush (1967) proposed a solution describing the growth and decay of groundwater mounds in response to uniform infiltration over a given time period. Key assumptions and limitations

- Assumes a water-table aquifer of infinite extent and finite thickness with a horizontal, impermeable base.
- Includes the Dupuit assumptions of horizontal flow and negligible change of transmissivity with a change in head.
- The spreadsheet assumes a flat groundwater table i.e. doesn't account for horizontal flow away from the site.
- No accounting for vertical anisotropy and neglects the unsaturated zone. The height of groundwater mounding is underestimated by the Hantush equation where vertical anisotropy is present and overestimated where an unsaturated zone is present.
- The method doesn't account for storage within the basin itself (with an estimated volume of ~30,000 m^3) nor the additional storage within the forebays (~19,000 m^3).
- It is noted that the spreadsheet provides a total mound height relative to an arbitrary starting water level but cannot account for the actual available height between the water table and basin IL.

Calculation Inputs

Key inputs for each basin and storm are summarised below

Parameter	
BASIN	
Basin Width [m][1]	~100
Basin Length [m][1]	~175
Basin Base Area [m^2][1]	14,000
Basin Soakage Area [m^2]	5,500
Design infiltration rate [mm/hr]	100
Discharge rate [m3/hr][2]	1,400
STORM EVENT	
2-year total inflow to basin [m]	12,037
Time to drain 2-year storm [hr][3]	9 (6)
10-year total inflow to basin [m]	23,213
Time to drain 10-year storm [hr][3]	17 (12)
100-year total inflow to basin [m]	44,315
Time to drain 100-year storm [hr][3]	32 (22)
CALCULATION INPUTS	
Recharge (infiltration) rate of [ft/day]	7.87
Specific yield[4]	0.25
Horizontal hydraulic conductivity [ft/day][5]	34.02
Basin ½ width [ft]	123
Basin ½ length [ft]	246
Infiltration period 2-year event [d]	0.46
Infiltration period 10-year event [d]	0.83
Infiltration period 100-year event [d]	1.62
Initial thickness of saturated zone [ft][6]	32

[1] The analysis uses the basin footprint (being the area of cut greater than 2 m bgl) rather than the smaller soakage field area, as the soakage field would only be accessed via a scruffy dome if water level rises, and

as the basin is not lined, some soakage out of full footprint is expected to occur. Note: if we assume soakage out of forebays (swales) also than total soakage area would be closer to 20,000 m^2.

[2] Total discharge rate (m^3/hr) = basin area (m^2) * design infiltration rate (m^3/hr/m^2)

[3] Time to drain (hr) = total storm volume (m^3) / discharge rate (m^3/hr). The value in brackets is the time to drain if we assume soakage via the forebays (swales) also.

[4] Published value for sand-gravel = 0.15 – 0.3 (Driscoll, 1995)

[5] CHT rate of 1.2x10^{-4} m/s from BIL site; **this is a critical input which needs to be verified for design**. The unfactored test rate is used, as the infiltration rate already incorporates a FoS of 4

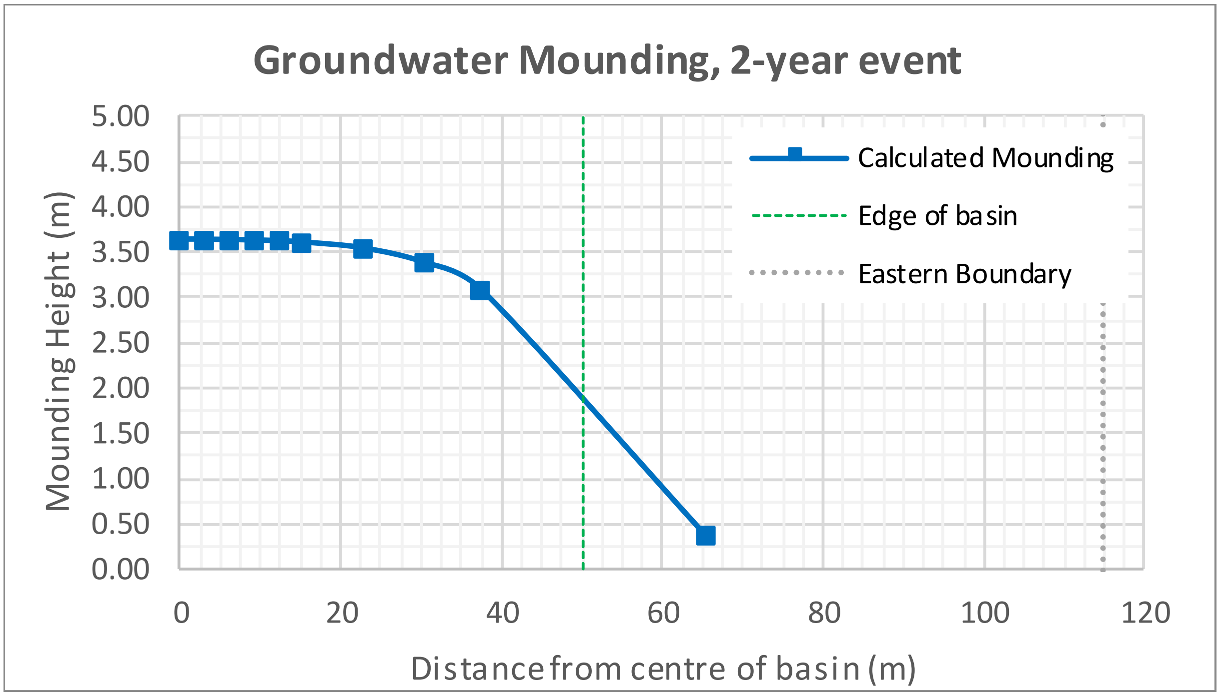
[6] Assume 10 m thickness based on Figure 4 of main report

Results

2-year storm

The assessment suggests that where all soakage is discharged via the basin:

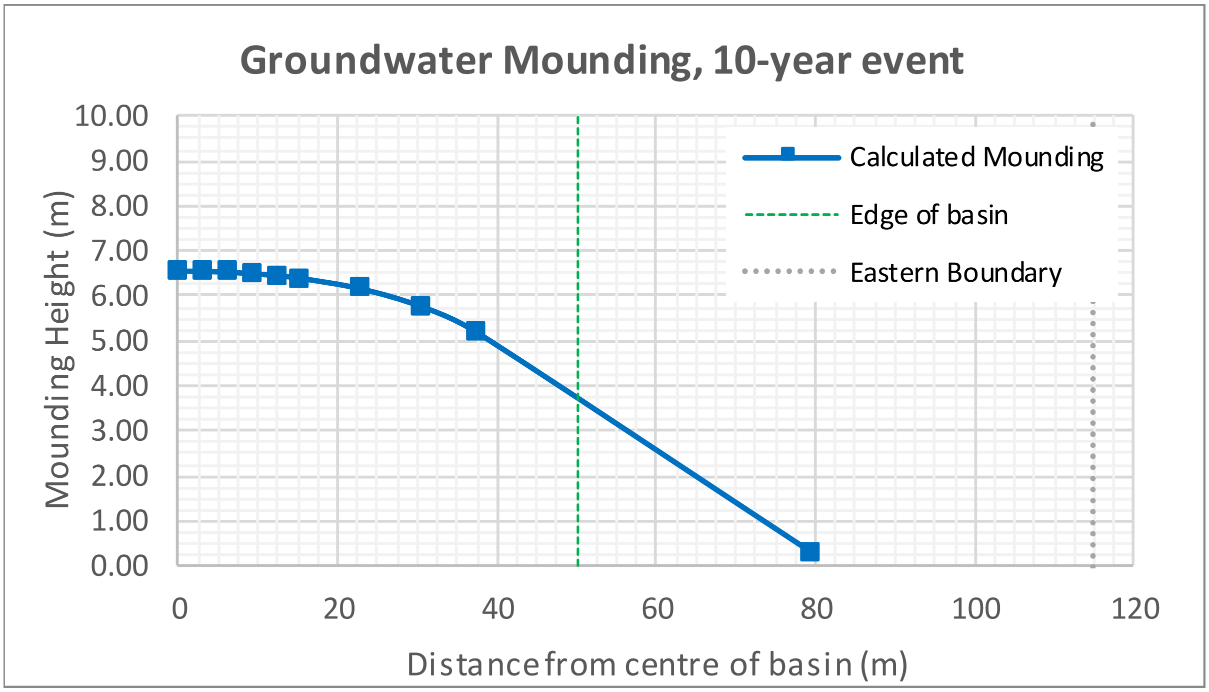
- A maximum mounding height at centre of basin of 3.6 m.
  It is noted that the spreadsheet provides a total mound height relative to an arbitrary starting water level but cannot account for the actual available height between the water table and basin IL. At the basin the deeper groundwater level is a maximum of ~2.8 m below the IL so the analysis indicates that the mound will daylight into the basin; however, the water would then be stored in the basin which has a much greater storage capacity than the ground so the max height of the mound will be smaller. The basin will be partially flooded for a period of time and the infiltration rate may slow down until the water is fully discharged.
- Measurable mounding (i.e. a change in level of 0.25 m) is calculated to extend ~65 m distance from the centre of basin, or ~15 m from the edge of basin
- There would be no measurable mounding at the property boundary



10-year storm

The assessment suggests that where all soakage is discharged via the basin:

- A maximum mounding height at centre of basin of 6.5 m.
  As per above, the maximum mounded height will actually be less, and the basin itself will be partially flooded and may take several days to fully clear.
- Measurable mounding (i.e. a change in level of 0.25 m) is calculated to extend ~80 m distance from the centre of basin, or ~30 m from the edge of basin
- Mounding of up to 4 m could occur at the edge of the basin.
  As noted above it will likely be less when accounting for storage in the basin but regardless even assuming a mound height of 4 m the groundwater level would remain at least 2 m below ground level i.e. no surface breakout / flooding.
- There would be no measurable mounding at the property boundary



The above assumes that all soakage is via the central stormwater reserve and does not account for any incidental soakage out of the unlined forebays (swales) to the south prior to reaching the main basin. This would be expected to provide a more distributed infiltration, over a shorter time period and hence less mounding.

100-year storm

The assessment suggests that where all soakage is discharged via the basin that the mounding would exceed the ground surface, however as noted above the method does not account for storage in the basin which has sufficient storage to fully hold the total storm volume allowing for a slower discharge to ground over time (with further buffer provided by the forebays).

As the results are not realistic in terms of actual operation, they are not presented further

APPENDIX C – STANTEC LETTER: TRAFFIC S92 RESPONSE

6 April 2021

Mr M Smith
3Ms of Cambridge
211 Zig Zag Road
RD1
CAMBRIDGE

CC: matt@3msofcambridge.co.nz

Dear Matt,

C2 Growth Area, Cambridge - S92 Response: Traffic

Stantec has been asked to provide technical evaluation and advise in response to two matters set out in the Waipa District Council S92 request as follows:

Transportation and Roading

30. Please provide further detail related to design layout, operation and performance of the intersection of Road 10 with Cambridge Road and Chartwell Properties intersection on the opposite side to prove it will operate safely and efficiently. This may require traffic engineering support.

31. The Council remains concerned there is a limit to the traffic that can safely and efficiently use Road 10 and Road 8 without the north/south Collector Road being constructed. Please provide an assessment of this including identification of a limit on the number of lots and development that is appropriate before the north/south collector road and intersection is required.

These are addressed as follows:

In response to item 30:
Detailed design layout, operation and performance of the intersection of Road 10 with Cambridge Road and Chartwell Properties intersection

A plan has been prepared showing the detailed design layout of the Road 10 and Chartwell Properties intersections with Cambridge Road (**Appendix A**). The plan extends west to include the proposed Road 8 intersection with Cambridge Rad. By way of a summary, the plan shows the following:

- An extension of the existing central painted median west across the site frontage to provide for continuity and integration of the proposed Road 8 and Road 10 intersections with both the Chartwell Properties as well as the Kelly Road intersections;
- Road 10 is shown safely separated from the Chartwell properties intersection by about 81m;
- Sidra modelling (attached) shows the expected right turn queue on Cambridge Road waiting to access Road 10 to be less than 1.0 vehicle 95th percentile back of queue PM peak at 2031, well clear of the Chartwell Properties access road and adequately providing for drivers to taper into the right turn median waiting area ahead of making the turn;
- The painted median has a width of 3.0m safely providing for vehicles waiting on it clear of the adjacent through traffic lanes;

- The through traffic lanes are shown as continuing to provide for the current level of service at 3.5m with retention of the road shoulder areas.

By way of further observation it is noted that:

- The location of Road 10 is consistent with the location for an intersection as shown on the C2 area Structure Plan, Appendix S19 – Cambridge C1 and C2/C3 Structure Plans. These also anticipated the Chartwell Properties intersection;
- The location of the proposed Road 10 was known by Council with some certainty, through its engagements with the land-owner, at the time the Chartwell intersection was granted consent to develop and form the new intersection there;
- The proposed Road 10 is separated from Kellv Rd by about 130m;
- Road 8 is also shown separated from Road 10 by a further 230m;
- Safe intersection sight distances in excess of the Regional Infrastructure Technical Specification (RITS) and Austroads Guide to Road Design Part 3: Geometric Design guidelines. These specify safe stopping sight distances in the range 64m to 81m for an operating speed of 60km/h and a range of 83m to 102m for an operating speed of 70km/h. Development of the C2 growth area is expected to be commensurate with relocation of the speed restriction sign across the site frontage creating a 50km/h speed restriction and an expected 60km/h design speed environment. On-site observations have indicated that in excess of 150m is available.
- The ultimate C2 Structure Plan also identified the Road 10 eventually being formed with the planned C2 Collector Road and other wider transport network connections. The Waipa District Council Long Term Plan (LTP) identifies a range of staged transport network improvement projects including staged implementation of the C2 Collector Road and roundabout intersection with Cambridge Rd, being the long term strategic solution for the growth area and indicatively expected to be budgeted for 2021-2023 financial years. Other key local project allocations include:
    - C2/C3 Collector Roads and Green Belt Connection – Land: $11.15M, 21/22-28/29;
    - C2 & C3 Structure Plan roading: $25.14M, 21/22 – 30/31;
    - C1 Structure Plan Roading: $1.5975M, 24/25-26/27; as well as a range of
    - Urbanisation and cycleway project undertakings for Hamilton Road, Victoria Road and Kelly Road.
- Detailed engineering design of the intersections are recommended to be subject to an independent road safety audit. The safety audit recommendations shall be resolved to the satisfaction of Waipa District Council prior to the commencement of physical works on-site;
- A temporary traffic management plan shall be prepared by a suitably qualified person and submitted to Council for approval prior to the commencement of physical works on-site.

On the matters of design layout, operation and performance; and based on the assessments described above, it is concluded the proposed location of Road 10 is aligned with the Structure Plan operational intentions and is able to be safely formed and located as proposed.

<u>In response to item 31:</u>
Capacity performance of the intersection of Road 8 and Road 10 with Cambridge Road

The operational performance expectations for the proposed Road 8 and Road 10 intersections are assessed as follows.

Previous technical assessments of the potential for local trip generation due to both the C2 area as well as other growth areas generating demand effects on the Cambridge Road corridor have been determined by BBO Consultants for Council in consultation with Stantec acting for the applicant. The underlying and broader growth demands have also been factored in to forecast traffic demand expectations out to 2031 on the frontage and through the intersections in a consistent way with the basis of prior demand forecasts for the Structure Plan areas.

Those traffic generation assumptions for the C2 growth area have previously been based on the full site being developed as residential living. Current proposals however have identified that the "Super-Lot Site" proposed by 3Ms is to be developed as retirement living. A refined forecast of local traffic demands has therefore been developed to reflect the current proposal and development expectations.

The trip generation demand assessments are attached as **Appendix B** and are summarised as follows:

- Scenario 1 describes an assessment based on substantial development of the applicant's proposed C2 area on the 2021 transport network;
- Scenario 2 described full development of the applicant's proposed C2 Structure Plan area on the 2021 transport network; and
- Scenario 3 describes full development of the entire C1, C2 and C3 Structure Plan areas on a connected 2031 transport network.

The corresponding AM and PM distributed peak period turning demands at both Road 8 and Road 10 intersections are set out at **Appendix C**.

Modelled intersection performance characteristics for both the AM and PM peak periods for each of the Scenarios are set out at **Appendix D**.

By way of a summary, the following key results have been determined for the most critical of the intersection movements, the right turn from the C2 area onto Cambridge Road.

Table 1: Road 8 Intersection Right Turn Out Performance Summary

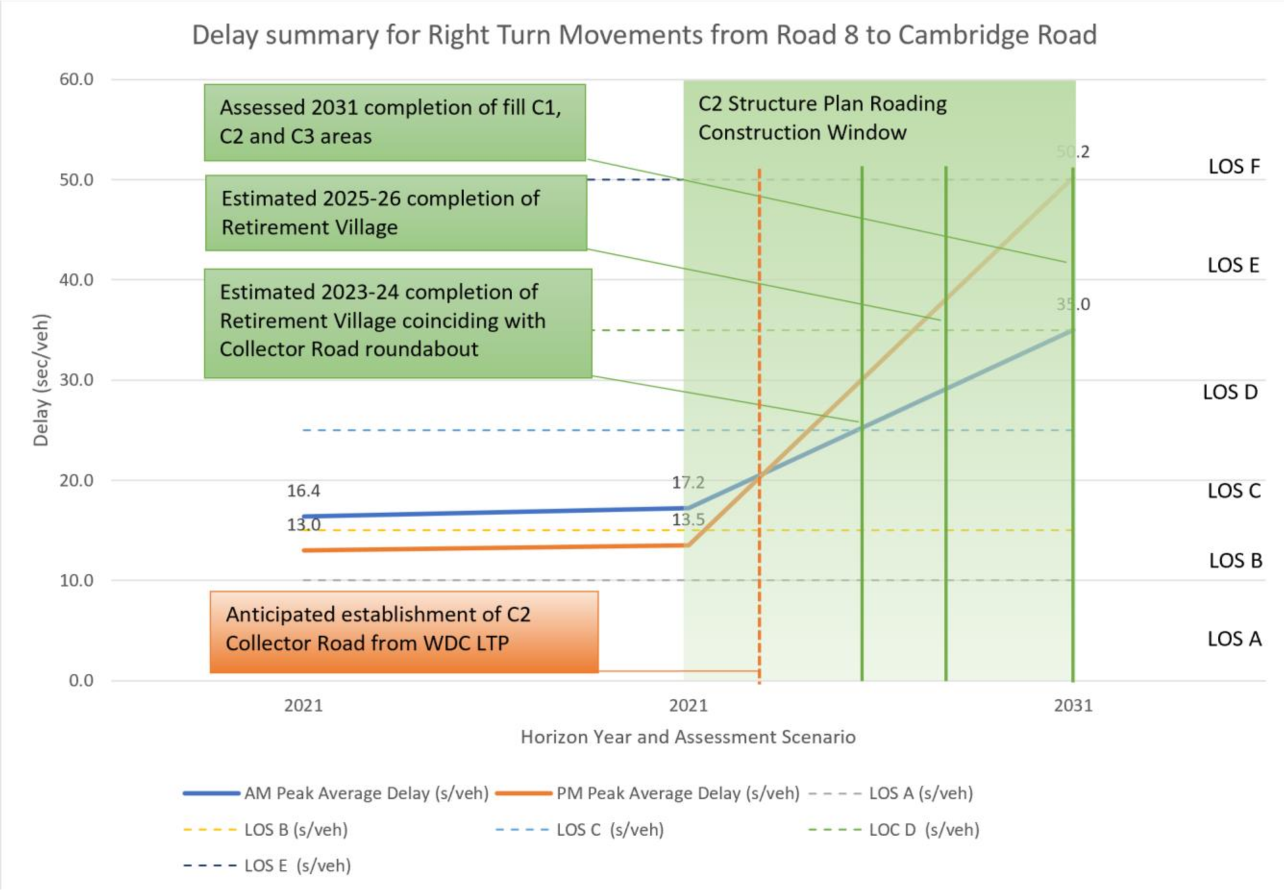
Intersection	Development Stage Description	Year	AM Peak Average Delay (s/veh)	PM Peak Average Delay (s/veh)
Road 8 - RT-out	C2 retirement village, 100 new +47 existing dwellings + school	2021	16.4	13.0
Road 8 - RT-out	Further C2 76 dwellings to full development scenario	2021	17.2	13.5
Road 8 - RT-out	Full C2 with Full C1 and C3 plus background growth	2031	35.0	50.2

Table 2: Road 10 Intersection Right Turn Out Performance Summary

Intersection	Development Stage Description	Year	AM Peak Average Delay (s/veh)	PM Peak Average Delay (s/veh)
Road 10 - RT-out	C2 retirement village, 100 new +47 existing dwellings + school	2021	18.1	13.6
Road 10 - RT-out	Further C2 76 dwellings to full development scenario	2021	19.7	14.4
Road 8 - RT-out	Full C2 with Full C1 and C3 plus background growth	2031	42.9	48.5

These results have been further accumulated into a graphical form to show the expected network performance together with other changes on the transport network.

Figure 1: Road 8 Intersection with Cambridge Road - Graphical Summary of Right Turn Out Performance



Delay summary for Right Turn Movements from Road 8 to Cambridge Road

Assessed 2031 completion of fill C1, C2 and C3 areas

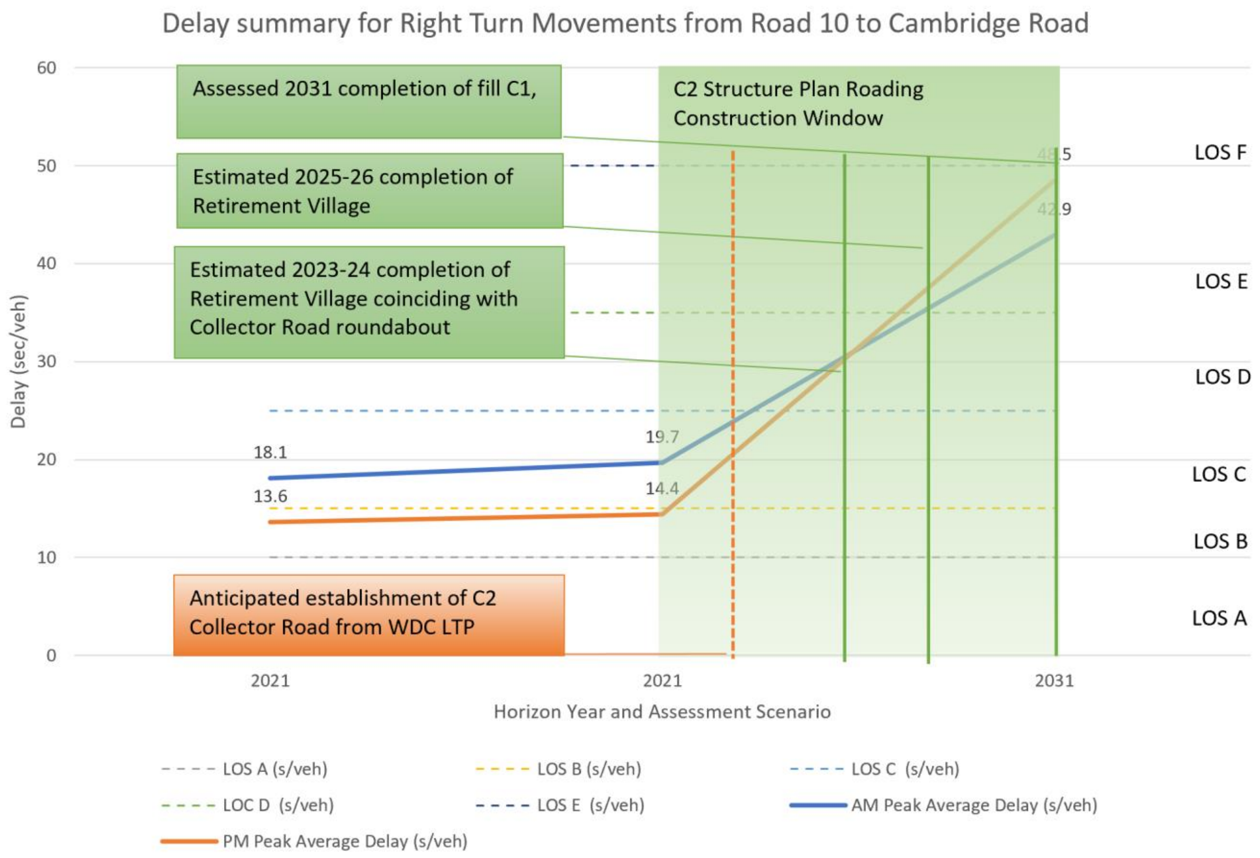
Estimated 2025-26 completion of Retirement Village

Estimated 2023-24 completion of Retirement Village coinciding with Collector Road roundabout

Anticipated establishment of C2 Collector Road from WDC LTP

C2 Structure Plan Roading Construction Window

Figure 2: Road 10 Intersection with Cambridge Road - Graphical Summary of Right Turn Out Performance



The data and the graphs show the following features:

- Right turn out performance results for both intersections across all three scenarios;
- Results for both the AM and PM peak periods (s/veh);
- The green band on the graph highlights the 2021-31 period across which the range of works, provisioned within the Waipa District LTP, are expected to occur together with formation of the local road networks and connections comprising the C1, C2 and C3 Structure Plan Growth Cells;
- The green text boxes together with the vertical lines are intended to provide some indicative practical representation of the timeframe by which the C2 and surrounding C1/C3 development may be expected to be progressed / completed, having regard for construction timeframes. Importantly, this does not suggest a proposed development staging, but rather provides some practical context based on what is apparent at this time. It demonstrates alignment between strategic transport network planning and proposed development staging.

The results shown in the data sets and within the two graphs (for both Road 8 and 10 intersections) can be summarised as follows:

- Scenario 1, part development of the applicant's C2 growth area (refer **Appendix B**) indicates delay expectations in the range 13.0 to 18.1 s/veh on the right turn out movements, assuming it was to occur in 2021. This represents an operating level of service performance in the range LOS B to C, a relative efficient but not unencumbered level of service;

- Scenario 2 represents full development of the applicant's proposal, as if it were loaded onto the 2021 network. Again, with delay expectations for the right turn out movement in the range 13.5 to 19.7 s/veh (LOS B to C) an acceptably efficient level of performance is expected for this movement;
- Scenario 3 not only introduces 10 years of wider District growth demands, it further loads potential future and full development expectations for the remaining C2 as well as the C1 and C3 growth areas. The resulting change in traffic demands and local road connected network distributions suggests performance for the right turn out movement in the range LOS E (AM peak) to F (PM peak). In this regard, it is evident the applicant's C2 development proposal alone will readily be able to be accommodated.

  The graphs for Scenario 3 also represent full future trip demands from these growth cells, the results indicate some peak period delay effects, particularly in the PM period. The orange dashed line on the graphs indicates the expected early introduction, through the LTP, of the C2 Collector Road and Roundabout, which will provide the primary access/egress movement capacity for the C2 area. It can therefore be concluded that the applicant's proposal with respect to both Road 8 and Road 10 intersections will perform acceptably at the level of development intensity proposed and with the anticipated local road connectivity.

Conclusions and Recommendations.

On the bases of these assessments the following conclusions are made:

- On the matters of design layout, operation and performance; and based on the assessments described above, it is concluded the proposed location of Road 10 is aligned with the Structure Plan operational intentions and is able to be safely formed and located as proposed; and
- The capacity and performance expectations for both Road 8 and Road 10 will be sufficient and appropriately timed to safely provide for the activities proposed, including in the first couple of years while construction is progressed and prior to the C2 Collector Road connection and roundabout.

Yours sincerely

Apeldoorn, Mark
Practice Leader: Transport Advisory
Stantec New Zealand

Appendix A: Plan showing the indicative arrangement for the Cambridge Road intersections.

NOT FOR CONSTRUCTION

REV	REVISIONS	DRN	CHK	APP	DATE
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
B	EXTEND PROPOSED MEDIAN	MS	-	-	06.04.2021
A	DRAWING CREATED	TL	MS	-	30.3.2021

SURVEYED		
DESIGNED	-	-
DRAWN	MATTHEW SPROULL	06.04.2021
CAD REVIEW	-	-
DESIGN CHECK		
DESIGN REVIEW		
APPROVED		
PROF REGISTRATION:		

Stantec

Client:

CAMBRIDGE ROAD RIGHT TURN BAYS
-

CONCEPT DRAWING
-

Status Stamp **WORKING PLOT**

Date Stamp **06.04.2021**

Scales 1:500

Drawing No. 310204689-01-B-1

Rev. B

C:\Users\msproull\OneDrive - Stantec Office 365\Documents\31020XXXX - 3MS For MA\Right_turn_bays - Standard\310204689-01-B.dwg

Appendix B: Road 8 and 10 Trip Generation Demand Forecasts

2021 Part development in C2 Structure Plan area as follows:

According to the 2017 Structure Plan Transport Assessment, C2 and C3 yield is expected to be 1500-2000hh. Therefore adopt 1750hh. 400hh of these is in C3. Therefore, C2 = 1350hh

C1 yield expected to be 275-375hh. Therefore assume 325hh

Neighbourhood centre in C1 is 2.6ha. Assume 40% site coverage and 15 trips per peak pm hour generation

Mode Share Targets by 2041			**Expected = "1", Aspirational = "2"**
Active Modes	**10%**	Expected	1
Public Transport Internally	**5%**	Expected	1
Public Transport Externally	**10%**	Expected	1
Local Trips percentage split	**15%**	Within Zone C1, C2 or C3)	
External trips percentage split	**85%**		

Distribution split

	AM		PM	
	In	Out	In	Out
Residential	20%	80%	55%	45%
School	55%	45%	45%	55%
School	70%	30%	45%	55%

2021 Part Development in C2

C1 2031

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips / hr	External Trips/hr	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit					Active Modes	PT	Veh	AM			PM		
				AM	PM	15%	85%	All Peaks			In	Out	Total	In	Out	Total
Residential		1.2	Dwelling	0	0	0	0	0	0	0	**0**	**0**	0	**0**	**0**	0
						0			**0**							
Neighbourhood centre		8	100 sq.m/GFA	-	0	100%	0%	0	0	0	**0**	**0**	0	**0**	**0**	0

C2 2031

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips / hr	External Trips/hr	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit					Active Modes	PT	Veh	AM			PM		
				AM	PM	15%	85%	All Peaks			In	Out	Total	In	Out	Total
3Ms	100	1.2	Dwelling	120	120	18	102	2	11	107	**21**	**86**	107	**59**	**48**	107
Ryman	80	0.4	Care beds	32	32	5	27	0	3	29	**6**	**23**	29	**16**	**13**	29
Ryman	46	0.4	Assisted Suites	18	18	3	16	0	2	16	**3**	**13**	16	**9**	**7**	16
Ryman	202	0.5	Townhouses	101	101	15	86	2	9	90	**18**	**72**	90	**50**	**41**	90
other		1.2	Dwelling	0	0	0	0	0	0	0	**0**	**0**	0	**0**	**0**	0
existing households	47	1.2	Dwelling	56	56	8	48	1	5	50	**10**	**40**	50	**28**	**23**	50
	475			328	328	**328**			**328**		59	234	293	161	132	293
Primary School	300	2	Pupils	600	60	90	510	9	56	536	**295**	**241**	536	**23**	**29**	52
						600			**600**							

C3 2021

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips / hr	External Trips/hr	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit					Active Modes	PT	Veh	AM			PM		
				AM	PM	15%	85%	All Peaks			In	Out	Total	In	Out	Total
Chartwell and St Peters		1.2	dwelling	0	0	0	0	0	0	0	**0**	**0**	0	**0**	**0**	0
Te Awa Lifecare	32	0.4	beds	13	13	2	11	0	1	11	**16**	**16**	32	**16**	**16**	32
Te Awa Lifecare	12	0.4	Unit	5	5	1	4	0	0	4						
Te Awa Lifecare	11	0.5	apartments	6	6	1	5	0	1	5						
Te Awa Lifecare	25	0.5	villas	13	13	2	11	0	1	11						

2021 Full Development in Applicant's C2 Structure Plan Area

According to the 2017 Structure Plan Transport Assessment, C2 and C3 yield is expected to be 1500-2000hh. Therefore adopt 1750hh. 400hh of these is in C3. Therefore, C2 = 1350hh
C1 yield expected to be 275-375hh. Therefore assume 325hh
Neighbourhood centre in C1 is 2.6ha. Assume 40% site coverage and 15 trips per peak pm hour generation

Mode Share Targets by 2041			**Expected = "1", Aspirational = "2"**
Active Modes	**10%**	Expected	1
Public Transport Internally	**5%**	Expected	1
Public Transport Externally	**10%**	Expected	1
Local Trips percentage split	**15%**	Within Zone C1, C2 or C3)	
External trips percentage split	**85%**		

	Distribution split			
	AM		PM	
	In	Out	In	Out
Residential	20%	80%	55%	45%
School	55%	45%	45%	55%
School	70%	30%	45%	55%

2021 Baseline - Full Development in 3Ms C2 Area

C1 2031

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips	External	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit			/ hr	Trips/hr	Active Modes	PT	Veh	AM			PM		
				AM	PM	15%	85%	All Peaks			In	Out	Total	In	Out	Total
Residential		1.2	Dwelling	0	0	0	0	0	0	0	**0**	**0**	0	**0**	**0**	0
						0		**0**								
Neighbourhood centre		8	100 sq.m/GFA	-	0	100%	0%	0	0	0	**0**	**0**	0	**0**	**0**	0

C2 2031

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips	External	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit			/ hr	Trips/hr	Active Modes	PT	Veh	AM			PM		
				AM	PM	15%	85%	All Peaks			In	Out	Total	In	Out	Total
3Ms	276	1.2	Dwelling	331	331	50	282	5	31	296	**59**	**236**	296	**163**	**133**	296
Ryman	80	0.4	Care beds	32	32	5	27	0	3	29	**6**	**23**	29	**16**	**13**	29
Ryman	46	0.4	Assisted Suites	18	18	3	16	0	2	16	**3**	**13**	16	**9**	**7**	16
Ryman	202	0.5	Townhouses	101	101	15	86	2	9	90	**18**	**72**	90	**50**	**41**	90
other		1.2	Dwelling	0	0	0	0	0	0	0	**0**	**0**	0	**0**	**0**	0
existing households	47	1.2	Dwelling	56	56	8	48	1	5	50	**10**	**40**	50	**28**	**23**	50
	651			539	539	**539**		**539**			96	385	481	265	216	481
Primary School	300	2	Pupils	600	60	90	510	9	56	536	**295**	**241**	536	**23**	**29**	52
						600		**600**								

C3 2021

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips	External	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit			/ hr	Trips/hr	Active Modes	PT	Veh	AM			PM		
				AM	PM	15%	85%	All Peaks			In	Out	Total	In	Out	Total
Chartwell and St Peters		1.2	dwelling	0	0	0	0	0	0	0	**0**	**0**	0	**0**	**0**	0
Te Awa Lifecare	32	0.4	beds	13	13	2	11	0	1	11	**16**	**16**	32	**16**	**16**	32
Te Awa Lifecare	12	0.4	Unit	5	5	1	4	0	0	4						
Te Awa Lifecare	11	0.5	apartments	6	6	1	5	0	1	5						
Te Awa Lifecare	25	0.5	villas	13	13	2	11	0	1	11						

2031 Full Development in C1, C2 and C3 Structure Plan Areas with Background Growth

According to the 2017 Structure Plan Transport Assessment, C2 and C3 yield is expected to be 1500-2000hh. Therefore adopt 1750hh. 400hh of these is in C3. Therefore, C2 = 1350hh

C1 yield expected to be 275-375hh. Therefore assume 325hh

Neighbourhood centre in C1 is 2.6ha. Assume 40% site coverage and 15 trips per peak pm hour generation

Mode Share Targets by 2041			**Expected = "1", Aspirational = "2"**
Active Modes	**10%**	Expected	1
Public Transport Internally	**5%**	Expected	1
Public Transport Externally	**10%**	Expected	1

Distribution split				
	AM		PM	
	In	Out	In	Out
Residential	20%	80%	55%	45%
School	55%	45%	45%	55%
School	70%	30%	45%	55%

Local Trips percentage split	**60%**	Within Zone C1, C2 or C3)
External trips percentage split	**40%**	

2041Baseline

C1 2031

Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips	External	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit			/ hr	Trips/hr	Active Modes	PT	Veh	AM			PM		
				AM	PM	60%	40%	All Peaks			In	Out	Total	In	Out	Total
Residential	325	1.2	Dwelling	390	390	234	156	23	27	339	**68**	**271**	339	**187**	**153**	339
						390			**390**							
Neighbourhood centre	2.6	8	100 sq.m/GFA	-	2080	100%	0%	208	104	1768	**1238**	**530**	1768	**796**	**972**	1768

C2 2031

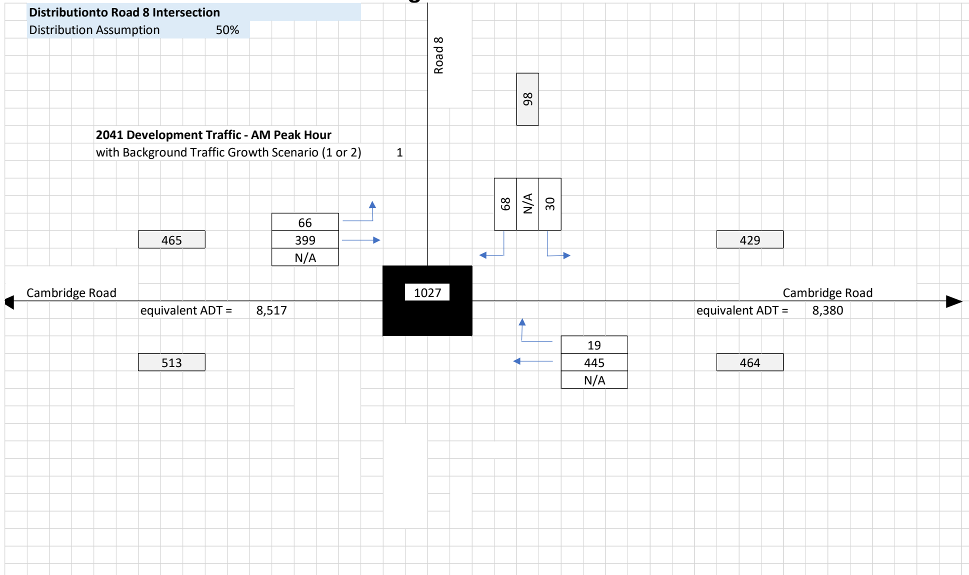
Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips	External	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit			/ hr	Trips/hr	Active Modes	PT	Veh	AM			PM		
				AM	PM	60%	40%	All Peaks			In	Out	Total	In	Out	Total
3Ms	276	1.2	Dwelling	331	331	199	132	20	23	288	**58**	**231**	288	**158**	**130**	288
Ryman	80	0.4	Care beds	32	32	19	13	2	2	28	**6**	**22**	28	**15**	**13**	28
Ryman	46	0.4	Assisted Suites	18	18	11	7	1	1	16	**3**	**13**	16	**9**	**7**	16
Ryman	202	0.5	Townhouses	101	101	61	40	6	7	88	**18**	**70**	88	**48**	**40**	88
other	703	1.2	Dwelling	844	844	506	337	51	59	734	**147**	**587**	734	**404**	**330**	734
existing households	47	1.2	Dwelling	56	56	34	23	3	4	49	**10**	**39**	49	**27**	**22**	49
	1354			1383	1383	**1383**			**1383**		241	962	1203	662	541	1203
Primary School	300	2	Pupils	600	60	360	240	36	42	522	**287**	**235**	522	**23**	**29**	52
						600			**600**							

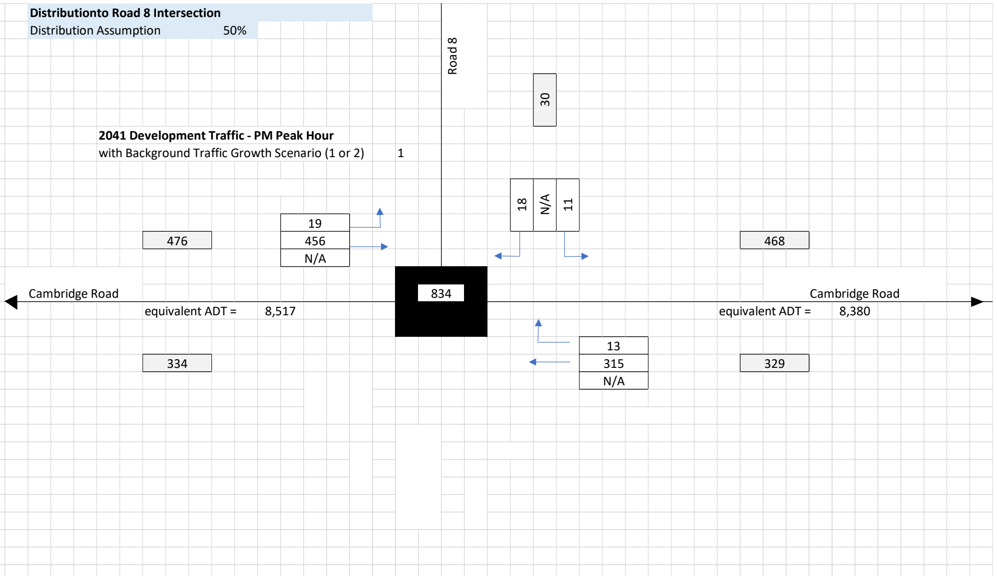
C3 2021

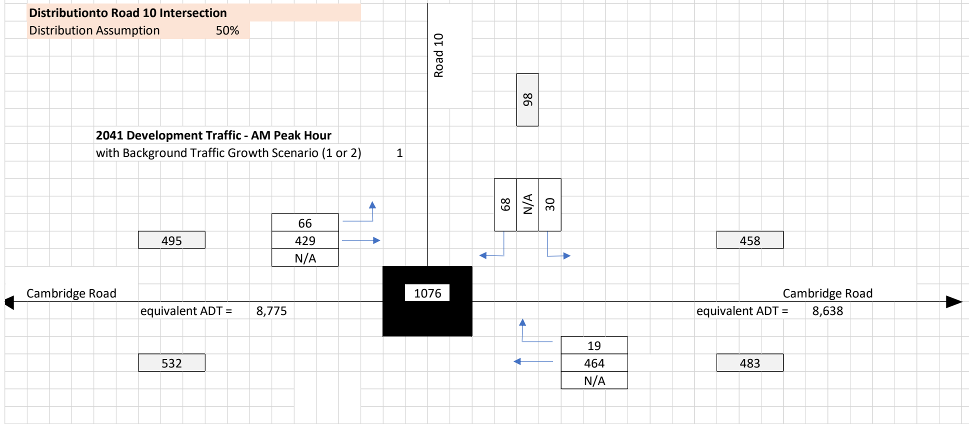
Land use	Quantity	Peak Hour Trip Rate		Peak Hour Trips		Local Trips	External	Mode Share			Peak Hour Vehicle Trips					
		Rate	Unit			/ hr	Trips/hr	Active Modes	PT	Veh	AM			PM		
				AM	PM	60%	40%	All Peaks			In	Out	Total	In	Out	Total
Chartwell and St Peters	400	1.2	dwelling	480	480	288	192	29	34	418	**84**	**334**	418	**230**	**188**	418
Te Awa Lifecare	32	0.4	beds	13	13	8	5	1	1	11	**15**	**15**	31	**15**	**15**	31
Te Awa Lifecare	12	0.4	Unit	5	5	3	2	0	0	4						
Te Awa Lifecare	11	0.5	apartments	6	6	3	2	0	0	5						
Te Awa Lifecare	25	0.5	villas	13	13	8	5	1	1	11						

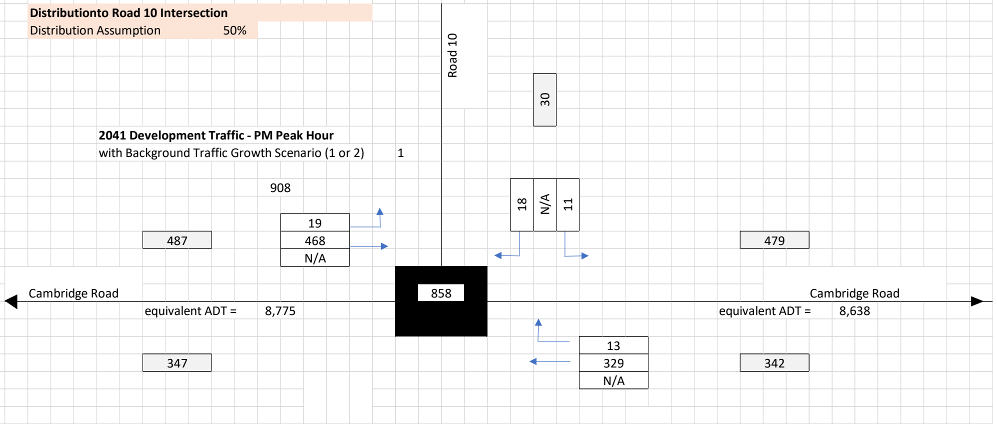
Appendix C: The corresponding AM and PM distributed peak period turning demands at both Road 8 and Road 10 intersections

Scenario 1: Turning Movements

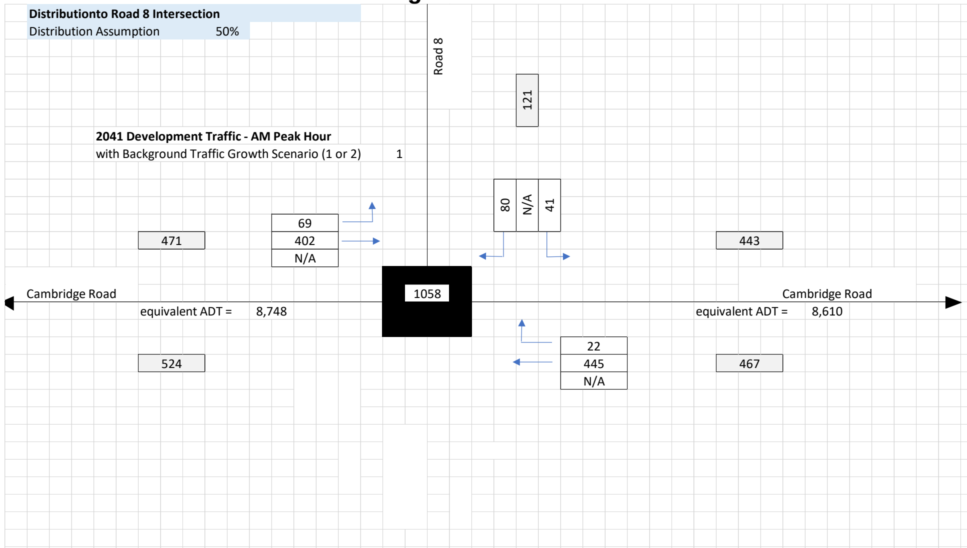


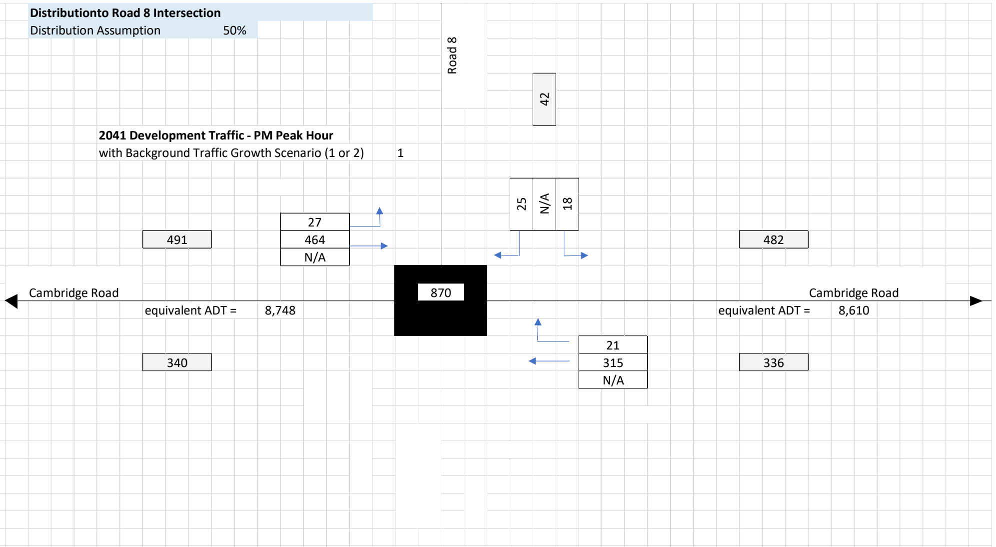


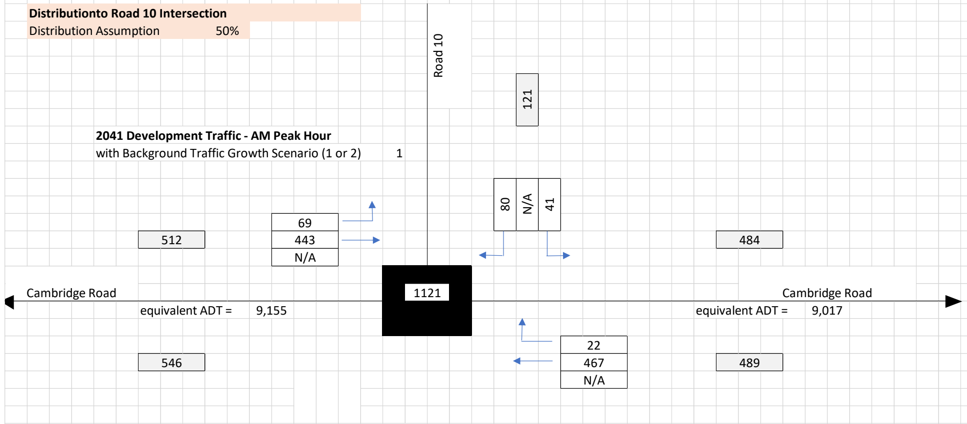


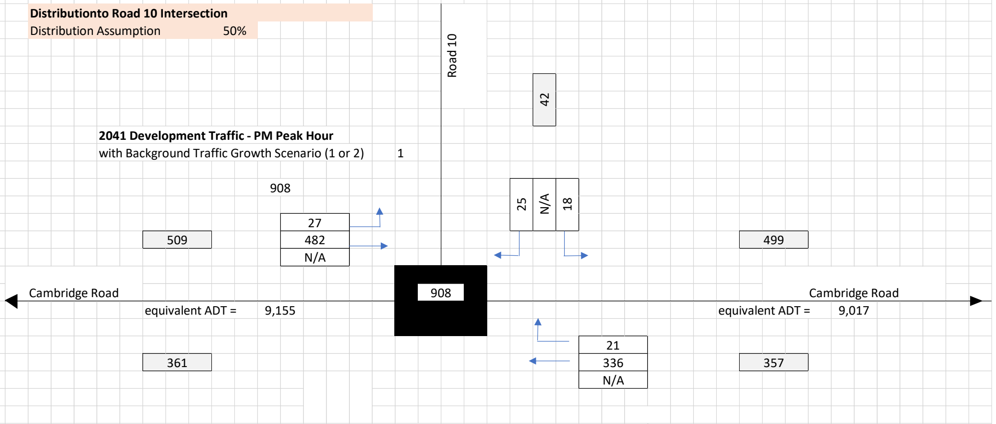


Scenario 2: Turning Movements

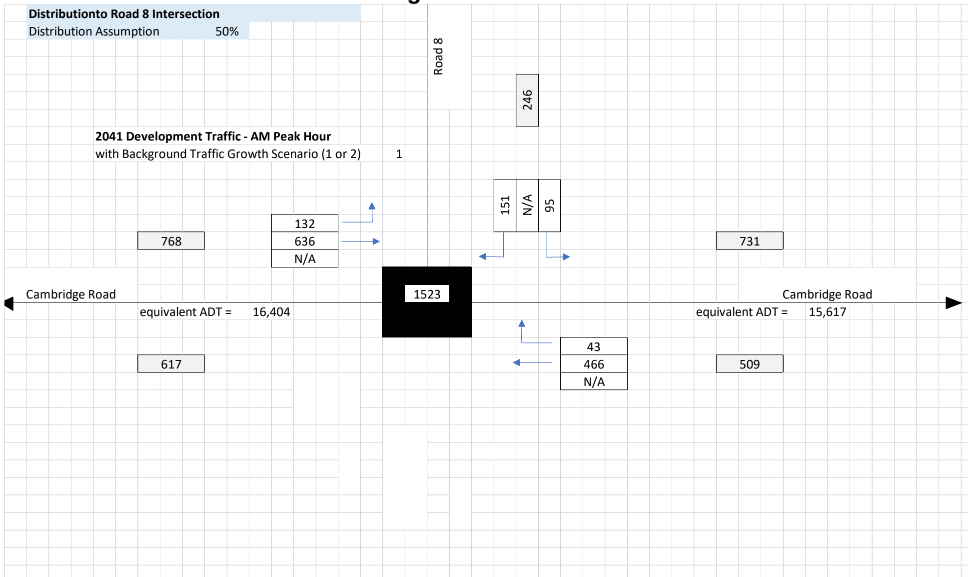


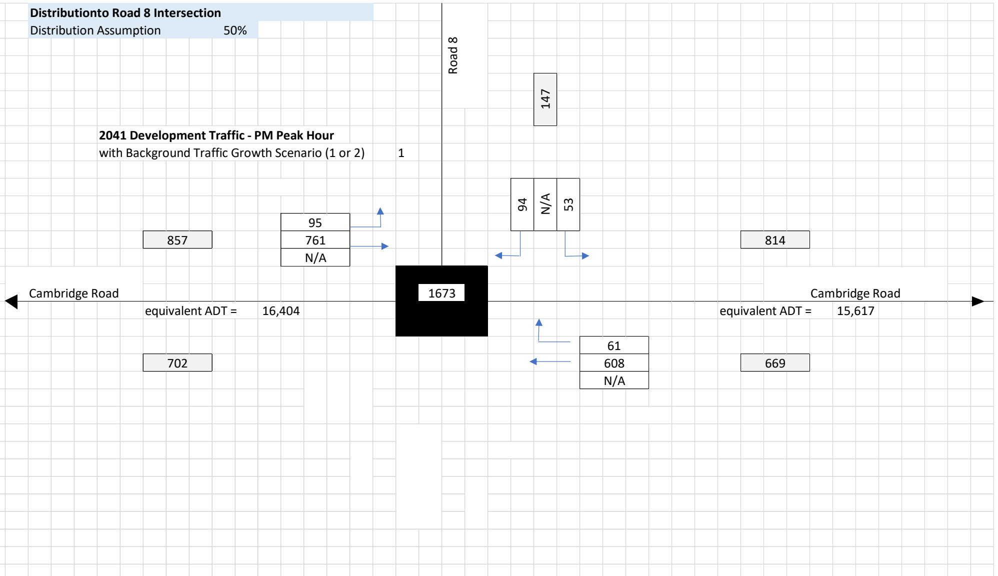


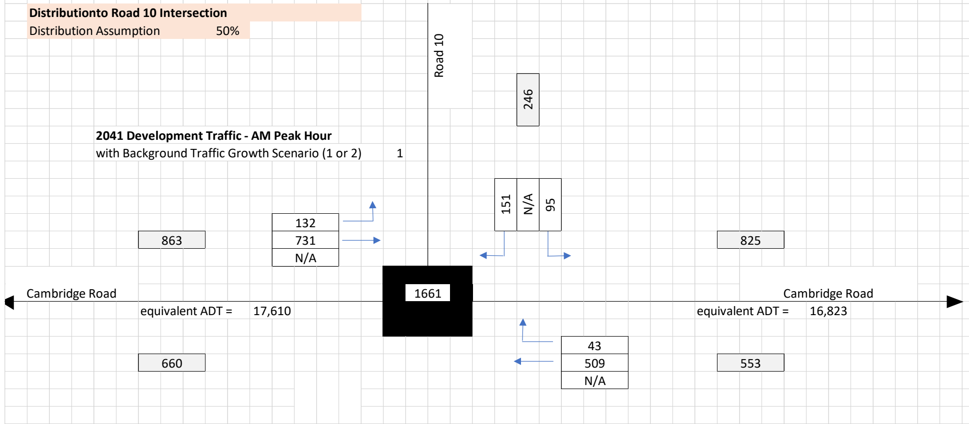


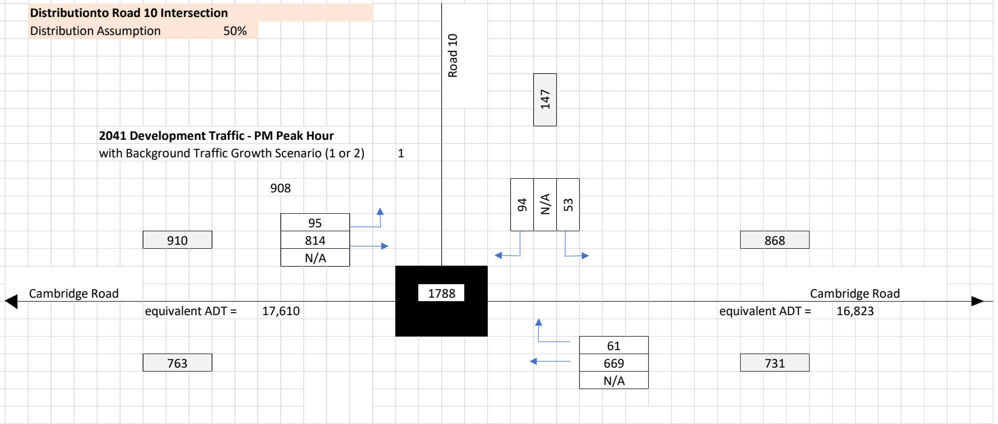


Scenario 3: Turning Movements



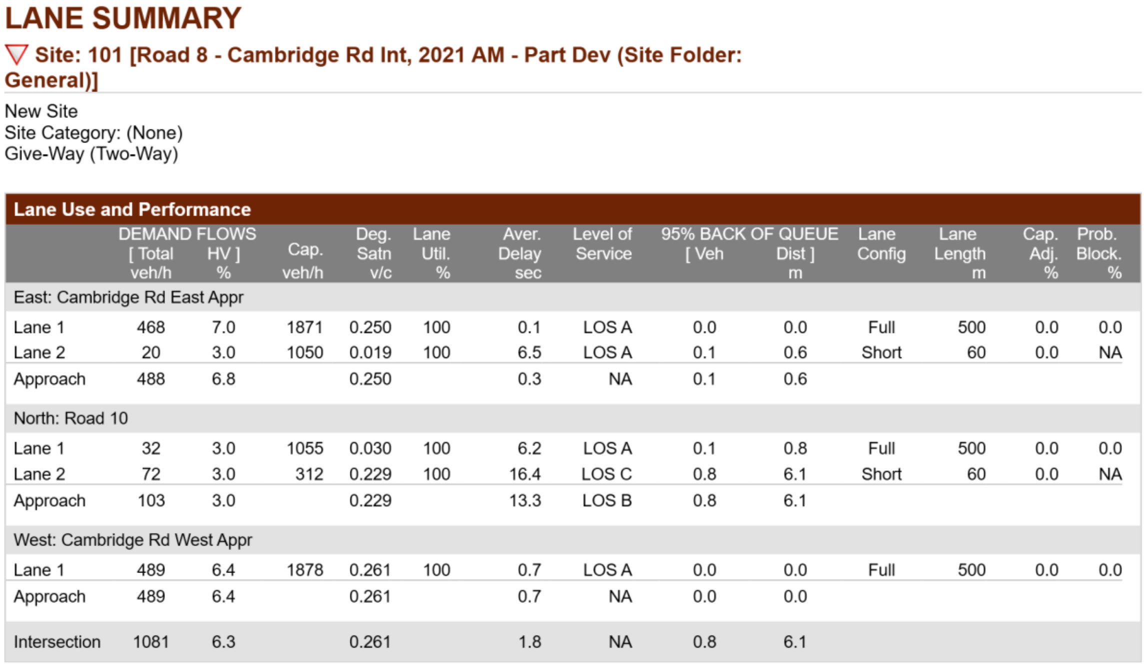






Appendix D: Intersection Modelling Results

Scenario 1: Road 8 and 10, AM and PM, 2021 Results



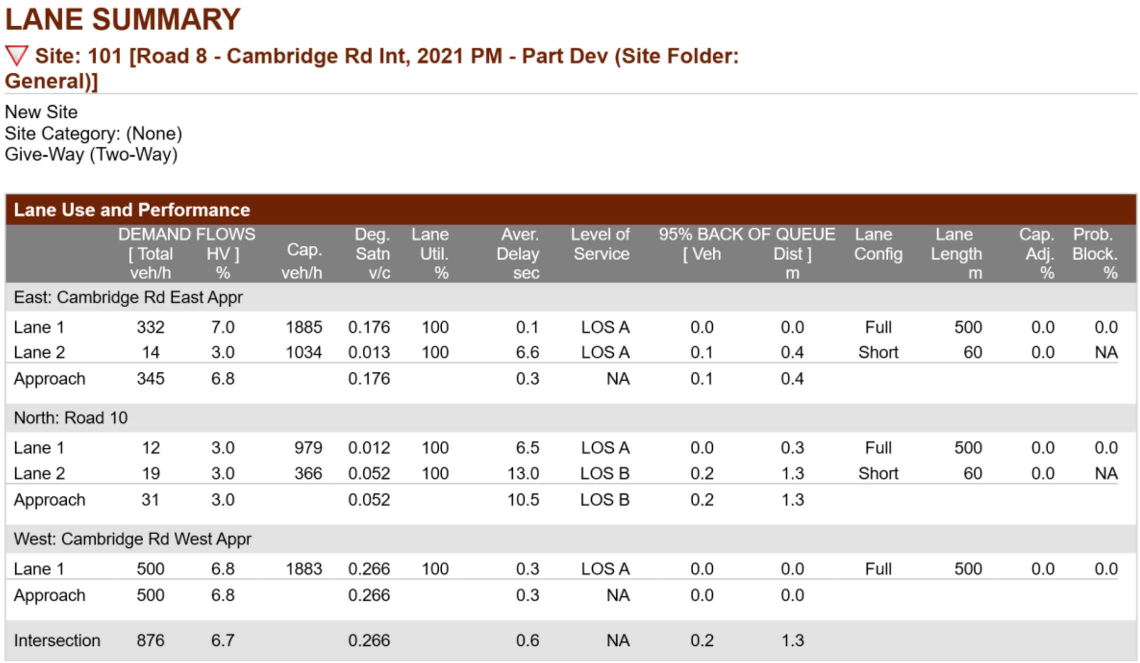
LANE SUMMARY

▽ Site: 101 [Road 8 - Cambridge Rd Int, 2021 AM - Part Dev (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	468	7.0	1871	0.250	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	20	3.0	1050	0.019	100	6.5	LOS A	0.1	0.6	Short	60	0.0	NA
Approach	488	6.8		0.250		0.3	NA	0.1	0.6				
North: Road 10													
Lane 1	32	3.0	1055	0.030	100	6.2	LOS A	0.1	0.8	Full	500	0.0	0.0
Lane 2	72	3.0	312	0.229	100	16.4	LOS C	0.8	6.1	Short	60	0.0	NA
Approach	103	3.0		0.229		13.3	LOS B	0.8	6.1				
West: Cambridge Rd West Appr													
Lane 1	489	6.4	1878	0.261	100	0.7	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	489	6.4		0.261		0.7	NA	0.0	0.0				
Intersection	1081	6.3		0.261		1.8	NA	0.8	6.1				



LANE SUMMARY

▽ Site: 101 [Road 8 - Cambridge Rd Int, 2021 PM - Part Dev (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	332	7.0	1885	0.176	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	14	3.0	1034	0.013	100	6.6	LOS A	0.1	0.4	Short	60	0.0	NA
Approach	345	6.8		0.176		0.3	NA	0.1	0.4				
North: Road 10													
Lane 1	12	3.0	979	0.012	100	6.5	LOS A	0.0	0.3	Full	500	0.0	0.0
Lane 2	19	3.0	366	0.052	100	13.0	LOS B	0.2	1.3	Short	60	0.0	NA
Approach	31	3.0		0.052		10.5	LOS B	0.2	1.3				
West: Cambridge Rd West Appr													
Lane 1	500	6.8	1883	0.266	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	500	6.8		0.266		0.3	NA	0.0	0.0				
Intersection	876	6.7		0.266		0.6	NA	0.2	1.3				

LANE SUMMARY

▽ Site: 101 [Road 10 - Cambridge Rd Int, 2021 AM - Part Dev (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	488	7.0	1871	0.261	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	20	3.0	1006	0.020	100	6.7	LOS A	0.1	0.6	Short	60	0.0	NA
Approach	508	6.8		0.261		0.4	NA	0.1	0.6				
North: Road 10													
Lane 1	32	3.0	1015	0.031	100	6.4	LOS A	0.1	0.8	Full	500	0.0	0.0
Lane 2	72	3.0	285	0.251	100	18.1	LOS C	0.9	6.7	Short	60	0.0	NA
Approach	103	3.0		0.251		14.5	LOS B	0.9	6.7				
West: Cambridge Rd West Appr													
Lane 1	521	6.5	1878	0.277	100	0.7	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	521	6.5		0.277		0.7	NA	0.0	0.0				
Intersection	1133	6.3		0.277		1.8	NA	0.9	6.7				

LANE SUMMARY

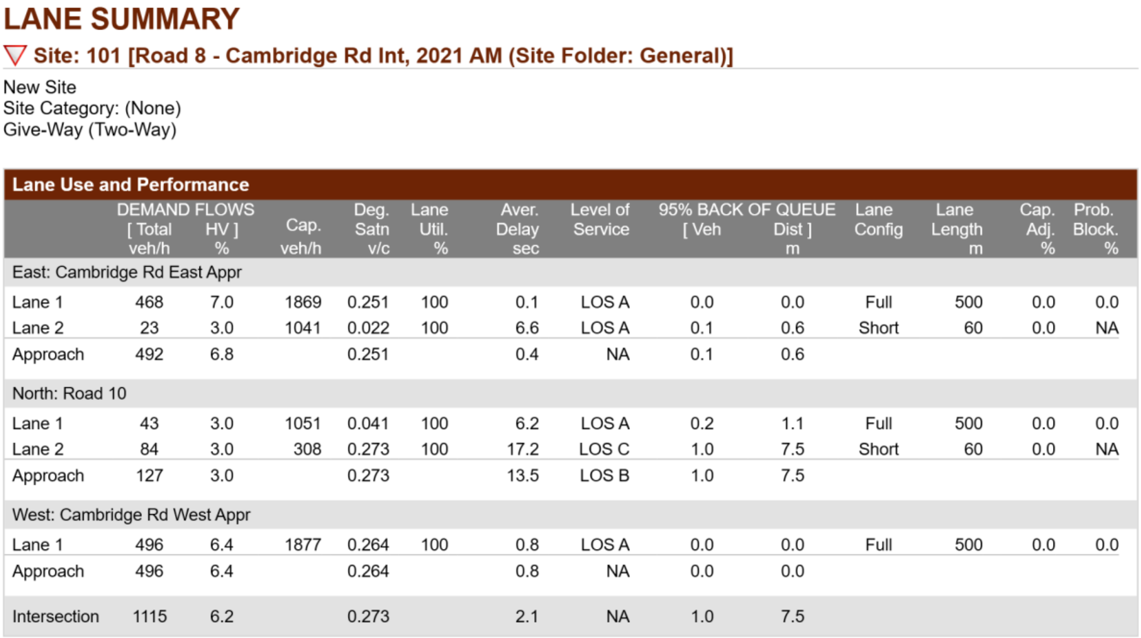
▽ Site: 101 [Road 10 - Cambridge Rd Int, 2021 PM - Part Dev (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	346	7.0	1885	0.184	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	14	3.0	1016	0.013	100	6.7	LOS A	0.1	0.4	Short	60	0.0	NA
Approach	360	6.8		0.184		0.3	NA	0.1	0.4				
North: Road 10													
Lane 1	12	3.0	963	0.012	100	6.5	LOS A	0.0	0.3	Full	500	0.0	0.0
Lane 2	19	3.0	349	0.054	100	13.6	LOS B	0.2	1.3	Short	60	0.0	NA
Approach	31	3.0		0.054		10.9	LOS B	0.2	1.3				
West: Cambridge Rd West Appr													
Lane 1	513	6.8	1883	0.272	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	513	6.8		0.272		0.3	NA	0.0	0.0				
Intersection	903	6.7		0.272		0.6	NA	0.2	1.3				

Scenario 2: Road 8 and 10, AM and PM, 2021 Results



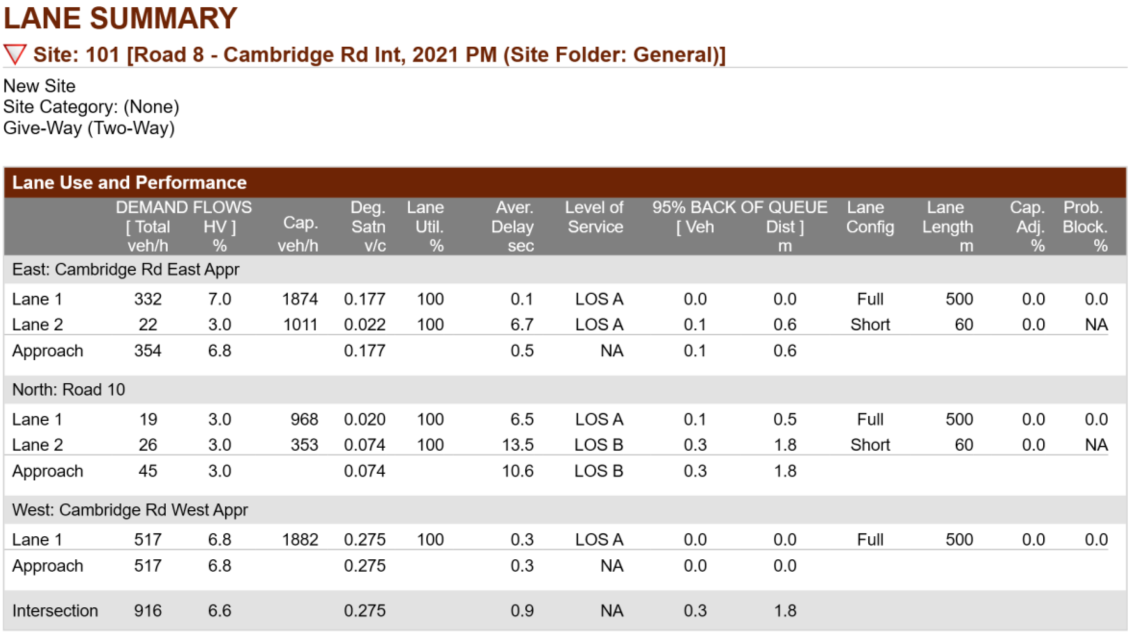
LANE SUMMARY

▽ Site: 101 [Road 8 - Cambridge Rd Int, 2021 AM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	468	7.0	1869	0.251	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	23	3.0	1041	0.022	100	6.6	LOS A	0.1	0.6	Short	60	0.0	NA
Approach	492	6.8		0.251		0.4	NA	0.1	0.6				
North: Road 10													
Lane 1	43	3.0	1051	0.041	100	6.2	LOS A	0.2	1.1	Full	500	0.0	0.0
Lane 2	84	3.0	308	0.273	100	17.2	LOS C	1.0	7.5	Short	60	0.0	NA
Approach	127	3.0		0.273		13.5	LOS B	1.0	7.5				
West: Cambridge Rd West Appr													
Lane 1	496	6.4	1877	0.264	100	0.8	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	496	6.4		0.264		0.8	NA	0.0	0.0				
Intersection	1115	6.2		0.273		2.1	NA	1.0	7.5				



LANE SUMMARY

▽ Site: 101 [Road 8 - Cambridge Rd Int, 2021 PM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	332	7.0	1874	0.177	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	22	3.0	1011	0.022	100	6.7	LOS A	0.1	0.6	Short	60	0.0	NA
Approach	354	6.8		0.177		0.5	NA	0.1	0.6				
North: Road 10													
Lane 1	19	3.0	968	0.020	100	6.5	LOS A	0.1	0.5	Full	500	0.0	0.0
Lane 2	26	3.0	353	0.074	100	13.5	LOS B	0.3	1.8	Short	60	0.0	NA
Approach	45	3.0		0.074		10.6	LOS B	0.3	1.8				
West: Cambridge Rd West Appr													
Lane 1	517	6.8	1882	0.275	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	517	6.8		0.275		0.3	NA	0.0	0.0				
Intersection	916	6.6		0.275		0.9	NA	0.3	1.8				

LANE SUMMARY

▽ Site: 101 [Road 10 - Cambridge Rd Int, 2021 AM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS		Cap.	Deg.	Lane	Aver.	Level of	95% BACK OF QUEUE		Lane	Lane	Cap.	Prob.
	[Total veh/h	HV] %	veh/h	Satn v/c	Util. %	Delay sec	Service	[Veh	Dist] m	Config	Length m	Adj. %	Block. %
East: Cambridge Rd East Appr													
Lane 1	492	7.0	1868	0.263	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	23	3.0	982	0.024	100	6.9	LOS A	0.1	0.7	Short	60	0.0	NA
Approach	515	6.8		0.263		0.4	NA	0.1	0.7				
North: Road 10													
Lane 1	43	3.0	996	0.043	100	6.5	LOS A	0.2	1.2	Full	500	0.0	0.0
Lane 2	84	3.0	273	0.308	100	19.7	LOS C	1.2	8.5	Short	60	0.0	NA
Approach	127	3.0		0.308		15.2	LOS C	1.2	8.5				
West: Cambridge Rd West Appr													
Lane 1	539	6.5	1878	0.287	100	0.7	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	539	6.5		0.287		0.7	NA	0.0	0.0				
Intersection	1181	6.2		0.308		2.1	NA	1.2	8.5				

LANE SUMMARY

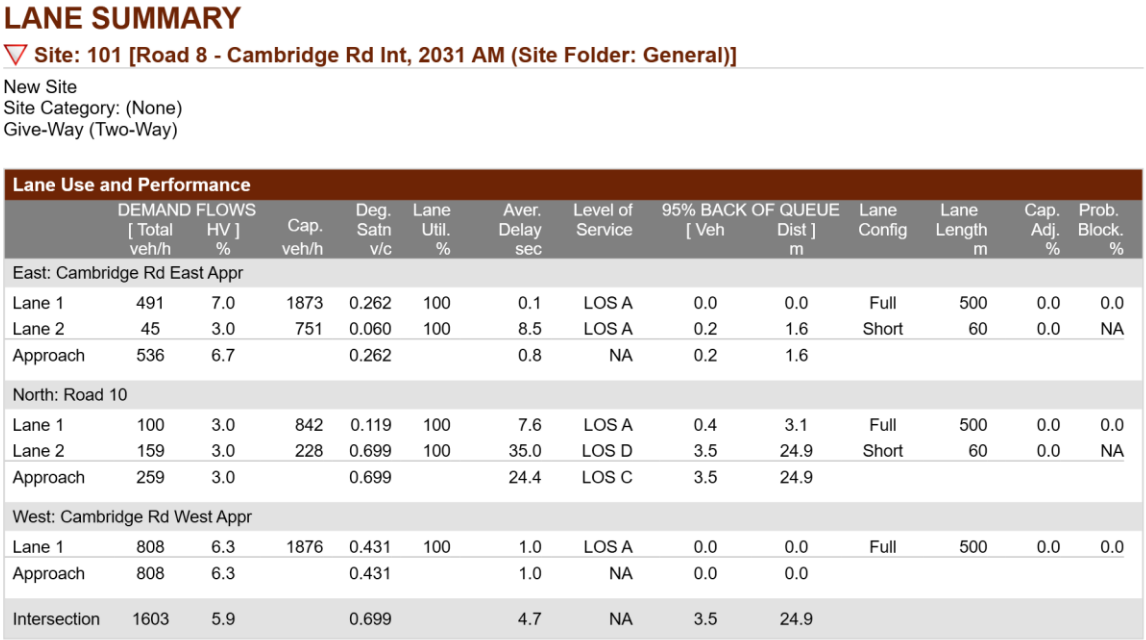
▽ Site: 101 [Road 10 - Cambridge Rd Int, 2021 PM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS		Cap.	Deg.	Lane	Aver.	Level of	95% BACK OF QUEUE		Lane	Lane	Cap.	Prob.
	[Total veh/h	HV] %	veh/h	Satn v/c	Util. %	Delay sec	Service	[Veh	Dist] m	Config	Length m	Adj. %	Block. %
East: Cambridge Rd East Appr													
Lane 1	354	7.0	1874	0.189	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	22	3.0	985	0.022	100	6.8	LOS A	0.1	0.6	Short	60	0.0	NA
Approach	376	6.8		0.189		0.5	NA	0.1	0.6				
North: Road 10													
Lane 1	19	3.0	945	0.020	100	6.7	LOS A	0.1	0.5	Full	500	0.0	0.0
Lane 2	26	3.0	329	0.080	100	14.4	LOS B	0.3	2.0	Short	60	0.0	NA
Approach	45	3.0		0.080		11.2	LOS B	0.3	2.0				
West: Cambridge Rd West Appr													
Lane 1	536	6.8	1882	0.285	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	536	6.8		0.285		0.3	NA	0.0	0.0				
Intersection	957	6.6		0.285		0.9	NA	0.3	2.0				

Scenario 3: Road 8 and 10, AM and PM, 2031 Results



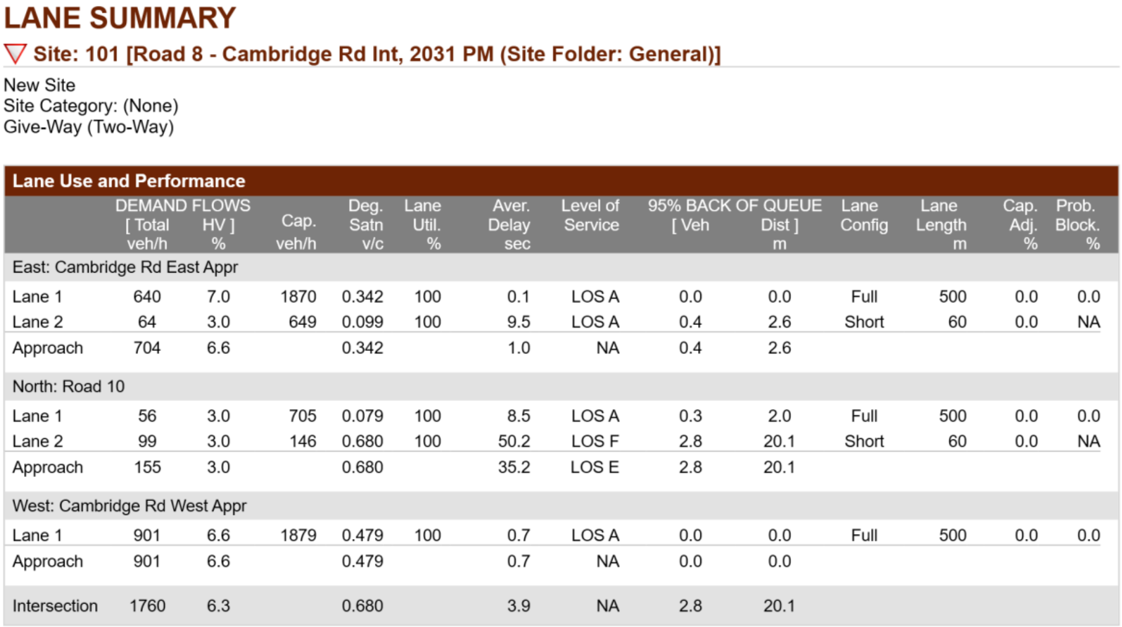
LANE SUMMARY

▽ Site: 101 [Road 8 - Cambridge Rd Int, 2031 AM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	DEMAND FLOWS HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	95% BACK OF QUEUE Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	491	7.0	1873	0.262	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	45	3.0	751	0.060	100	8.5	LOS A	0.2	1.6	Short	60	0.0	NA
Approach	536	6.7		0.262		0.8	NA	0.2	1.6				
North: Road 10													
Lane 1	100	3.0	842	0.119	100	7.6	LOS A	0.4	3.1	Full	500	0.0	0.0
Lane 2	159	3.0	228	0.699	100	35.0	LOS D	3.5	24.9	Short	60	0.0	NA
Approach	259	3.0		0.699		24.4	LOS C	3.5	24.9				
West: Cambridge Rd West Appr													
Lane 1	808	6.3	1876	0.431	100	1.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	808	6.3		0.431		1.0	NA	0.0	0.0				
Intersection	1603	5.9		0.699		4.7	NA	3.5	24.9				



LANE SUMMARY

▽ Site: 101 [Road 8 - Cambridge Rd Int, 2031 PM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	DEMAND FLOWS HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	95% BACK OF QUEUE Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Cambridge Rd East Appr													
Lane 1	640	7.0	1870	0.342	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	64	3.0	649	0.099	100	9.5	LOS A	0.4	2.6	Short	60	0.0	NA
Approach	704	6.6		0.342		1.0	NA	0.4	2.6				
North: Road 10													
Lane 1	56	3.0	705	0.079	100	8.5	LOS A	0.3	2.0	Full	500	0.0	0.0
Lane 2	99	3.0	146	0.680	100	50.2	LOS F	2.8	20.1	Short	60	0.0	NA
Approach	155	3.0		0.680		35.2	LOS E	2.8	20.1				
West: Cambridge Rd West Appr													
Lane 1	901	6.6	1879	0.479	100	0.7	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	901	6.6		0.479		0.7	NA	0.0	0.0				
Intersection	1760	6.3		0.680		3.9	NA	2.8	20.1				

LANE SUMMARY

▽ Site: 101 [Road 10 - Cambridge Rd Int, 2031 AM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS		Cap.	Deg. Satn	Lane Util.	Aver. Delay	Level of Service	95% BACK OF QUEUE		Lane Config	Lane Length	Cap. Adj.	Prob. Block.
	[Total	HV]						[Veh	Dist]				
	veh/h	%	veh/h	v/c	%	sec			m		m	%	%
East: Cambridge Rd East Appr													
Lane 1	536	7.0	1872	0.286	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	45	3.0	689	0.066	100	9.1	LOS A	0.2	1.7	Short	60	0.0	NA
Approach	581	6.7		0.286		0.8	NA	0.2	1.7				
North: Road 10													
Lane 1	100	3.0	776	0.129	100	8.1	LOS A	0.5	3.3	Full	500	0.0	0.0
Lane 2	159	3.0	207	0.766	100	42.9	LOS E	3.9	28.2	Short	60	0.0	NA
Approach	259	3.0		0.766		29.5	LOS D	3.9	28.2				
West: Cambridge Rd West Appr													
Lane 1	908	6.4	1877	0.484	100	0.9	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	908	6.4		0.484		0.9	NA	0.0	0.0				
Intersection	1748	6.0		0.766		5.1	NA	3.9	28.2				

LANE SUMMARY

▽ Site: 101 [Road 10 - Cambridge Rd Int, 2031 PM (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Lane Use and Performance

	DEMAND FLOWS		Cap.	Deg. Satn	Lane Util.	Aver. Delay	Level of Service	95% BACK OF QUEUE		Lane Config	Lane Length	Cap. Adj.	Prob. Block.
	[Total	HV]						[Veh	Dist]				
	veh/h	%	veh/h	v/c	%	sec			m		m	%	%
East: Cambridge Rd East Appr													
Lane 1	704	7.0	1870	0.376	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	64	3.0	638	0.101	100	9.6	LOS A	0.4	2.6	Short	60	0.0	NA
Approach	768	6.7		0.376		0.9	NA	0.4	2.6				
North: Road 10													
Lane 1	56	3.0	690	0.081	100	8.7	LOS A	0.3	2.0	Full	500	0.0	0.0
Lane 2	99	3.0	148	0.668	100	48.5	LOS E	2.7	19.1	Short	60	0.0	NA
Approach	155	3.0		0.668		34.1	LOS D	2.7	19.1				
West: Cambridge Rd West Appr													
Lane 1	957	6.6	1879	0.509	100	0.7	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	957	6.6		0.509		0.7	NA	0.0	0.0				
Intersection	1880	6.3		0.668		3.6	NA	2.7	19.1				