REPORT

Geotechnical and Three Waters Engineering Report for HLG Site

for Dean Hawthorne

Rev Final Draft - 13/03/2023

BTW COMPANY

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Geotechnical and Three Waters Engineering Report for HLG Site

for Dean Hawthorne

Reviewed

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Rev Final Draft - 13/03/2023

CONTENTS

TABLES

FIGURES

1 INTRODUCTION

BTW Company Limited (BTW) have been engaged by Dean Hawthorne to carry out a preliminary geotechnical and three-waters servicing assessment of the land area identified in Figure 1.1, for the purposes of rezoning the site to Deferred Industrial Zoning.

This assessment will review the proposed Plan Change 17 (PC17) re-zoning submission for the property to the south of the site (Kama Trust) and provide comment on how the project site would be serviced and could integrate with and enhance the currently proposed PC17 submission.

The site is located to the northwest of the town of Cambridge. The property assessed comprises multiple lots and part lots with street address 376, 372, 364, 358, 412, 346 Peake Rd and 24 Hautapu Rd, Cambridge (Lot 1 DPS 57935, Lot 1 DP 553825, Lot 2 DP 553825, Lot 2 DP 361070, Lot 1 DP 512688, PART Lot 1 DP 556525, PART Lot 1 DP 532855). The proposed site runs adjacent to the Mangaone Stream.

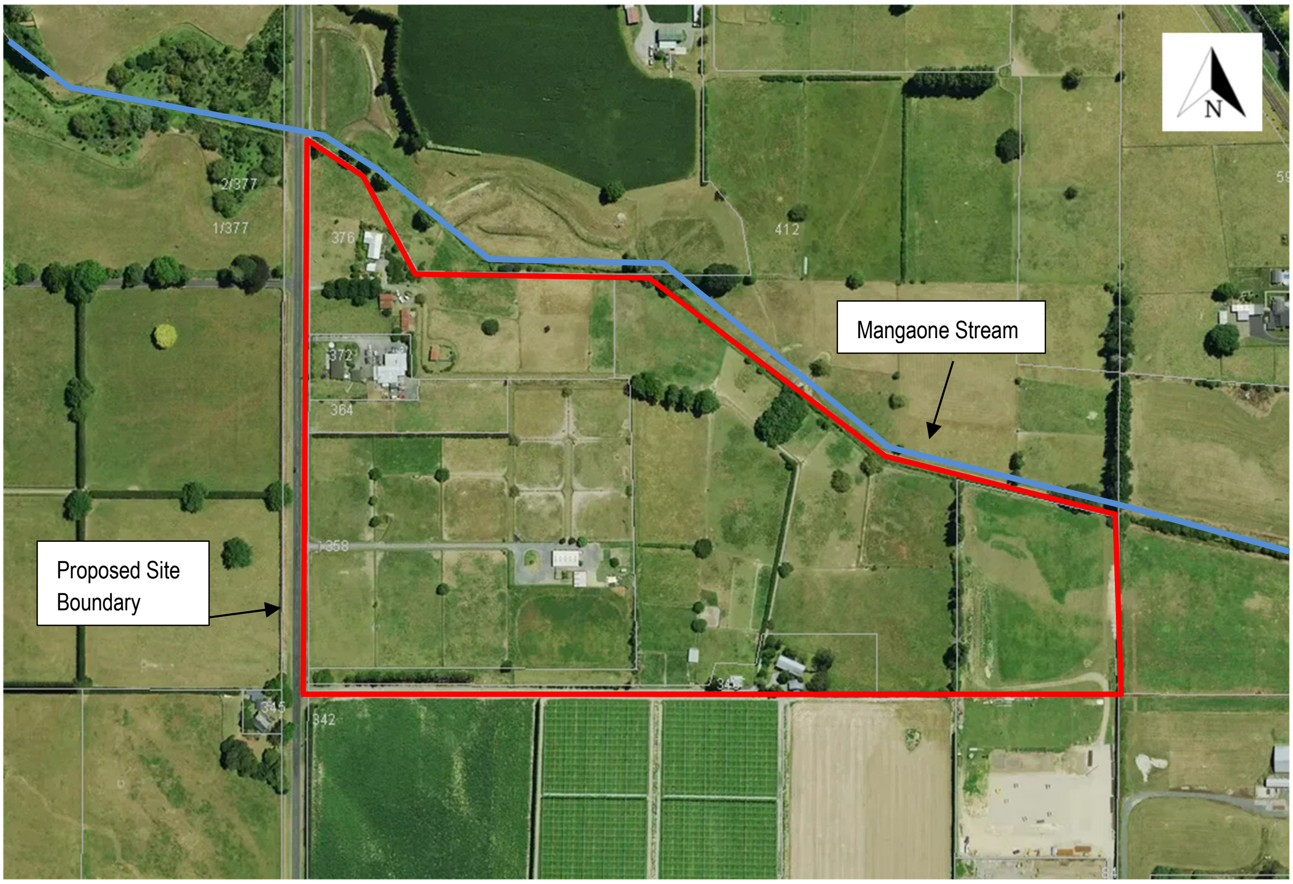

Figure 1.1: Site Plan - Area of assessment bounded by the red line

The proposed site is approximately 17 hectares in size and is located north of the C9 growth cell area and the Karma Trust PC17 site. The proximity of the site to the C9/C8 cells and Kama Trust area is shown in Figure 1.2.

Figure 1.2: Site Location in Relation to Growth Cells C8/C9 and the Kama Trust area

This Engineering and Geotechnical assessment includes the following:

- Desktop review of the site to determine the anticipated soil types and provide general commentary on the suitability of the soils to support typical residential dwellings.
- Assesses the initial feasibility of servicing the site with water and wastewater from the Waipa District Council owned water and wastewater reticulations, based on the three-waters assessment and modelling previously completed for the PC17 submission and provides preliminary estimates on the proposed development water demand and wastewater outflows.
- Provides a preliminary review of stormwater management requirements and required infrastructure, identifying anticipated stormwater reserve areas, suitable stormwater management techniques and potential integration with the PC17 submission to present opportunities for integrated infrastructure with improved holistic stormwater solutions.

1.1 Existing Services

Existing services nearby the site are shown in Figure 1.3. Connections to council owned services are reliant on the future development of services in future growth areas, growth cells C8 and C9 (location of growth cells identified in Figure 1.2).

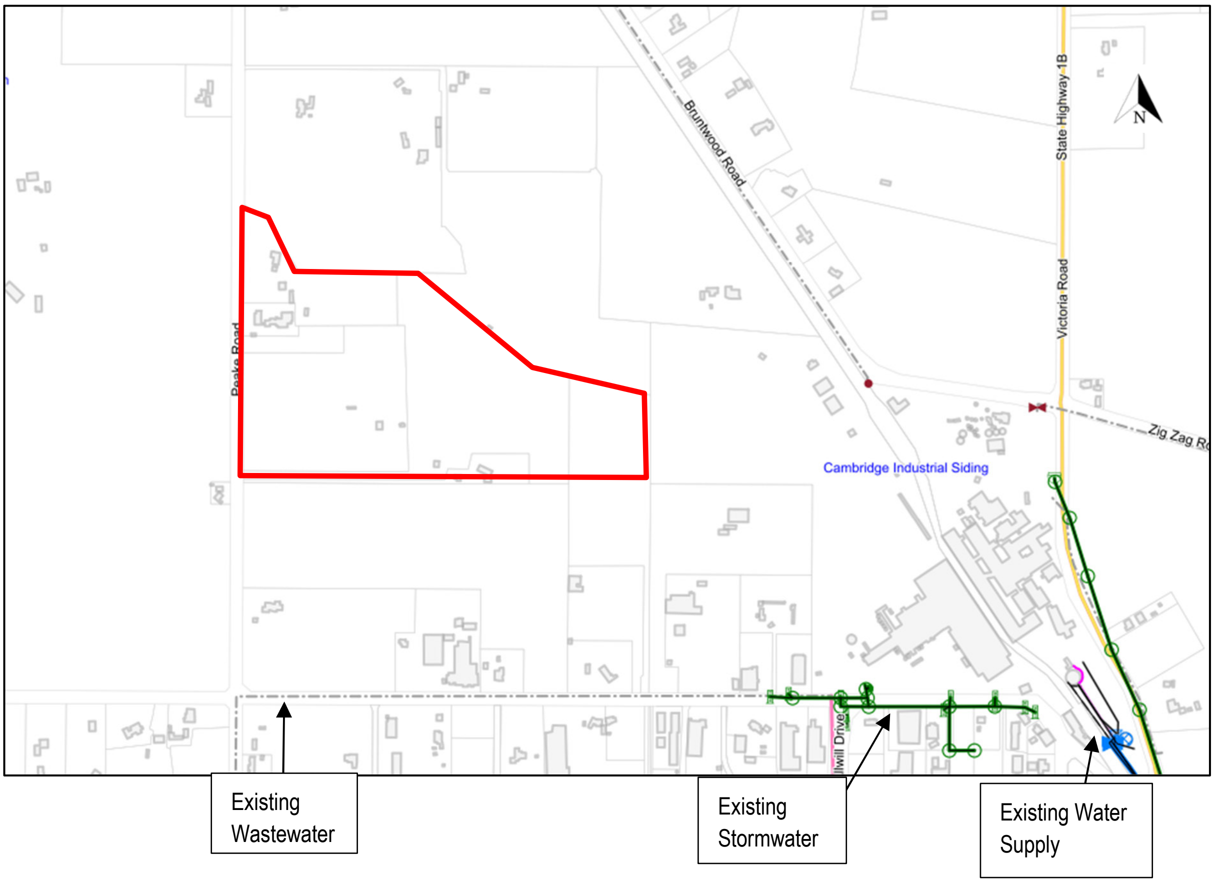

Figure 1.3: Existing Services in relation to site – Waipa District Council GIS

1.2 Report Disclaimer and Limitations

This report has been prepared based on a preliminary desktop assessment of the site only, referring to existing information and literature. The outcomes of this assessment and report are subject to on-site investigations, site specific geotechnical investigations, water and wastewater network modelling, site-specific stormwater management assessment, and catchment wide flood modelling.

2 PRELIMINARY GEOTECHNICAL DESKTOP ASSESSMENT

A preliminary desktop geotechnical assessment of the site has been completed to identify landforms and anticipated soil types for the site. Review of existing literature and previous geotechnical experience with similar local soils has enabled this preliminary assessment of the suitability of on-site soils to support industrial development to be completed. For future subdivision design and consenting stages, additional site-specific geotechnical testing and analysis will be required.

2.1 Previous Reporting

2.1.1 Karma Trust – 342 Peake Road Cambridge

A preliminary geotechnical report was undertaken for the Karma Trust[1] at 342 Peake Road, Cambridge. The key findings from the report were:

- Ground conditions are likely to be Hinuera Formation silt and sand.
- Groundwater in the area is likely to be greater than 4 m below existing ground level.
- The potential for settlement of low strength soils and liquefaction are likely to be the primary hazards for the site.
- The site is likely to be within liquefaction performance level L0 to L1 (insignificant to mild hazard).
- The potential for settlement would need to be mitigated for development.
- Additional geotechnical assessment is needed to confirm ground conditions and recommendations.
- Additional input will be required for elements of future development including:
  - Settlement screening.
  - Detailed liquefaction assessment.
  - Earthworks specifications, if proposed.
  - Site specific pavement investigation, testing and design.
  - Engineered foundation design for structures.

2.1.2 Stormwater Disposal – Allwill Drive

Several investigations into the soakage potential of the soils within Allwill Drive, Hautapu have been undertaken by BTW Company[2,3] using both falling head and constant head infiltration test methods.

These tests were in soils of similar composition to those expected to occur within the development site and yielded the following soakage rates as shown in Table 2.1.

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[1] HD Geo: 2021: Preliminary Geotechnical Report, 342 Peake Road, Cambridge. Prepared for: Karma Trust. Project No.: HD1857, Reference: PGR-1; Dated: 12 February 2021. 39p.

[2] BTW Company; 2019: Geotechnical Report, Stormwater Soakage Rates and Groundwater Levels of Allwill Drive, Cambridge. Report prepared for: GHD. Job No.: 190767; Revision: B; Date: 29/07/2019. 24p.

[3] BTW Company; 2020: Geotechnical Report, Stormwater soakage rates at the southern end (turning head) of Allwill Drive, Hautapu. Report prepared for: Waipa District Council. Job No.: 191347.05; Revision: A; Date: 18/06/2020.23p.

Table 2.1: Soakage Rates measured in Allwill Drive

<table>
<tr><th>Soakage Test</th><th>Test</th><th>Soakage Rate (mm/hr)</th><th>Receiving Soil</th><th>Depth of Test (m BEGL)</th><th>Ground Water Test (m BEGL)</th><th>Notes</th></tr>
<tr><td rowspan="4">Falling Head</td><td>A (@ intersection)</td><td>3,000</td><td rowspan="2">pumiceous, silty SAND</td><td>0.6</td><td>1.7</td><td>-</td></tr>
<tr><td>B</td><td>2,200</td><td>0.8</td><td>1.5</td><td>-</td></tr>
<tr><td>C</td><td>82</td><td>medium grained SAND with some silt</td><td>2.0</td><td rowspan="4">NE</td><td>-</td></tr>
<tr><td>D (@ turning head)</td><td>4,200</td><td>medium grained SAND with minor silt</td><td>2.1</td><td>-</td></tr>
<tr><td rowspan="2">Constant Head</td><td>A (@ turning head)</td><td>13,494</td><td rowspan="2">medium to coarse SAND with variable ratios of gravel and silt</td><td rowspan="2">3.6</td><td rowspan="2">Rate held constant for 30 minutes</td></tr>
<tr><td>B (@ turning head)</td><td>9,740</td></tr>
</table>

NE = Not encountered

The soakage rates for the Allwill Drive stormwater design project allowed for a 50% reduction factor; however, these rates shown in Table 2.1 are significantly higher than the currently proposed 30 mm/hour for the PC17 stormwater soakage basin.

2.2 Geology/Geomorphology

Published geological information shown in the Kear and Schofield[4] map shows that the property sits on the surface of the Hinuera Formation.

The Hinuera Formation is comprised of alluvial sediments consisting of pumiceous and rhyolitic gravel, sand and silt and minor peat, deposited in two phases and then subsequently capped by a 0.4 to 0.6 m thick, late quaternary (< 60,000 year old) tephra mantel[5] which occurs in the field as a yellowish brown clayey Silt.

The first phase of Hinuera Formation deposition occurred between 65,000 to 24,000[6] years ago when a high energy river system deposited coarse, sand gravel and pumice, with occasional silt lenses across the Waikato Basin as a braided river sequence. The sediments deposited during this period tend to be well sorted, dense soils, with high strengths.

During the second phase (22,000 to 17,000 years ago)[7] due to changes in the climate and sediment supply, this sediment sequence comprises of course sands at the base and finer sediments at the top. The sediment composition at the top of the sequence comprises of fine sand and silt which were deposited in a low energy river basin. The finer soil tends to be less well sorted, have lower densities and strength than those deposited during phase one.

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[4] Kear D, Schofield JC: Sheet N65 Hamilton (First Edition – 1976 Reprint) 'Geological Map of New Zealand' 1:63,360. Department of Scientific and Industrial Research, Wellington. 1 sheet.

[5] Selby MJ and Lowe DJ; 1992: The Middle Waikato Basin and Hills. In Soons JM and Selby MJ (editors); Landforms of New Zealand. Second Edition Longman Paul Limited. 531p.

[6] Selby and Lowe; 1992.

[7] Lowe DJ; 2010: Introduction to the landscapes and soils of the Hamilton Basin. In: Lowe DJ; Neall VE, Hedley M; Clothier B; Mackay A; 2010: Guidebook for Pre-conference North Island, New Zealand 'Volcanoes to Oceans' field tour (27-30 July). 19th World Soils Congress, International Union of Soil Sciences, Brisbane. Soil and Earth Sciences Occasional Publication No. 3, Massey University, Palmerston North, pp. 1.24-1.61.

After 17,000 years, the Waikato River became intrenched into its current location[8] and proceeded to incise into the underlying sediment sequence. Abandoned river channels can be observed throughout the Waikato Basin from failed down cutting attempts[9].

The Hamilton Basin was infilled by multiple braided river channels that migrated laterally across the fan building up at a uniform height[10], giving rise to typical Hinuera Surface ridge and swale effect the amplitude of which varies across the basin[11]. which can have a physical expression in the landform ranging from very low relief (less than 100 mm to 1.5 m[12]). The ridge and swales were deposited parrel to the flow and delineate the flow direction and velocity of the paelo-channel.

The ridges comprise of courser sediments and the hollows of finer sediments[13]. Abandoned braided river channels were infilled white to pale grey impervious silts. In places peat formed in these channels resulting in interbedded peat and silt deposits[14]. The thickness of the Hinuera Formation varies across the fan[15] but is thickest at the fan head (80 m) at the mouth of the Maungatautari Gorge, to the southeast of Cambridge, 40 m thick just north of Hamilton and 20 m thick just south of Ngaruawahia.

Some interesting points regarding the Hinuera Formation are provided by Hume et al.[16]:

> *'Two apparently paradoxical features are noted. The first is the variability of the Hinuera sediments in detail. No stratigraphic columns are alike, and the sequence of lithologies, textures, and sedimentary structures is as different between the sections erected at the same locality as between other sections. The second feature is the uniform appearance of the Hinuera Formation on a regional scale.'*
>
> *The different lithologies typically occur in erosional contact with each other, both vertically and latterly, and <u>do not occur in in regular or predictable stratigraphic patterns</u>'*

2.2.1 Tauwhare braded river channel

Located to the north of the development area is the 'Tauwhare course' and its outflow channels[17]. The 'Tauwhare course' is inferred to be one of the last upbuilding fans of the Waikato Basin and this directly formed the Hinuera Surface in the area as shown in Figure 2.1. As such there is no channel (such as seen in current Waikato River) as the 'channel' has built up the basin floor relative to is surrounds. Given the manner in which sediment moves through a braided river channel it is likely that sand will be the dominant grain size within the channel with much lesser amounts of silt occurring. It is expected that the soils within the Tauwhare course' occur und the site in question as well.

8 Lowe DJ; 2010.

9 McCraw J; 2011: The Wandering River, Landforms and geological history of the Hamilton Basin. Geosciences Society of New Zealand, Guidebook. 16p. Refer to Chapter 5.

10 Selby and Lowe; 1992.

11 McCraw J; 2011. Refer pictures on pages 20 and 21.

12 Kear D, Schofield JC; 1978. Refer to page 109.

13 Kear D, Schofield JC; 1978: Geology of the Ngaruawahia Subdivision. Department of Scientific and Industrial Research, Wellington. New Zealand Geological Bulletin 88. 168p. Refer to Figure 41.

14 Selby and Lowe; 1992.

15 Selby and Lowe; 1992. Refer to Figure 10.6.

16 Hume TM, Sherwood AM, Nelson CS; 1975: Alluvial sedimentology of the Upper Pleistocene Hinuera Formation, Hamilton Basin, New Zealand. Journal of the Royal Society of New Zealand, 5:4, 421-462.

17 McCraw J; 2011. Refer page 30.

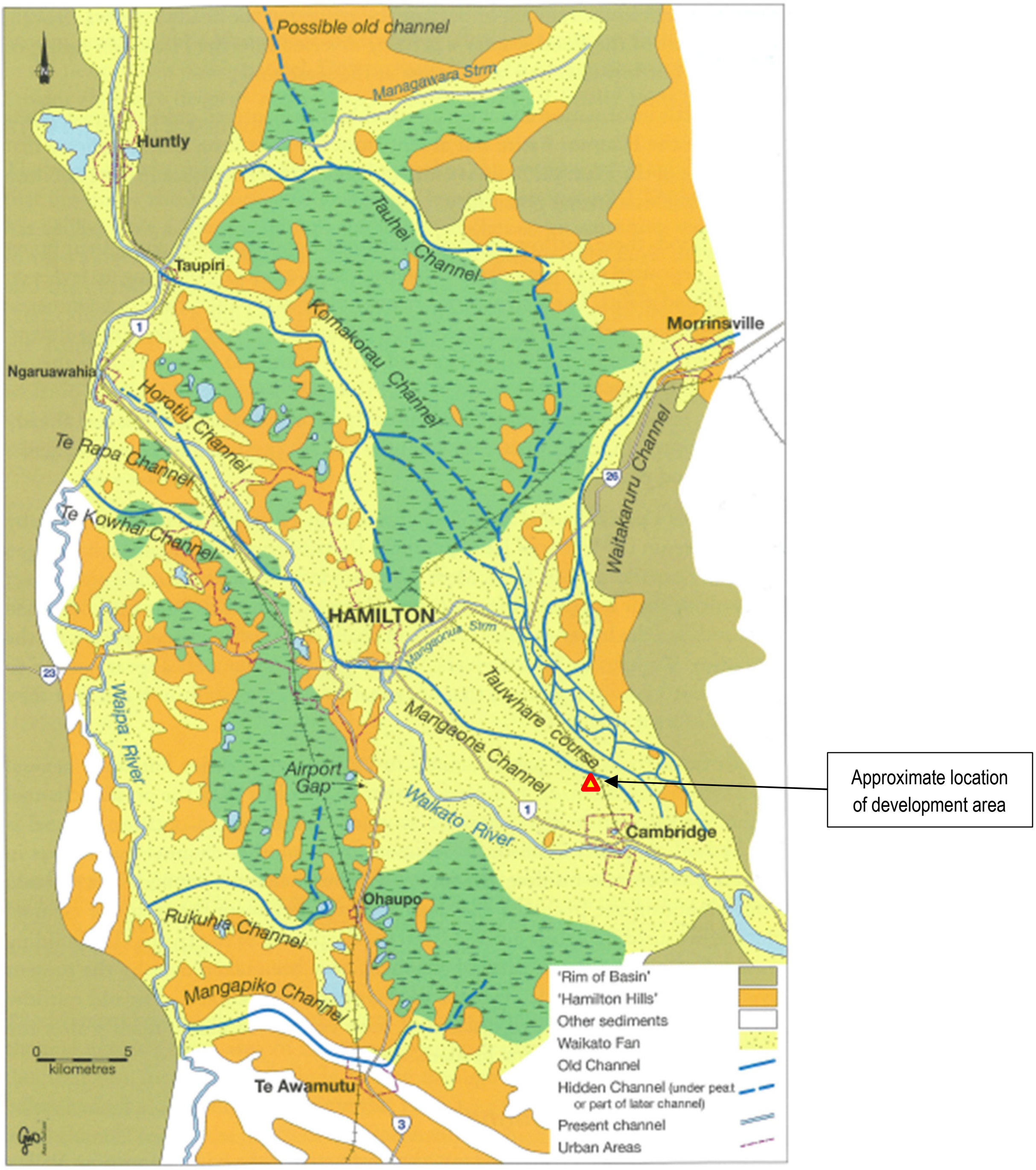

Figure 2.1: Preserved outflow channels of the Waikato River[18]

2.3 Geomorphology

Ground models (temporal and spatial) for the development are presented below.

[18] McCraw J; 2011. Refer page 27.

2.3.1 Ground Model

A generic ground model[19] is presented in Figure 2.2 and is thought to be representative of the expected soil / landform in the Waikato. The soils are generally expected to be as described in the preceding sections.

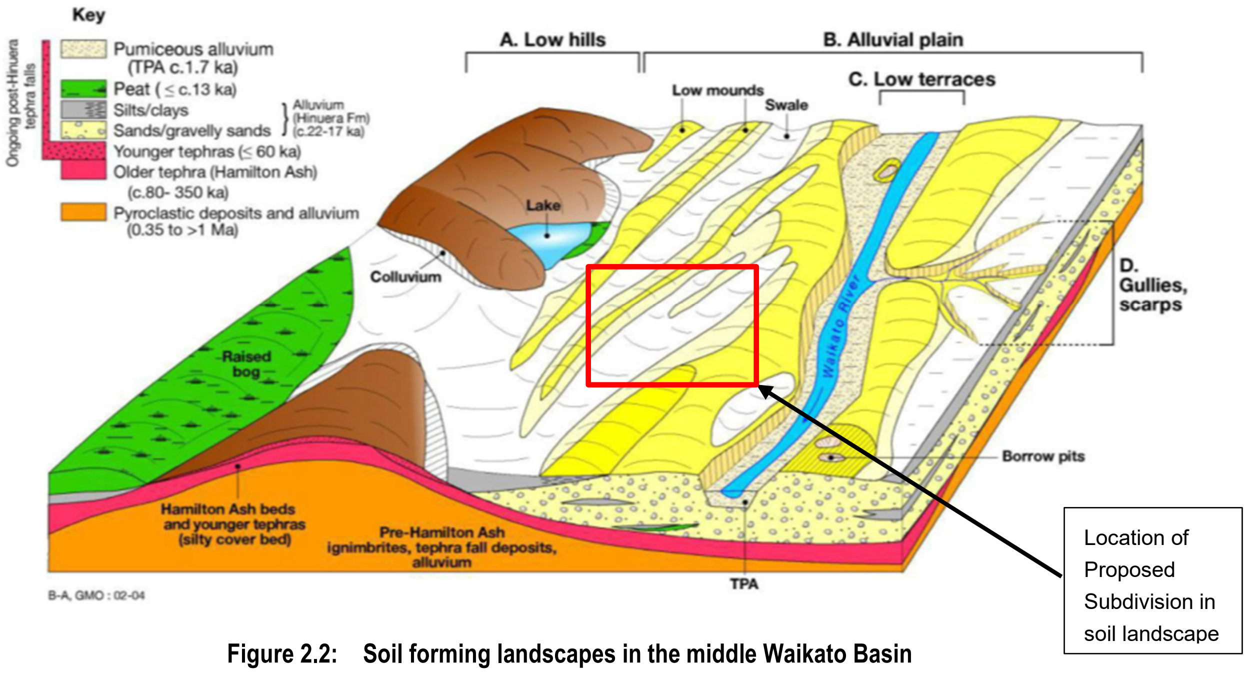

Figure 2.2: Soil forming landscapes in the middle Waikato Basin

2.3.2 Sediment Type Lithofacies Model

Within the Hinuera Formation, a predictable relationship between sediment types and depositional environment exists[20]. The relationship between the main lithofacies and geological materials (engineering soils) in the Hamilton Basin is shown in Figure 2.3 and Table 2.2.

The site testing locations soil logs were evaluated according to the lithofacies model below (refer to Table 2.3).

Table 2.2: Sediment and environmental interpretations of the lithofacies of the Hinuera Formation[21]

Lithofacies	Occurrences	Dominant Texture	Depositional Environment	Stratigraphic Position
A1	Extremely common	Gravelly sand	Active braided channel	Channel
A2	Rare			
B	Fairly common	Sand		
C	Uncommon	Sandy gravel		
C1	Rare	Gravelly sand		
D	Moderately common	Silt	Abandoned braided channel	Overbank
E	Uncommon		Flood basin	

[19] Lowe, D. J.; 2010: Introduction to the landscapes and soils of the Hamilton Basin. In: Lowe, D.J.; Neall, V.E., Hedley, M; Clothier, B.; Mackay, A. 2010. Guidebook for Pre-conference North Island, New Zealand „Volcanoes to Oceans" field tour (27-30 July). Palmerston North: 19th World Soils Congress, International Union of Soil Sciences, Brisbane. Soil and Earth Sciences Occasional Publication No. 3, Massey University.

[20] Hume et al.: 1975.

[21] Adapted from Hume et al.

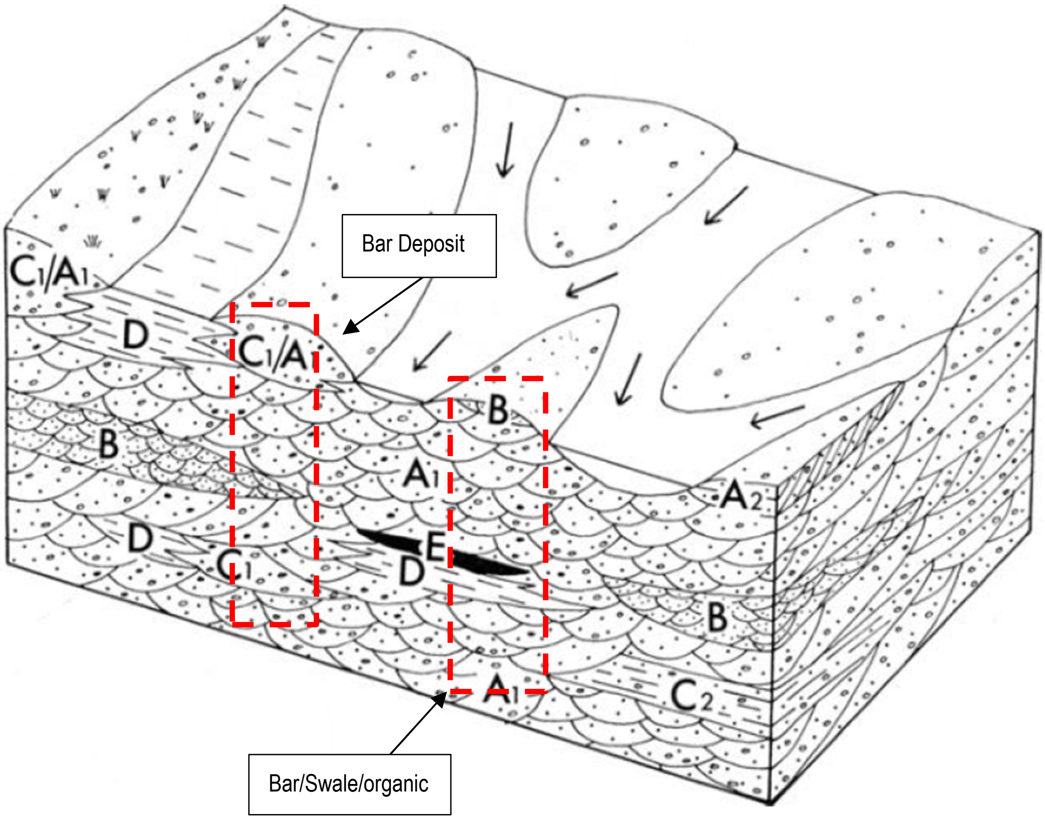

Figure 2.3: Summary diagram showing temporal and depositional relationships of lithofacies and geological materials (engineering soils) in the Hinuera Formation.[22]

This schematic section shows a detailed model of the build-up of multiple braided channels in the Hamilton Basin. Abandoned braided channels would become depressions or swales where overbank and occasional flood basin deposits of silt and organic peaty soils respectively could build up. These poorly drained silt swales are separated by well-drained gravelly ridges or river bar deposits.

In Figure 2.3 the soils expected to occur are shown in the dashed box as a bar deposit comprising of sand of various ground sizes and densities. The expected alternative is a sequence dominated mainly sand grains with the occasional silt and or organic layer (expected to be very rare in occurrence) forming a bar/sale and or organic sequence.

2.3.3 Gully System

Located approximately 150 m to the north of the development area is the incised Mangaone Stream (approximately 5 m[23] below the level of the development site. The incised stream forms part of the 'Hintion Gully' system[24]. These gullies form a dendritic system and are explained in detail in McCraw[25]. This arm of the 'Hintion Gully' system is likely to have incised into the

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[22] Hume et al.: 1975.

[23] Waikato Regional Council - Contours: n.d.: https://waikatomaps.waikatoregion.govt.nz/Viewer/?map=8d6d6fda779b4e59951953ae97d0ec4a. Accessed 11/01/2023.

[24] McLeary WH: 1972: A study of the gully systems of the Waikato Basin with particular reference to those in and surrounding the city of Hamilton. Research project submitted as part of a Diploma of Landscape Architecture, University of Canterbury. 182p,

[25] McCraw J; 2011. Refer page 49.

underlying Hinuera formation in response to the Waikato River downcutting about 17,000 years ago[26].

2.3.4 Groundwater

A review of groundwater for the C2 growth cell[27] in Cambridge was undertaken which provided commentary on the Cambridge area as a whole. Beca's assessment of the regional groundwater depth and perched water tables is provided below:

> 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. 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.[28]

Approximate groundwater depth as modelled by Beca is presented in Figure 2.4. Applying this model to the area would tend to suggest that ground water occurs at a depth of 5 m ± 1 m BEGL[29] at the proposed development area.

This depth assumption would be correct when considering that Mangaone Stream is located approximately 200 m to the north with the water body occurring 5 m below the ground level of the proposed development area. Given the proximity of the development to the Mangaone Stream and the streams depth, the stream is likely acting as the local draw down point in the landscape suggesting the ground water depth assumption would be correct.

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[26] McCraw J; 2011. Refer page 37.

[27] Beca; 2020: 3Ms Cambridge Subdivision, Technical Assessment of Groundwater Effects Stage One. Prepared for: 3MS of Cambridge GP Ltd. Dated: 1 December 2020; Reference: 3201678-73331603-13. 64p.

[28] Beca; 2020. Refer to page 5.

[29] Beca; 2020. Refer to Appendix B, page 9 – Figure 19.

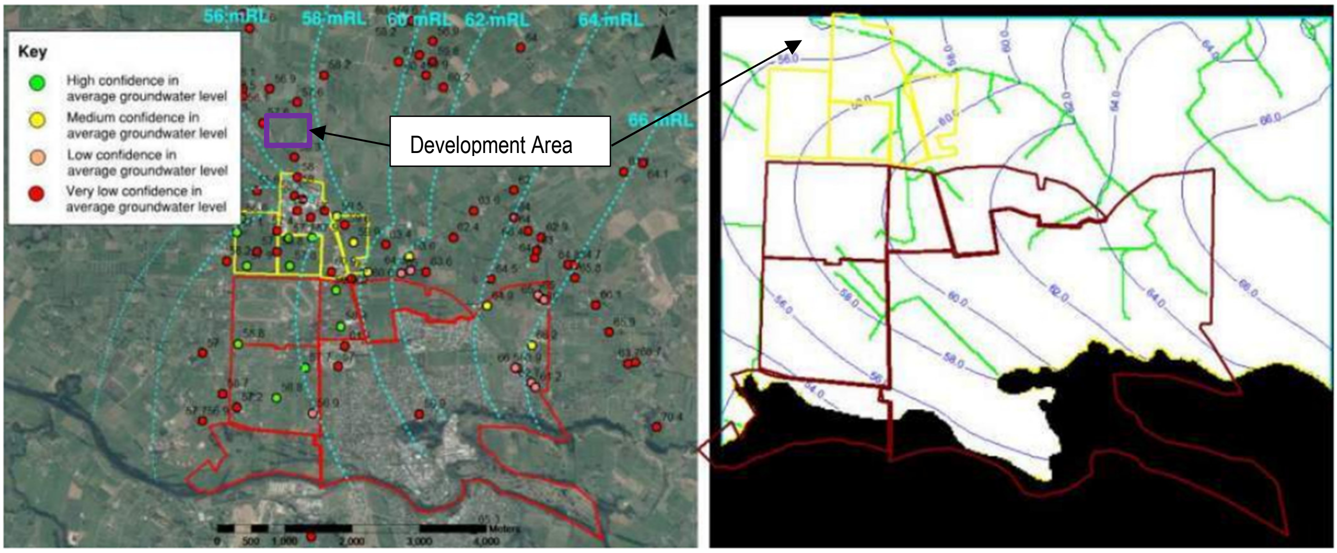

Figure 2.4: Approximate groundwater depth in the Cambridge Area[30]

2.4 NZGD Assessment

A review of the New Zealand Geotechnical Database (NZGD)[31] was undertaken to assess previous testing near the proposed building site. The generalised soil sequence is tabulated in Table 2.3.

Table 2.3: Generalised soil sequence of nearby NZGD boreholes.

Landform	Location	Test	No. of tests at location reviewed	Position from area of investigation (Approximate)	Depth to end of log (m BEGL)	Groundwater depth (m BEGL)	Sediment Lithofacies (Soil sequence)
Hamilton Gully	Whey Site	BH & CPT	6	970 m SE	2.0;14.0; 24	0.5 to 1.2 or 23	Post HF peat over HF Bar/Swale/organic
	Fonterra SW Pond	BH	2	750 m SE	12.4	0.9 to 3.3	Bar Bar/Swale/organic
Hinuera Surface	372 Victoria Road	HA	1	1.3 km NE	2.0	NE	Swale
	295 Peake Road	HA	5	780 m SW	2.0	NE	Bar
	273 Peake Road	HA	4	1.0 km SW	2.0	NE	Bar
	280 Peake Road	HA	13	770 m S	2.0 or 3.0	NE	Bar (major) Bar/Swale (minor)
	90 Hautapu Road	HA	6	600 m SE	2.0 or 4.0	NE	Bar (major) Bar/Swale (minor) Bar/Swale/organic (v minor)

BH = machine Bore Hole; HA = Hand Auger; CPT = Cone Penetrometer Test; NE = Not Encountered; HF = Hinuera Formation

30 Beca; 2020. Refer to Appendix B, page 10.

31 New Zealand Geotechnical Database; n.d.: https://www.nzgd.org.nz/ARCGISMapViewer/mapviewer.aspx. Accessed 12/01/2023.

2.5 Liquefaction Risk - Screening

2.5.1 Liquefaction Risk Assessment – Simple Screening Method

BTW has undertaken a liquefaction risk assessment in accordance with the method and guidelines presented in MBIE[32].

The seismic parameters for this assessment are presented in Table 2.4.

Table 2.4: Seismic parameters for liquefaction assessment

Return Period (years)	500
Location:	Cambridge [33]
α_{max} (g):	0.28
Earthquake Magnitude	5.9

The geology and groundwater levels estimated under the site are presented in Section 2.2 and Section 2.3.4, summarised in Table 2.5.

Table 2.5: Summary of geological age and Groundwater levels[34]

Geology	Groundwater Level (metres)
Greater than Pleistocene in age	Assumed at 5.0 m BEGL

The inputs for magnitude-corrected peak ground acceleration as outlined in Idriss and Boulanger[35] are presented in Table 2.6.

Table 2.6: Inputs for magnitude-corrected peak ground acceleration

Magnitude Scaling Factor (MSF)	**Peak Ground Acceleration** ($PGA_{7.5}$)
1.521	0.18

With reference to Table 2.5 and Table 2.6 it can be seen that:

- The age of the soil is greater than latest Pleistocene.
- The shaking intensity of a 500-year event is 0.18g which is less than the triggering point of 0.3g.
- Groundwater is predicted to be at a depth of 5.0 m BEGL.

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32 Ministry of Business, Innovation and Employment; 2017: Planning and engineering guidance for potentially liquefaction-prone land. Resource Management Act and Building Act aspects. Ministry of Business, Innovation and Employment, Building System Performance Branch, Wellington. 134p. Refer to Section 4.4.4.

33 Ministry of Business, Innovation and Employment; 2021: Earthquake geotechnical engineering practice, Module 1. overview of the guidelines. 47p. Appendix A. Table A1: Peak Ground Acceleration and Earthquake magnitude values recommended for Geotechnical Assessment for site classes A., B, C, D and E for level ground conditions, pages 36 – 34.

34 MBIE: 2017. Refer to page 41, recommends that 'For screening purposes using this table, a high groundwater scenario should be assumed (e.g. a typical seasonal high groundwater level)'.

35 Idriss IM and Boulanger RW; 2008: Soil liquefaction during earthquakes. Oakland, California: Earthquake Engineering Research Institute. 237p.

Therefore, using the MBIE (2017) semi-quantitative screening method it can be concluded that liquefaction damage is unlikely at this location during a 1 in 500-year seismic event.

Lateral Spread

Given the cohesive nature of the soils encountered, and the low groundwater encountered, it is assumed that any lateral spreading will occur as a block movement and will be limited in its extent.

2.6 Review of Expected Soil Properties

Based on the literature above and previous geotechnical experiences with the soils in that portion of the upper Waikato Basin (on and adjacent to the 'Tauwhare course') and with Hinuera Formation soils the following general comments are provided for the expected soils within the development area.

Tephra Sequence

The soil sequence encountered in the area as revealed by the NZGD soil log review (refer to Table 2.3) shows that the soils comprise of a near surface sequence of clayey Silt (post 60,000 year old tephra) typically have an ultimate bearing capacity of less than 300 kPa, typically 200 kPa. Generally, in these areas with higher silt content, a foundation improvement layer of excavated soil replaced with imported hard fill (pit sand or similar) for light weight buildings on concrete foundations is suitable. For pavement designs, this soil type is also cut to waste.

Bar Sequence (Sand soils)

The underlying Hinuera Formation sand typically form a coarsening downwards sequence where the soil strength increases with depth, or the soil strength alters from medium dense to dense but maintains 300 kPa (ultimate) once achieved in a sand bar depositional environment. Soakage in this setting is excellent, refer to the constant head soakage test results in Table 2.1.

Road pavements may be formed in these sand soils with relative ease and may require only a proof roll proper to the subgrade being placed. An assumed California Bearing Ratio (CBR) of 5% can be assumed for these soils.

Bar/Swale Sequence (Sand/Silt soils)

For soils in a bar/swale setting the statement for the sand bar portion (above) holds true. In the swale lenses portion of the sequence[36] of fine-grained sand mixed with silt or clayey Silt layers can occur. These soils depending on their thickness can have an ultimate bearing capacity in the range of 150 to 200 kPa. These silt lenses have an irregular temporal and spatial distribution and will be identified through site specific testing. Very sparsely distributed and typically thin layers of organic Silt may occur within the development area. Where encountered these soils will need to be removed.

For a portal framed building the foundation pad can be increased to lower the foundation demand on the soil sequence if the low strength zone occurs within a critical distance of the foundation or excavated and replaced with hard fill.

Stormwater soakage into these soils can be variable with the clayey Silt soils acting as a confining bed within the soil sequence. The silt layer will slow the migration of the water down through the profile. In certain circumstances water may perch on this layer. Provided that the receiving sand soil of a soakage system is at least 1 m deep, water may be disposed of into a soakage system that overlies a silty soil layer. For soakage systems with large contributing areas a secondary flow

[36] Silt lenses are typically < 0.5 m thick.

will need to be considered if a low permeable silt occurs with c. 1.5 m of the base of a soakage system.

In general the pavement comments within the bar sequence section above hold true depending on the depth to the silt layer. If the silt layer occurs within a depth of 1.0 m to the base of the pavement[37] then it is likely that it will need to be removed and replaced with hardfill. The sand underlying this silt (assuming a low silt content) will likely have a CBR in the order of 5%.

Groundwater

Groundwater is expected to be at depth of approximately 5 m BEGL based on groundwater models and sites geomorphology. Local testing has demonstrated (refer Table 2.3) that groundwater is not likely in the first 4 m and not in the first 2 m of the soil profile. It should be noted that after prolonged wet periods water may be locally perched especially in bar/swale soil sequence within the top 4 m of the soil sequence.

2.7 Geotechnical Summary

In summary, based on a preliminary desktop review of the site; the site soils are deemed to be generally suitable to support the proposed industrial type development.

The foundation soils are sufficient to support light weight portal framed industrial type buildings with low to moderate foundation bearing demands with only minor shallow (< 1 m) subgrade improvement layers. Deep piles or excavation and replacement with imported hardfill is not envisaged.

Onsite soakage will be an option for the development, provided that a secondary overland flow paths are provided for (refer to Section 5.6).

Liquefaction is not expected to be a site hazard nor is lateral spread. A consideration for future building foundation design will be the consolation of soft soils at depth, especially if the soil sequence consists of bar-swale/organic soils in a structure’s footprint.

[37] Assuming local road with light traffic in a light industrial setting.

3 WATER SERVICING

3.1 Design Objectives

The following design objectives have been identified as outcomes for the design of the water supply system:

- Propose a workable conceptual water supply servicing option(s) for industrial development of the site.
- Consider sustainable and environment outcomes through incorporation of water sensitive design elements into the built environment.
- Calculate water supply demands and requirements in accordance with Waipa District Council (WDC) level of protection and level of service requirements (as defined within RITS).

3.2 Existing Water Supply Infrastructure

WDC GIS information indicates that the site is not currently serviced by any water supply infrastructure.

3.3 Water Supply Concept and Previous Modelling Scenario Outcomes

Site water supply servicing is proposed to be supplied via extension of the future Waipa District Council water supply network, which will be constructed for the development and servicing of the C8 and C9 growth cells[38] .

The proposed site sits directly adjacent to the Kama Trust site, that is currently seeking industrial development rezoning. The proposed Kama Trust site industrial development water demands have been previously modelled to assess the impact on the WDC owned Cambridge water reticulation, considering the C8 and C9 growth cell development and water infrastructure installation. Refer to Appendix A for a copy of this modelling report, and Figure 3.1 for a plan showing how the proposed C8 and C9 growth cells water infrastructure will conceptually be located. This report concluded the following:

> *'**Industrial Supply**: 7.5m³/ha/day, or 0.087L.s.ha based on 30 people/ha, with the peak hour on the peak day of 0.435 L/s/ha (factor of 5.0) as per Harrison Grierson's Cambridge C8/C9 Master Plan – Water Supply Report.*
> ***Required Minimum Working Pressure**: LoS of 300kPa (30 m) pressure at every connection point.*
> ***Fire Flow**: New Zealand Fire Service Code of Practice; SNZ PAS 4509:2008 and subsequent amendments, to the satisfaction of the New Zealand Fire Service. Commercial requirement FW3 – 50 L/s. The minimum residual pressure at the required fire flow is 10 m.'*

As the Kama Trust site can be sufficiently supplied with water from the council reticulation, and our project site has a similar general elevation to the Kama Trust site, conceptually it appears feasible

[38] WSP 12/7/22; Water Supply Hydraulic Assessment for Hautapu Industrial Kama Trust Plan Change Report referencing Harrison Grierson; Cambridge C8/C9 Mater Plan – Water Supply Report. . As referenced within Appendix A

to service the site through extension of the future Waipa District Council water supply reticulation network.

Further detailed modelling assessment of the calibrated Waipa District Council Cambridge water model will be required to confirm this approach, essentially an extension to the modelling report completed for the Kama Trust site. The anticipated water demand for the project site and recommended water sensitive techniques are outlined in the following sections of this report.

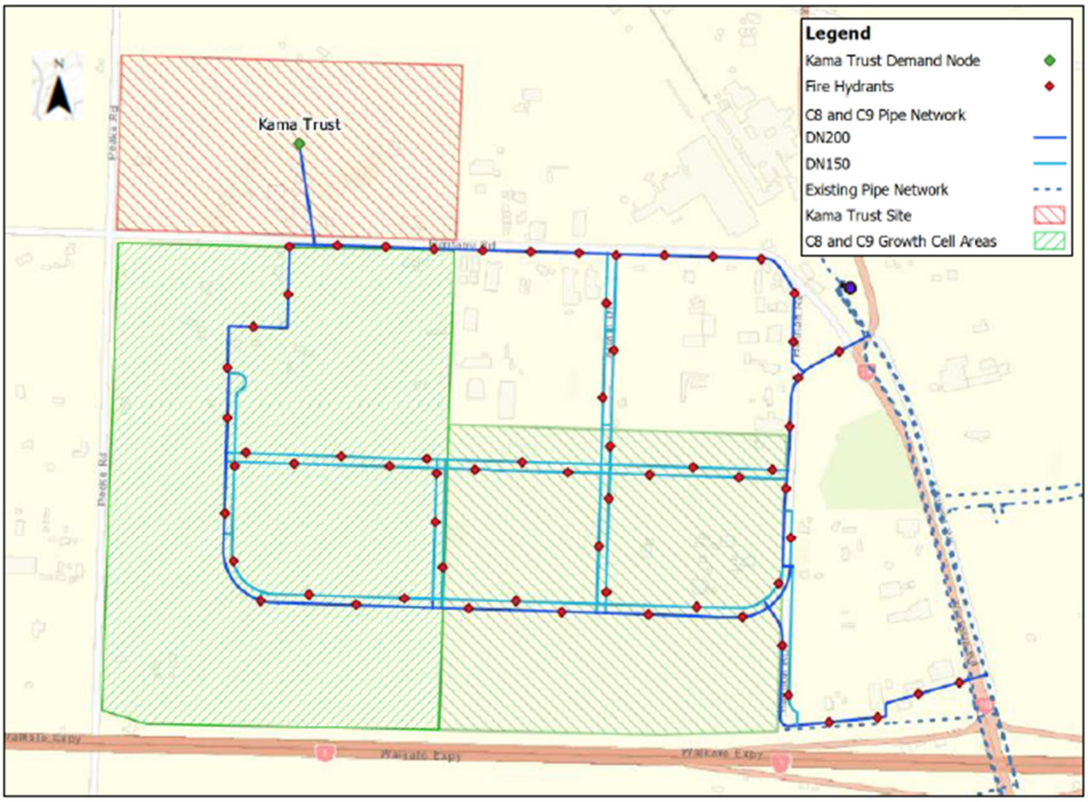

Figure 3.1: C8 and C9 Growth areas and Kama Trust area modelled water supply network (Appendix A)

3.4 Water Supply Design Parameters

Design parameters have been taken from the WSPs Hydraulic Analysis for Hautapu Industrial Kama Trust Plan Change Report (Appendix A) or the Regional Infrastructure Technical Specifications (RITS), Section 6 Water Supply, version May 2018.

Where suitable parameters that do not exist in the above documents, then parameters have been adopted with reference to NSZ 4404:2010 Land development and subdivision infrastructure, the Waipa District Development and Subdivision Manual 2015, or SNZ PAS4509:2008.

Table 3.1 below describes some of the key assumed parameters used in this assessment.

Table 3.1: Key Design Parameters for the Water Supply Assessment

Parameter	Value required by specification	Reference specification
Design Life	100 years	RITS 6.2.1
Fire Supply Service Level Required	FW3 to be provided for medium fire load business	SNZ PAS 4509 2008

Parameter	Value required by specification	Reference specification
FW3 Fire Water Requirements	Fire Water Requirements for FW3 as defined in Table 2 (Page 20) Water supply system to be designed to provide 60% of annual peak demand in addition to the fire flow demand	SNZ PAS 4509 2008, Table 2 SNZ PAS 4509 2008, Section 4.2
Industrial Water Demand	40 L/person/day Peaking Factor = 5	GD06 Table 18
Persons per area	45 people/ha	RITS Table 5-3

3.5 The water demand is based on Water Supply – Estimated Site Demand and Flows

Water demand calculations for the proposed industrial site are presented in this section (in accordance with the design parameters outlined in Section 3.4). Persons per area is taken at 45 people/ha with a medium density industrial area assumption. These factors would require further investigation at later stages in development when more detail on industrial land use is known.

The calculated industrial water use demand can be seen in Table 3.2.

Table 3.2: Industrial Water Use Calculation

Area (ha)	Demand (L)	Average Demand (L/s/ha)	Peak Demand (L/s)
17	17 Ha x 45 people/ Ha x 42 L/p/day 30, 600 L/Day	0.71 L/s (demand over a 12-hour period)	3.5 L/s (demand over a 12-hour period)

FW3 Fire water flows coupled with 60% of annual peak industrial flows will be required to service the development for firefighting purposes (as per the Table 3.1 parameters). The total flow rates are estimated in Table 3.3.

Table 3.3: Total Water Use Calculation

FW3 Fire Water Flow	60% of Annual Peak Industrial Flow	Total Expected Peak Fire Water Flow
50 L/s	2.13 L/s	52.13 L/s

3.5.1 Water Sensitive Design

Water sensitive design techniques such as rainwater harvesting and low-flow fittings and fixtures on the proposed development dwellings will be a key consideration in the master planning phase of this project, which may likely reduce the demand calculated in Section 3.5.

Industrial development typically has building typologies with relatively large roof areas, which provide an excellent means of collecting clean roof water to be re-used for both potable and non-potable sources.

3.5.2 Alternative Water Supply Servicing Options

On-site water supply via rainwater harvesting and alternative water sources (such as on-site bore water) provide potential alternative water servicing options, if there are issues arising from supplying water to the site via the Waipa District Council owned water reticulation.

4 WASTEWATER MANAGEMENT

4.1 Design Objectives

The following design objectives have been identified as outcomes for the design of the wastewater management system:

- Propose a workable conceptual wastewater servicing option(s) for industrial development of the site.

- Calculate wastewater requirements that meets the Waipa District Council (WDC) level of protection and level of service requirements (as defined within RITS).

4.2 Existing Wastewater Infrastructure

WDC GIS information indicates that the site is not currently serviced by any wastewater infrastructure.

4.3 Wastewater Servicing Concept

Potential site wastewater servicing options for the project site are listed below:

- Connect to the reticulated council wastewater system to be installed with the growth of cells C8 and C9 (preferred option).
- Standalone on-site land application treatment and disposal systems, developed on a site-specific/on-lot manner.

The option to connect to future Waipa District Council owned and maintained wastewater network is the preferred method. This is very likely to be the simplest, most cost-effective option and have the lowest environmental impact, compared to smaller bespoke on-site disposal systems.

Upon development of the C8 and C9 growth cells, a wastewater pumping station and subsequent council owned network will be installed[39]. This network will be connected to the existing Cambridge wastewater network, conveyed to the Cambridge wastewater treatment facility.

Both the Kama Trust industrial area and our project site area wastewater flows can be conceptually conveyed to the C8 and C9 growth cell wastewater system and pumping station(s). Topography on the site compared to the C8 and C9 growth cells is relatively flat, and the preferrable method of gravity-system wastewater conveyance will likely be feasible, pending the final depth and location of the proposed wastewater pumping stations within the C8 and C9 growth cells. During future design stages, this will need to be confirmed with the option of adding a local pump station for the project site/Kama Trust site a potential option.

The C8 and C9 growth cell wastewater pumping station(s) would be designed and built with capacity to accept wastewater from the Kama Trust site as well as the project site.

4.4 Wastewater Design Parameters

Design parameters have been taken from the 2015 Waipa District Development and Subdivision Manual (WDDSM) and the WLASS RITS, Section 5 Wastewater, Version May 2018. The WDDSM and RITS overlap in content with RITS outdating WDDSM.

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[39] Harrison Grierson 08/03/2022; Kama Trust 3 Waters Assessment Report

Table 4.1 describes key assumed parameters used in this wastewater assessment.

Table 4.1: Key Wastewater Design Parameters

Parameter	Value required by specification	Reference
Design life	100 years	RITS 5.2.1
Area	17 hectares	
Stormwater reserve area	15%	
Infiltration allowance	2250 l/ha/d	RITS 5.2.4.2
Surface water ingress allowance	16500 l/ha/day	RITS 5.2.4.2
Water Consumption	40 l/person/day	GD06 Table 18
Peaking factor	2.6	RITS Table 5-2
Persons per area (population equivalent)	45 persons/ha	RITS Table 5-3

4.5 Wastewater Disposal – Estimated Site Discharge Volumes and Flows

Wastewater flows for Average Daily Flow, Peak Daily Flow and Peak Wet Weather Flow scenarios were calculated using formula given in RITS Section 5.2.4.2. The formula can be seen in Figure 4.1.

Average Daily Flow (ADF)

The Average Daily Flow is calculated as the sum of the infiltration allowance and the daily wastewater flow:

Equation 5-1: Average daily flow (ADF)

$$ADF\ (m^3/day) = (infiltration\ allowance \times catchment\ area) + (water\ consumption \times population\ equivalent)$$

Peak Daily Flow (PDF)

The system shall achieve a daily self-cleaning velocity the Peak Daily Flow.

Equation 5-2: Peak daily flow (PDF)

$$PDF\ (L/sec) = ((infiltration\ allowance \times catchment\ area) + (peaking\ factor \times water\ consumption \times population\ equivalent)) \div 86400$$

Peak Wet Weather Flow (PWWF)

The system shall accommodate the design Peak Wet Weather Flow without surcharge.

Equation 5-3: Peak wet weather flow (PWWF)

$$PWWF\ (L/sec) = ((infiltration\ allowance \times catchment\ area) + (surface\ water\ ingress \times catchment\ area) + (peaking\ factor \times water\ consumption \times population\ equivalent)) \div 86400$$

Table 5-2: Wastewater Peaking Factors

Figure 4.1: Exert from RITS Section 5.2.4.2

Wastewater flows have been calculated allowing for a stormwater reserve area including the riparian buffer off the stream and the stormwater basin area. The calculated wastewater flows can be seen below.

Table 4.2: Calculated Wastewater Flows

Parameter	Value	Reference
Wastewater Catchment area	15.24 ha	Area - stormwater reserve area
Average Daily Flow (ADF)	61.72 m^3/day	RITS Equation 5-1
Peak Daily Flow (PDF)	1.21 L/sec	RITS Equation 5-2
Peak Wet Weather Flow (PWWF)	4.12 L/sec	RITS Equation 5-3

$$ADF = (2{,}250\ L/Ha/day\ x\ 15.2\ Ha) + (40\ L/person/day\ * 686\ population\ equivalent\ )$$

$$ADF = 34.29\ m^3/day + 27.4\ m^3/day$$

$$ADF = 61.7\ m^3/day$$

$$PDF = \frac{(2250\ L/Ha/day\ x15.2\ Ha) + (2.6\ PF\ x\ 40\ L\ /person\ /day\ \ x\ 686\ population\ equivalent)}{86{,}400}$$

$$PDF = 1.21\ L/sec$$

$$PWWF = \frac{(2250\ L/Ha/day\ x\ 15.2) + (16{,}500\ x\ 15.2) + \ (2.6\ \ PF\ x\ 40\ L\ /person\ /day\ \ x\ 686\ population\ equivalent)}{86{,}400}$$

$$PWWF = \frac{(34{,}289\ L\ /\ day) + (251{,}452\ L/day) + \ (102{,}000\ L/day)}{86{,}400}$$

$$PWWF = 4.12\ L/sec$$

4.5.1 Alternative Wastewater Servicing Options

On-site wastewater treatment and disposal systems (designed/implemented on a per Lot basis and operated and maintained by each property owner) are an alternative wastewater servicing solution, if there are issues arising from servicing the site via the Waipa District Council owned wastewater reticulation.

5 STORMWATER MANAGEMENT

5.1 Design Objectives

The following design objectives have been identified as outcomes for the design of the stormwater management system:

- Develop a conceptual overall stormwater management system for the sites development that provides site drainage and flood protection, while aligning with other urban design elements of the development.
- Achieve sustainable and environmental outcomes through incorporation of water sensitive design elements into the built environment.
- Design a conceptual stormwater system that meets the Waipa District Council (WDC) level of protection and level of service requirements (as defined within RITS).
- Give effect to Te Mana o te Wai by prioritising the health and well-being of the receiving water bodies and freshwater ecosystems by designing a stormwater management system that is consistent with this approach.
- Consider water sensitive design objectives and considers stormwater management in parallel with the ecology of a site, best practice urban design, the Te Aranga Design Principles and community values.
- Provide preliminary location, layout and size of the required stormwater management infrastructure for preliminary consideration and stakeholder engagement.

5.2 Catchment Assessment

The site sits directly adjacent to the Mangaone Stream, at the upper reaches of its catchment. The Mangaone Stream originates approximately 6 km south east of the site and continues downstream of the site where it is joined by the Mangaomapu Stream, discharging into the Waikato River at the south east edge of Hamilton city, a further 14 km downstream of site. The approximate upper catchment of the Mangaone Stream from the site is 1560 ha while the total catchment of the Mangaone Stream is 12500 ha. The site in relation to the Mangaone Stream, its catchment, its upper catchment and the Waikato River can be seen in Figure 5.1.

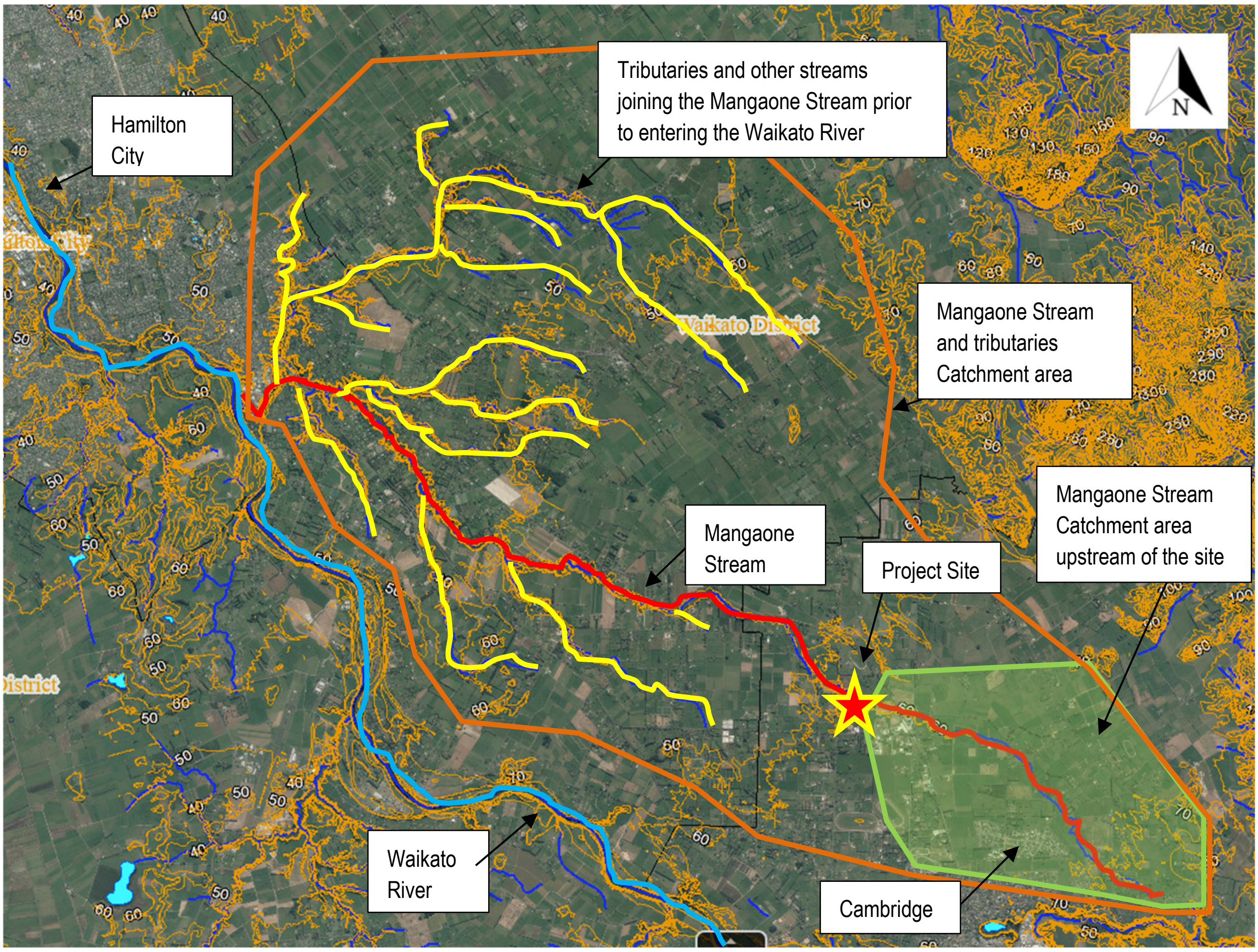

Figure 5.1: Site in Relation to Waterways and Upstream Catchment

The site is relatively flat with a slight fall toward the Mangaone Stream along the northern boundary. The site contours and overland flow paths can be seen in Figure 5.2.

Immediately downstream of the site is the Peake Road culvert. This culvert is of unknown size at this level of investigation. This potential constraint suggests the downstream receiving environment requires further investigation in later stages of development and design. The location of the Peake Road culvert can be seen in Figure 5.2.

Current localised drainage in the area includes a minor roadside drain along the western edge of Peake Road capturing run-off from the western side of Peake Road and this area of the road itself. This prevents any cross-boundary flows to the site occurring from the west.

There is a natural depression to the east of the centre of the site that collects run-off from the site and directs it to the Mangaone Stream. There is an elevated area in the centre of the site creating localised diversion of stormwater that will effectively be directed around the elevated area and on to the Mangaone Stream. The natural elevation and depression features can be seen on Figure 5.2.

The site contains multiple tree lines and stands of trees that are not affecting the flow of stormwater.

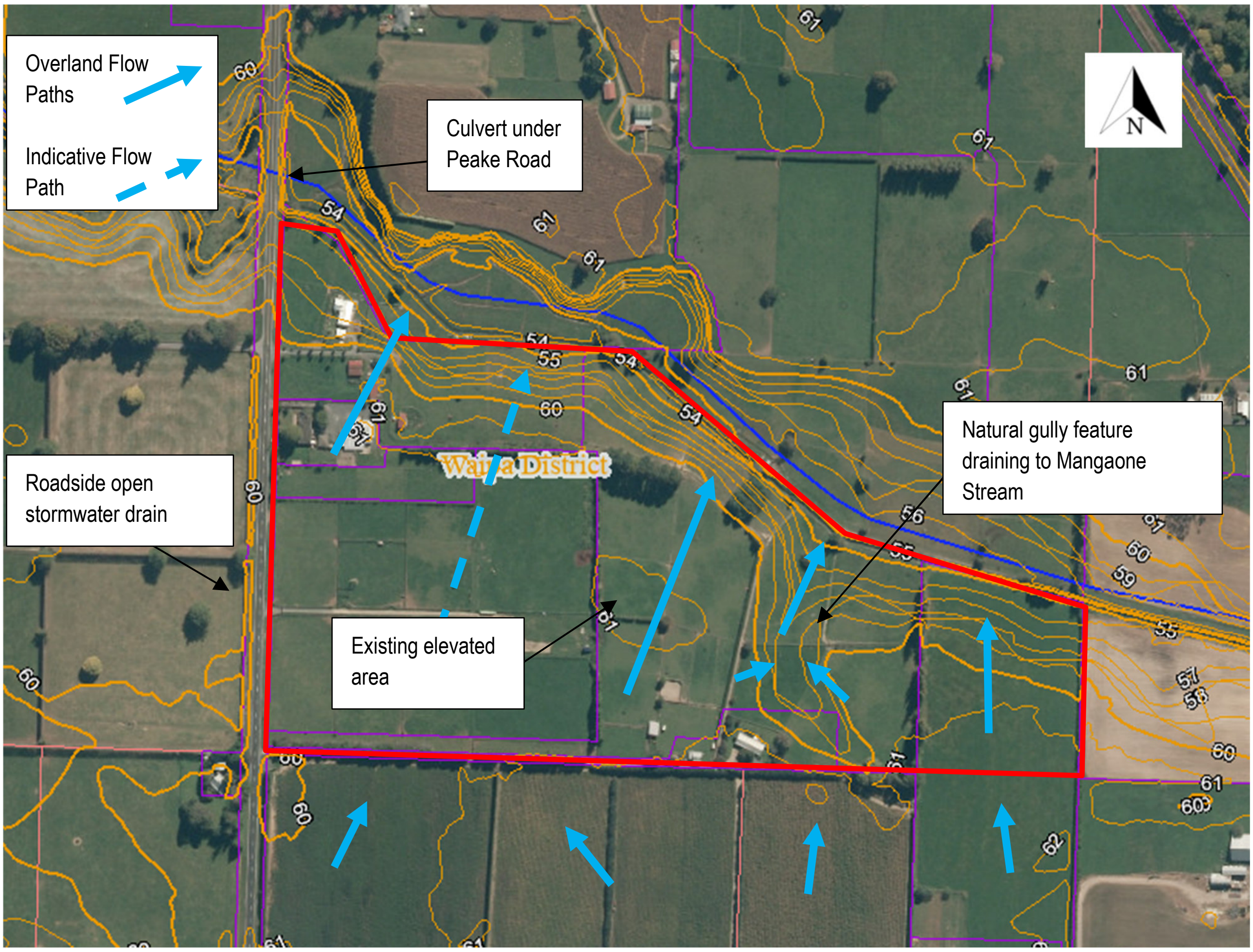

Figure 5.2: Site Contours and Overland Flow Paths

The estimated groundwater depth for the C8 and C9 growth cells, and into the Kama Trust area can be seen on Figure 5.3. With assumption being that the groundwater for the site follows the indicative profiles, is at a depth greater than 4 m.

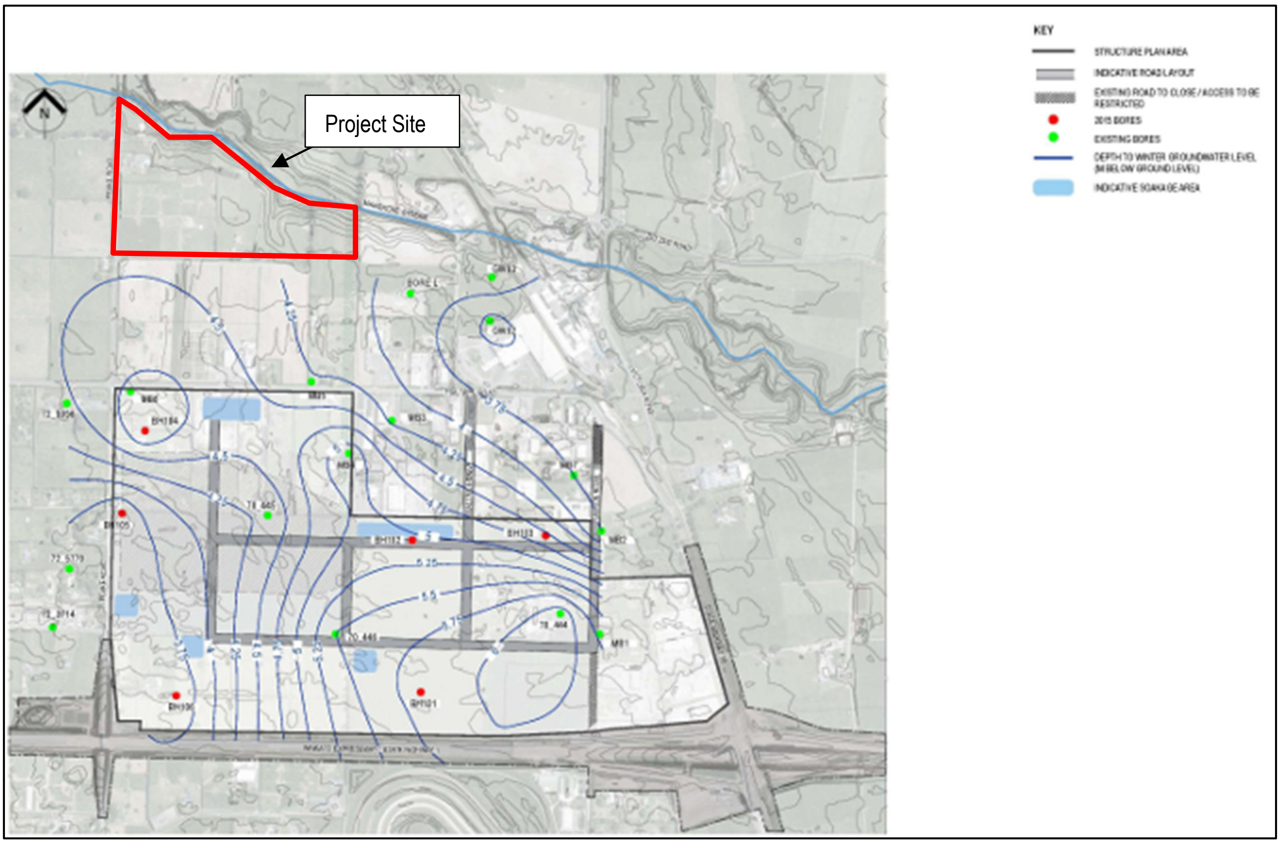

Figure 5.3: Estimated Winter Groundwater Contours – Appendix S5: Hautapu Industrial Structure Plan, Urban Design and Landscape Guidelines 2019

A desktop review of the Mangaone Stream and the downstream receiving environment shows several low-lying areas adjacent to the site which are consistent with indicators of natural wetlands. Further assessment should be undertaken to confirm whether the areas meet the definition of a natural inland wetland to determine whether the requirements of the National Environmental Standards for Freshwater (NES-F) are applicable to site development.

5.3 Existing Stormwater Infrastructure

WDC GIS information indicates that the site is not currently serviced by any stormwater infrastructure.

5.4 Desktop Flood Hazard Assessment

An initial review of the Waikato Regional Council Hazards Portal confirmed that the site is outside of the region wide flooding extents and flood management areas, as shown below.

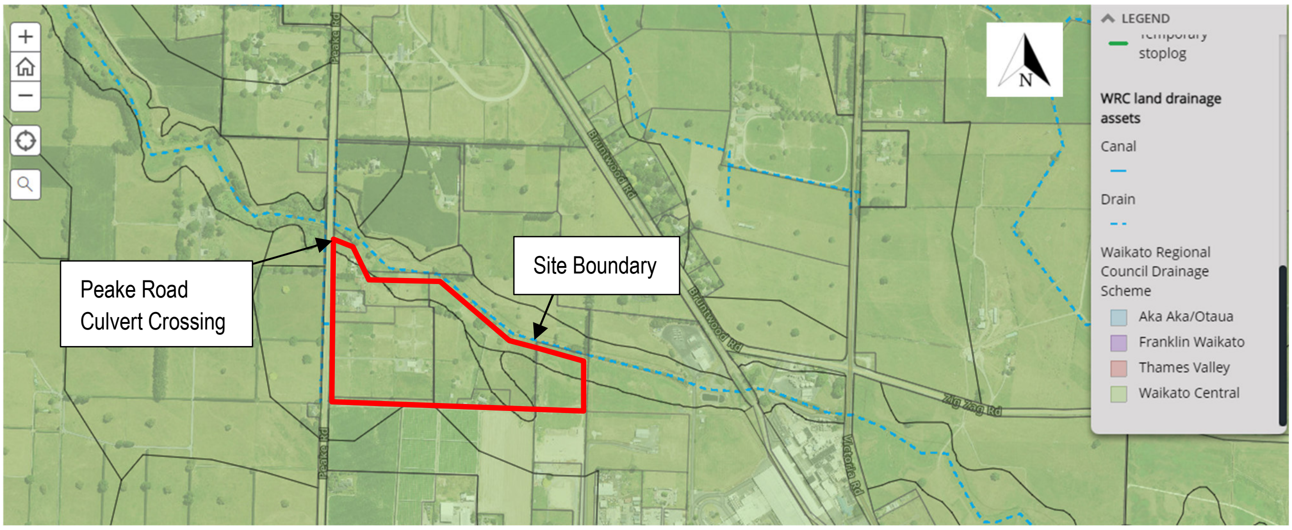

Figure 5.4: River Flood Extents – Waikato Regional Council Hazards Portal – River Flooding (Sourced 13/1/2023)

It is anticipated that the site is elevated from topwater levels within the adjacent Mangaone Stream, which are likely set by headwater levels by the downstream culvert at the Peake Road Crossing. The site is generally elevated from the Peake Road crossing (approx RL 55) which would likely set flood water levels should this culvert be blocked or undersized to convey stream flows.

The existing local site drainage system would require upgrades to mitigate local ponding and provide appropriate primary and secondary level of service for the proposed development.

No further flood assessment has been undertaken for the site at the time of this report, however more detailed assessment will likely be required during future design.

5.5 Design Criteria

A preliminary stormwater design criteria for the proposed development has been determined from the catchment review based on the above considerations as outlined below in Table 5.1.

Table 5.1: Stormwater Design Criteria

DESIGN OBJECTIVES	DESIGN CRITERIA	DESIGN PARAMATERS	REFERENCE
STORMWATER MANAGEMENT			
Flood Hazard Management	Required	Flood hazard mitigation provided in accordance with RITS level of protection requirements. Secondary flow routes for major design storms via roads corridors into adjacent stormwater areas.	RITS
Flood Control	Maybe Required	Detention required, limiting the post-development 100 year ARI event flow rates to 80% of the pre-development 100 year ARI event flow rates. (No known downstream flooding issues however an assessment of downstream constraints and effects has yet to be undertaken)	RITS / WRC Stormwater Management Guide

DESIGN OBJECTIVES	DESIGN CRITERIA	DESIGN PARAMATERS	REFERENCE
Flow Attenuation	Required	Match pre-development flow rates for the 2 and 10 year ARI events through controlled attenuation and multi stage outlets or devices that reduce the run-off flow.	RITS / WRC Stormwater Management Guide
Water Quality Treatment	Required	Water quality treatment proposed for all hardstand surfaces. 1/3 of 2 year 24-hour ARI rainfall depth with climate change used to calculate water quality volume (WQV)	RITS / WRC Stormwater Management Guide
Extended Detention / Stream Erosion	Required	Discharge is into a natural stream or modified channel.	RITS / WRC Stormwater Management Guide
Volume	Required	Required when discharge is into a natural stream or modified channel Match pre-development volume run-off through reduced run-off practices and sub-catchment management	RITS
Retention / Groundwater Recharge	Required	Minimum retention of pre-development initial abstraction.	WRC Stormwater Management Guidelines, AC GD07
Natural Wetlands	Potentially Required	No known natural wetlands onsite, however areas of low lying terrain are typical of natural wetlands. Further investigation is recommended to delineate onsite natural wetlands. If natural wetlands are present onsite, the development should ensure that the pre-development hydrology (surface water and ground water) is maintained post development.	NES-F
Manage Cross Boundary SW Flows	Required	The concept design of the drainage system should consider and allow to maintain cross boundary flow for upstream catchment areas.	NZS4404
STORMWATER RETICULATION			
Primary drainage system	10 - year ARI design storm.	Infrastructure design will be undertaken using the RCP 6.0 climate change scenario for rainfall intensity. Capacity to convey the design storm without surcharge.	RITS
Outlet Scour Protection – Energy Dissipation/Rip rap apron	10-year ARI design storm	Scour protection at reticulation outlets using energy dissipaters and rock rip rap.	WRC TR2018
WATERWAYS/WETLANDS			
Te Mana o Te Wai	The vital importance of water.	▪ Tangata Whenua are actively involved in freshwater management. Identify the local approach to giving effect to Te Mana o Te Wai. ▪ Prioritise the health and well being of water bodies, then the essential needs of people, followed by other users ▪ Improve degraded water bodies and maintain or improve all others	NPS FW/ Te Aranga Principles

DESIGN OBJECTIVES	DESIGN CRITERIA	DESIGN PARAMATERS	REFERENCE
Stream Restoration	Ecological and Habitat Impact Assessment	Stream daylighting, restoration and planting of waterway. Enhanced habitat values and net ecological gains through restoration of the water course and riparian margins. Access points to stream to be provided.	NPS FW/ Te Aranga Principles
Protection and enhancement of wetlands	Required	Enhanced habitat values and net ecological gains through protections and restoration of any wetlands Access points to water bodies be provided. May require consent for diversion of water within 100m of a wetland.	NPS FW/ Te Aranga Principles
Mahinga Kai	Kai is safe to eat and harvest	Promote restoration of environment to support Mahinga Kai	NPS FW/ Te Aranga Principles
Māori names are celebrated	NA	Tangata whenua involvement in naming of reserve areas	Te Aranga Principles

5.6 Design Philosophy

The following design philosophies have been a focus throughout the development of this concept:

- Give effect to Te Mana o te Wai by prioritising the health and well-being of the receiving water bodies and freshwater ecosystems by designing a stormwater management system that protects the receiving environment from the effects of the development.
- Consideration of water sensitive design objectives to consider stormwater management in parallel with site ecology, urban design, and community values.
- Design post development surface water and groundwater infiltration and recharge patterns to maintain pre-development hydrology ensuring no global effects on the receiving environment.
- Consideration of adjacent development areas (Kama Trust block) to align stormwater management infrastructure and provide global efficiencies.

5.7 Methodology

The stormwater management design approach has been prepared with consideration of the physical environment, catchment assessment, design criteria and philosophy of the proposed development. Consultation with Tangata Whenua regarding the stormwater management system has yet to be undertaken.

Using the design objectives (Section 5.1) and design criteria (Section 5.5) identified, a system design summary (Section 5.8.1) has been prepared outlining the operation of the components of the system.

The following options were explored during consideration of the proposed management infrastructure:

- Discharge to ground via soakage
  - Capture and conveyance to a central soakage basin for discharge to ground (Method chosen for Kama Trust and C8/C9 growth cells)

  - At source disposal through the development of a small soakage systems and infrastructure.
- Discharge treated stormwater into the Mangaone Stream with appropriate stormwater management system to provide water quality treatment, extended detention, flow attenuation etc.

The preferred stormwater design is to collect and manage stormwater run-off within a centralised stormwater management facility with a controlled discharge into the Mangaone Stream. Groundwater recharge and reduction in stormwater volume will be achieved by at source soakage systems discharging run-off to ground. This stormwater management solution is discussed in Section 5.8.5, labelled as 'Option 1'.

The option to retain all runoff and discharge to ground through soakage has been explored. The stormwater for a 5 year and 10-year frequency design storm will be collected in the soakage basin and soaked directly to ground. The runoff from a 100-year storm will be collected and soaked with the added discharge at 80% of pre development rates via a spillway. This stormwater management solution is discussed in Section 5.8.6, labelled as 'Option 2'.

Preliminary hydrological and hydraulic modelling has been undertaken for both stormwater management options to identify the approximate footprint of the required infrastructure and facilities. The required infrastructure has been positioned logically considering the natural topography, existing flow paths, site constraints and urban design strategy and the infrastructure footprint proportioned based on the contributing catchment size.

This process has been iterated to arrive on the Stormwater Management General Layout Plans presented in Appendix B.

5.8 Stormwater Management Design

5.8.1 Stormwater Design Summary

The stormwater management system design for both options of stormwater disposal follows through the same methodology for all steps up to the final collection and disposal in the wetland/soakage basin. The stormwater management system design summary is presented in Table 5.2 which summarises the proposed implementation of the various system components.

Table 5.2: Stormwater Management System Design Summary

Stormwater Management Objective	Description
Industrial lots	Onsite soakage of pre-development initial abstraction with overflow directed to public reticulation discharging into integrated stormwater management facility. Requirement to be managed by consent notice.
Public Roads	Roadside collection via network of catchpits discharging into integrated stormwater management facility. Soakage of pre-development initial abstraction in the drainage channel taking flows to the constructed wetlands.
Retention/Groundwater Recharge	Soakage to ground of pre-development initial abstraction. Spread over site with soakage at multiple points to maintain pre-development hydrology. Ensure post development hydrology remains consistent with pre-development hydrology in consideration to surface water flows and groundwater recharge.

Stormwater Management Objective	Description
Primary Conveyance System	Catchpits and piped reticulation to the drainage channel, discharging to the stormwater management facility.
Secondary Conveyance System	Roads and/or channels grading secondary flows towards appropriate stormwater management features via dedicated flow paths.
Water Quality Treatment	Water quality treatment provided within catchment specific centralised integrated constructed wetland/soakage basin.
Extended Detention – Option 1	Extended detention provided within catchment specific centralised integrated constructed wetland.
Attenuation – Option 1	Flow attenuation (Restrict peak post-development flows to peak pre-development flows) of the 2 and 10 year ARI events within two catchment specific centralised integrated constructed wetland.
Flood Control – Option 1	Detention limiting the post-development 100 year ARI event flow rates to the pre-development 100 year ARI event flow rates within centralised integrated constructed wetland. Flood control (attenuation of 1% AEP post development flows to pre development discharge rates) has been included in the initial design however existing flood hazard mapping indicates that the downstream receiving environment is not subject to flooding and subject to further assessment of the downstream effects, flood control may not be required.
Extended Detention – Option 2	Extended detention provided within catchment specific centralised integrated constructed soakage basin.
Attenuation – Option 2	Flow attenuation (Retain post-development flows within the constructed wetland system) of the 2 and 10 year ARI events as well as the 100 year ARI event to 80% of pre development flows, within two catchment specific centralised integrated constructed wetland.
Flood Control – Option 2	Detention limiting the post-development 100 year ARI event flow rates to 80% of the pre-development 100 year ARI event flow rates within centralised integrated constructed wetland. Flood control (attenuation of 1% AEP post development flows to 80% of pre development discharge rates)

5.8.2 Retention / Groundwater Recharge

If the site is required to be hydraulically neutral, the development shall be required to match the pre-development hydrology including the pre-development groundwater recharge. This will be dependent on the downstream receiving environments, the presence of any natural wetlands and downstream constraints. The current assumption is that the groundwater recharge will consist of the initial abstraction.

The WRC Stormwater Management Guidelines recommend retention of the difference between the pre and post development total volume of run-off for smaller storms up to the 2-year ARI design storm and specify a minimum retention volume of the predevelopment initial abstraction.

Given the possible presence of natural wetlands downstream of site any proposed development will be subject to consent (non-complying) under the NES-F for diversion of water within 100m in which the applicant needs to demonstrate the proposed development has less than minor effect on the natural wetlands. This requires careful assessment of the existing wetland hydrology to understand the existing conditions, however generally the post development landform needs to replicate the pre-development wetland hydrology.

It is anticipated that bulk earthworks will be required to modify the existing landform to be suitable for development and is noted that the compaction associated with bulk earthworks decreases the natural permeability of the soil. It is noted that the initial geotechnical assessment has identified that the natural soils have a variable permeability and further soil assessment is required to understand the suitability of the ground for soakage to ground.

For master planning purposes, it is intended that groundwater recharge will primarily be achieved via retention and discharge to ground of onsite run-off. This provides a scattering of at source discharge points which replicate pre-development infiltration patterns and is preferable to fewer larger soakage systems.

Further assessment needs to be undertaken to understand the hydrology of any existing natural wetlands to ensure their baseline conditions are maintained and more detailed stormwater assessment will need to be undertaken to determine the post development hydrology is equivalent to the pre-development hydrology and whether hydraulic neutrality is a requirement.

5.8.3 Industrial Lot Stormwater

Each industrial lot will manage the soakage of the predevelopment initial abstraction with onsite soakage. This will be managed through a consent notice on each lot.

Excess run-off will be directed via controlled discharge to the road reticulations and directed to the drainage channel and into the constructed wetland.

5.8.4 Public Road Stormwater & Drainage Channel

A central drainage channel will be utilised to collect run-off from the roads and overland flow paths and provide open channel conveyance to centralised stormwater management areas.

The drainage channel will be designed to provide soakage of the initial abstraction for run-off from the public road areas whilst conveying larger flows to the centralised stormwater management areas. Localised stormwater pipe network will likely be needed to provide connections to the stormwater network.

5.8.5 Constructed Wetlands – Stormwater Management System Option 1

Constructed wetlands are proposed to provide integrated stormwater management for the site for the option 1 stormwater management system scenario.

The constructed wetlands shall be designed to provide the following stormwater management functions;

- Water Quality Treatment
- Extended Detention/Stream Erosion Protection
- Flow Attenuation
- Flood Protection (If required)

For master planning purposes, preliminary hydrological and hydraulic modelling has been undertaken to provide concept location, layout and size of the required constructed wetlands assuming flow attenuation (2 and 10 year ARI events) and flood control to limit the post development flow rates the pre-development 100-year ARI event are required (Given no downstream flooding issues are known attenuation of 80% of predevelopment flows has not been considered necessary for concept design purposes).

A summary of the hydrological and hydraulic assessment including methodology, input parameters and results is provided in Appendix C.

Preliminary results indicate a total storage volume of 12,600 m^3 between two facilities is required to provide the appropriate stormwater management.

The overall volume will be achieved through two management facilities located at catchment low points and integrated into the broader urban and landscape design. The respective management area has been proportioned based on the contributing catchment size. Table 5.3 provides a summary of the volumetric requirements. It is noted that preliminary modelling hasn't allowed for retention and efficiencies between flow attenuation and extended detention. A summary of the volumetric requirements for each catchment is provided in Table E 7 within Appendix E.

Table 5.3: Summary of Wetland Volumes

Design Parameter	Wetland 1 Volume (m3)	Wetland 2 Volume (m3)
Attenuation & flood control	3960	2500
Stream protection (EDV)	2500	1500
Water Quality Treatment (WQV)	1400	830
Total (nearest 100m^3)	**7800**	**4800**

For purposes of allocating a footprint to the constructed wetlands, an average maximum storage depth of 1.5 m (including 0.3 m dead zone for water treatment) has been assumed providing an approximate footprint for each area. Preliminary stormwater layout plans indicating the size and location of the infrastructure is included below in Figure 5.5 and attached in Appendix B.

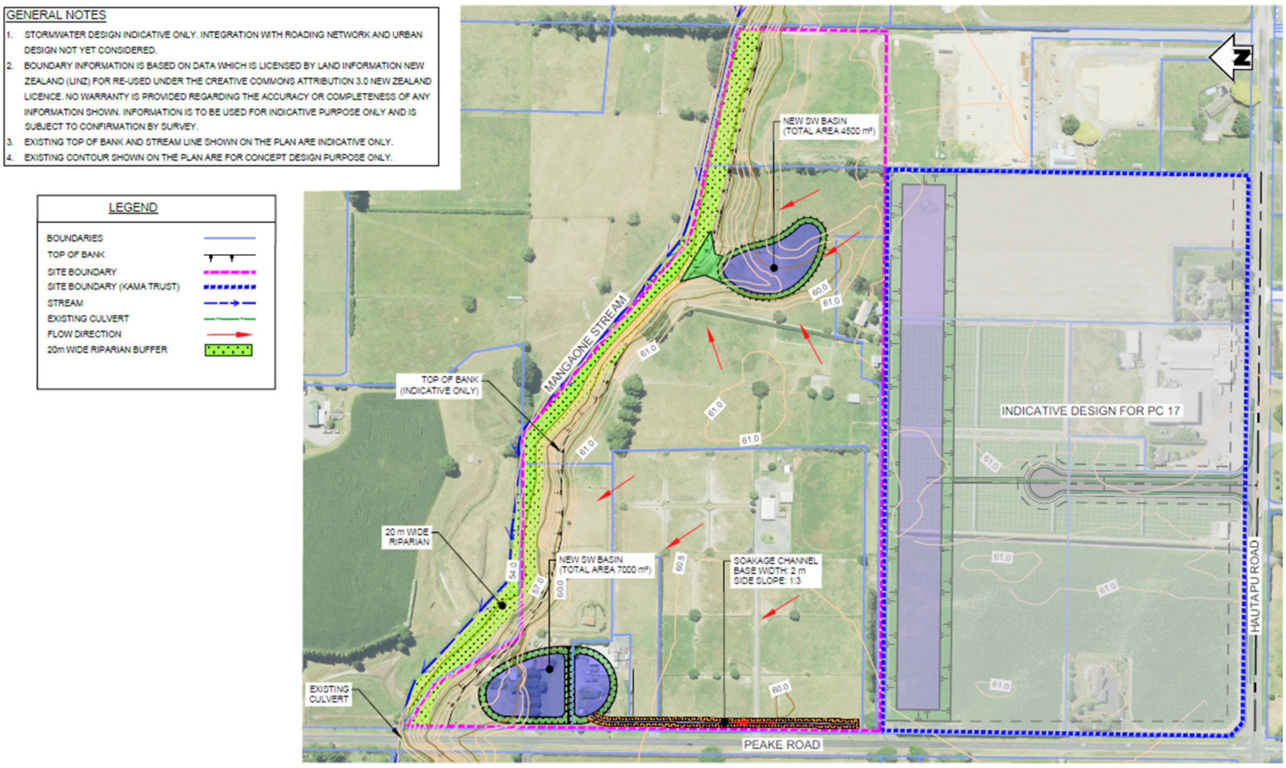

Figure 5.5: Potential Stormwater Basin Location

Further design should be undertaken to determine the final size, arrangement and details of the constructed wetland areas. Stormwater infrastructure should be further integrated with the proposed roading network and global urban design during future stages of assessment.

Sensitivity testing has not been undertaken in this assessment to understand the effect of RCP 8.0 rainfall data on the development proposal. This should be undertaken during the next stages of assessment as may impact the sizes of the proposed constructed wetlands.

5.8.6 Soakage Basin – Stormwater Management System Option 2

Constructed soakage basins are proposed to provide integrated stormwater management for the site for the option 2 stormwater management system scenario.

The soakage basins shall be designed to provide the following stormwater management functions;

- Water Quality Treatment
- Extended Detention
- Flow Attenuation
- Flood Protection

For master planning purposes, preliminary hydrological and hydraulic modelling has been undertaken to provide concept location, layout and size of the required constructed soakage basins assuming flow attenuation (2 and 10 year ARI events) and flood control to limit the post development flow rates to 80% of the pre-development 100-year ARI event are required. This necessity is based on potential downstream flooding, reducing additional consents required and potential to ease any strain on a potential heavy loaded waterway.

The soakage basins are designed to incorporate a spillway to release flows to overland flow paths at 80% of pre development flow in a 1% AEP design storm. Stormwater soakage basins have been recommended to be located near to the stream so overland flow paths will lead directly to the Mangaone Stream and not cause any harm to neighbouring properties including buildings.

A summary of the hydrological and hydraulic assessment including methodology, input parameters and results is provided in Appendix D.

Preliminary results indicate a total storage volume of 17,600 m^3 between two facilities is required to provide the appropriate stormwater management.

The overall volume will be achieved through two management facilities located at catchment low points and integrated into the broader urban and landscape design. The respective management area has been proportioned based on the contributing catchment size. Table 5.4 provides a summary of the volumetric requirements. It is noted that preliminary modelling hasn't allowed for retention and efficiencies between flow attenuation and extended detention. A summary of the volumetric requirements for each catchment is provided in Table E 7 within Appendix D.

Table 5.4: Summary of Soakage Basin Volumes

Design Parameter	Soakage Basin 1	Soakage Basin 2
Volume, m^3	10700	6800
Surface Area, m^2	9500	6350

For purposes of allocating a footprint to the constructed soakage basins, an average maximum storage depth of 1.2 m has been assumed providing an approximate footprint for each area. Preliminary stormwater layout plans indicating the size and location of the infrastructure is included below in Figure 5.6 and attached in Appendix B.

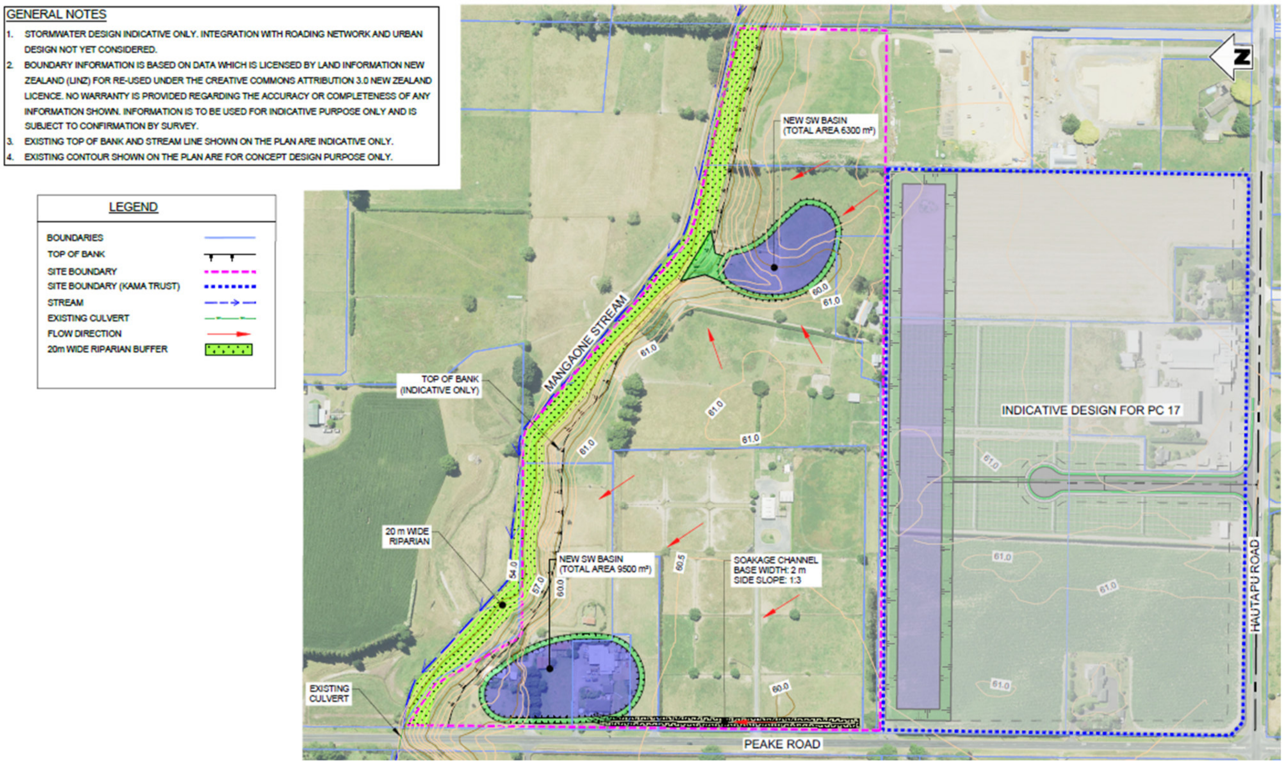

Figure 5.6: Potential Stormwater Soakage Basin Locations

Further design should be undertaken to determine the final size, arrangement and details of the soakage basin areas. Stormwater infrastructure should be further integrated with the proposed roading network and global urban design during future stages of assessment.

Sensitivity testing has not been undertaken in this assessment to understand the effect of RCP 8.0 rainfall data on the development proposal. This should be undertaken during the next stages of assessment as may impact the sizes of the proposed constructed wetlands.

5.9 Ecological Enhancement Opportunities

With the project site lying adjacent to the Mangaone Stream, the development provides opportunities for ecological enhancement.

5.9.1 Mangaone Stream Riparian Zone

With the inclusion of a stormwater basin on the site with discharge to the Mangaone Stream, there is opportunity for ecological enhancement. It is recommended a 20 m buffer corridor from the stream be allowed along the stream to protect the biodiversity, protect erosion, potential for riparian planting and generally ecologically enhance the boundary of the stream.

Any silts and residue within secondary overland flows will be captured in the buffer zone and not enter the stream. Water quality of the stream itself will be enhanced with additional plant growth.

5.10 Stormwater Recommendations

With the stormwater assessment conducted on the site, the following recommendations and conclusions are drawn:

- The site is not located in a known flood hazard zone and is generally elevated from the adjacent Mangaone Stream.
- Stormwater management infrastructure can be developed to provide an appropriate level of service and level of protection for the proposed development. The following disposal options are considered feasible for the site subject to further assessment:
  - Capture and conveyance to a central soakage basin for discharge to ground (Method chosen for Kama Trust and C8/C9 growth cells)
  - At source disposal through the development of a small soakage systems and infrastructure.
  - Discharge treated stormwater into the Mangaone Stream with an appropriate stormwater management system.
- The preferred stormwater design is to collect and manage stormwater run-off within a centralised stormwater management facility with a controlled discharge into the Mangaone Stream. Site specific design criteria was developed from undertaken a broader catchment assessment which indicated the system would be required to provide water quality treatment, extended detention, flow attenuation. Initial assessment indicated that flood control (attenuation of 1% AEP flows) will not be required however this is subject to further assessment of downstream effects.
- The alternate stormwater design is to collect and manage stormwater run-off within a centralised stormwater management facility with soakage to ground of the 50% and 10% AEP flows with discharge to 80% of predevelopment flows in a 1% AEP storm event via spillway to receiving environments. This stormwater management solution provides water quality treatment, extended detention and flow attenuation in line with the solution for the Kama Trust area and C8/C9 growth cells.
- Preliminary engineering design and hydrological and hydraulic modelling has been undertaken to identify the location, layout and size of the required stormwater management infrastructure for preliminary consideration and stakeholder engagement. Layout drawings are provided in Appendix B.
- Further assessment should be undertaken during future design stages to understand downstream constrains (and design parameters), topwater levels within the Managone Stream and integrate the proposed stormwater management system with roading network and other urban design considerations.

6 PLAN CHANGE 17 STORMWATER INTEGRATION OPPORTUNITIES

The site is adjacent the proposed Plan Change 17 site (Kama Trust) which is currently proposing rezoning to Deferred Industrial Zoning.

Under their existing proposal the Kama Trust block proposes to provide stormwater management infrastructure onsite via discharge to land in significantly sized soakage basins.

Integrated catchment wide stormwater management infrastructure could manage stormwater runoff from the proposed site and the Kama Trust development block.

The establishment of a discharge point into the Mangaone Stream allows for a much higher discharge rates than can be achieved via discharge to ground within the soakage systems currently proposed in the Kama Trust Block. This significantly reduces the footprint and size of the system required to attenuate post development flows.

Conceptual engineering design and hydrological and hydraulic modelling has been undertaken to understand the location, layout and size of the required stormwater management infrastructure for an integrated system and layout drawings are provided in Appendix B. The current PC17 proposal and an integrated layout design are compared in Figure 6.1.

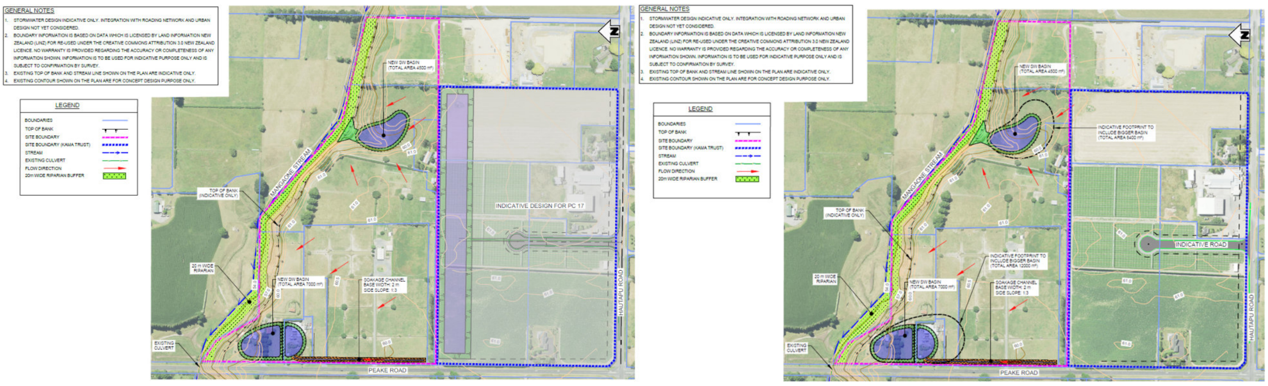

Figure 6.1: Independent stormwater solutions (Left). Integrated stormwater solution (Right)

The soakage basin shown in the Kama Trust area in Figure 6.1 is the integrated solution combining the proposed Basin 4 from the C9 growth cell with the Kama Trust soakage basin. Due to the pre development C9 overland flows not directly entering the Mangaone Stream, this basin volume has been removed from the scenario with the presumption being Basin 4 to be relocated back to its original proposed location as per WDC C8-C9 Master Plan. Other alternatives to this can be explored in future design phases.

An integrated approach would provide a more efficient and resilient system which discharges and overflows into the Mangaone Stream. It would avoid needing to create expensive soakage infrastructure decreasing the footprint, operation and maintenance costs and providing increased land use efficiency.

7 CONCLUSION

This assessment has reviewed the site and catchment characteristics with the objective of consolidating and defining the site-specific design parameters for the three waters requirements and the geotechnical suitability of the site for industrial development.

The following conclusions are provided from this assessment:

- Geotechnical assessment:
  - Based on a preliminary desktop review of the site; the site soils are deemed to be generally suitable to support development. The foundation soils are sufficient to support light weight height portal framed building with low to moderate foundation bearing demands with only minor shallow (< 1 m) subgrade improvement layers. Deep piles or excavation and replacement with imported hardfill is not envisaged.
  - Onsite soakage will be an option for the development provided that a secondary overland flow path is provided for.
  - Liquefaction is not expected to be a site hazard nor is lateral spread. The main thread will be the consolation of soft soils at depth especially if the soil sequence consists of bar-swale/organic soils in a structure’s footprint.
- Water Supply:
  - Water supply for the site is proposed to be serviced via integration with the Kama Trust sites connection to the C8/C9 growth cells council owned and maintained water supply.
  - Water supply usage is calculated with the required FW3 fire water flow of 50 L/s and 60% of the annual peak industrial flow of 3.34 L/s, the total water flow for the site is calculated at 53.34L/s.
- Wastewater:
  - Wastewater disposal for the site is proposed to be serviced via integration with the Kama Trust conveyance system to the C8/C9 growth cells council owned and maintained wastewater reticulation and C8/C9 pumping station(s), which subsequently pump flows through to the Cambridge wastewater treatment plant.
  - The conveyance system to the C8/C9 pumping station(s) may conceptually be via a gravity reticulation (preferred), or a local project site/Kama Trust site pumping station and rising main to the C8/C9 pumping station(s), pending future design phases.
  - Wastewater flows are calculated with an average daily flow of 75.44 m^3/day, peak daily flow of 1.61 L/sec and peak wet weather flow of 4.52 L/sec.
- Stormwater:
  - The site is not located in a known flood hazard zone and is generally elevated from the adjacent Mangaone Stream.
  - Stormwater management infrastructure can be developed to provide an appropriate level of service and level of protection for the proposed development.
  - The preferred stormwater design is to collect and manage stormwater run-off within a centralised constructed wetland/s with a controlled discharge into the Mangaone Stream.

- Alternate stormwater design is to collect and manage stormwater run-off within a centralised constructed soakage basin/s with 1% AEP flows released by spillway to receiving environments at 80% of predevelopment flows.

- Ecological Enhancement Opportunities include:
  - Opportunity exists for ecological enhancement along the Mangaone Stream with the introduction of a buffer zone containing biological diversity and riparian planting.

- Plan Change 17 Stormwater Integration Opportunities
  - Integrated catchment wide stormwater management infrastructure could manage stormwater runoff from the proposed site and the Kama Trust development block.
  - The establishment of a discharge point into the Mangaone Stream allows for a much higher discharge rates than can be achieved via discharge to ground within the soakage systems currently proposed in the Kama Trust Block. This significantly reduces the footprint and size of the system required to attenuate post development flows.
  - An integrated approach would provide a more efficient and resilient system which discharges and overflows into the Mangaone Stream. It would avoid needing to create expensive soakage infrastructure decreasing the footprint, operation and maintenance costs and providing increased land use efficiency.

APPENDIX A WATER SUPPLY HYDRAULIC ASSESSMENT FOR HAUTAPU INDUSTRIAL KAMA TRUST PLAN CHANGE REPORT BY WSP

Memorandum

To	Kama Trust Limited
Copy	Josy Cooper, Daniel Johnson, Ryan Mackinnon
From	Sanjana Prakash, Haiming Li
Office	Hamilton
Date	12 July 2022
File/Ref	3-39610.00
Subject	Water Supply Hydraulic Assessment for Hautapu Industrial Kama Trust Plan Change
Status	Final

1 Introduction

Kama Trust proposes a new industrial site on 98 - 108 Hautapu Road & 326 – 342 Peake Road, Cambridge. The proposed site is approximately 13 hectares and is located north of the C9 growth cell area. *Figure 1-1* shows the site location and the locations of C8 and C9 growth cells.

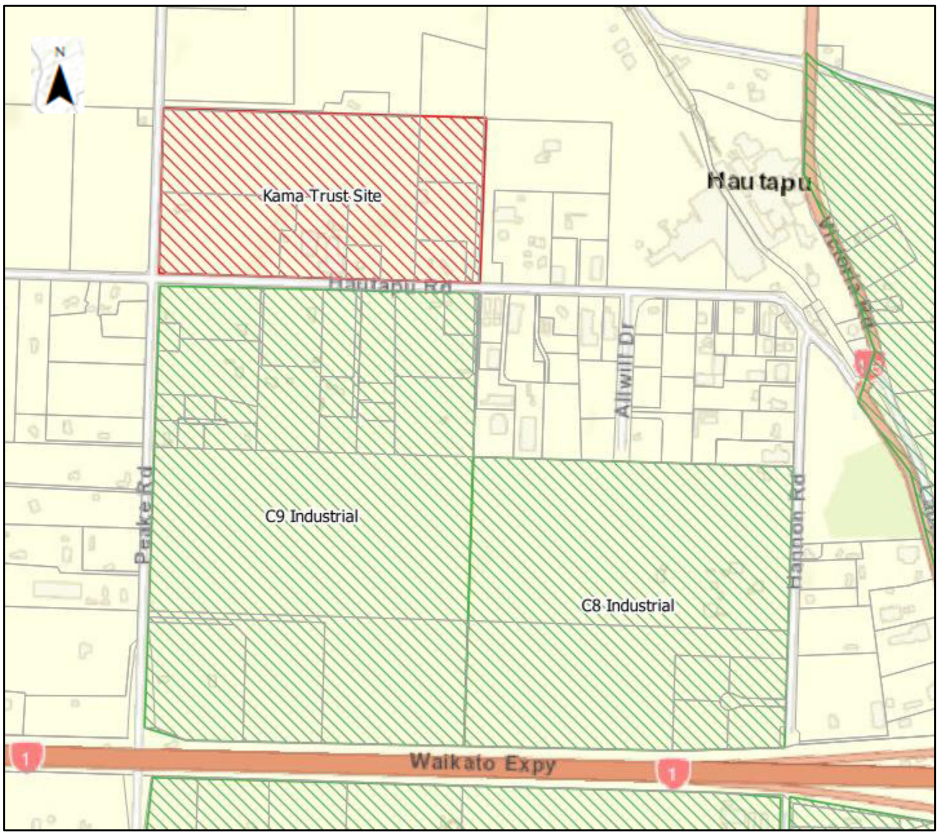

Figure 1-1 : Location of Proposed Kama Industrial Site

WSP
Hamilton
Level 2, 160 Ward Street (Bryce Street Entrance), Hamilton 3204
Private Bag 3057, Waikato Mail Centre, Hamilton 3240

+64 7 838 9344
wsp.com/nz

On behalf of Kama Trust, Josy Cooper approached WSP to undertake a portable water supply hydraulic assessment for the proposed land development to comply with Waipa District Council (WDC) development and subdivision requirements. WSP is to determine the suitable practice to supply the proposed site and confirm the supply strategy to support the land-use change of the proposed site.

There is no water supply network on the project site at the moment. As confirmed, WSP used the proposed water supply network for C8 and C9 growth cells (from Cambridge C8/C9 Mater Plan – Water Supply Report by Harrison Grierson) to supply the proposed site and assessed if the required Level of Service (LoS) and fire flow requirements can be achieved.

WDC will supply the proposed development's potable water as part of the North Cambridge water supply zone, if the proposed site becomes an industrial area.

2 Limitations

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This report is subject to the following limitations:

- WSP has provided the report based on the various assumptions contained in this report.
- Where WSP has obtained information from a government register or database, WSP have assumed that the information is accurate. Where an assumption has been made, subject of that assumption. WSP are not aware of any reason why any of the assumptions are incorrect.
- A change in circumstances, facts, information after the report has been provided may affect the adequacy or accuracy of the report. WSP is not responsible for the adequacy or accuracy of the report as a result of a change.

3 Key Assumptions

The following assumptions have been made in the hydraulic assessment:

- Utilise the 2021 Waipa base model (last updated in March 2022) with full C8 and C9 growth cell demands to carry out the hydraulic analysis
- Incorporate water reticulation for C8/9 as proposed for servicing C8/9
- Review the 2050 Waipa growth model to confirm the strategic infrastructure upgrades to service North Cambridge and Hautapu
- Use the supplied Cambridge C8/C9 Master Plan – Water Supply Report as the basis of the hydraulic assessment (provided by WDC on 14 April 2022), i.e. C8/9 water supply requires the installation of booster pumps from the present Hautapu reservoir to service full C8/9 development at required LoS

4 Acceptance Criteria

WSP have used the following as the acceptance criteria:

- RITS: Section 6 – Water Supply

Based on these specifications, the following hydraulic analysis is provided to confirm the current network is sufficient to provide the required LoS to all connections based on the following:

a) **Industrial Supply:** 7.5m^3/ha/day, or 0.087L/s/ha based on 30 people/ha, with the peak hour on the peak day of 0.435 L/s/ha (factor of 5.0) as per Harrison Grierson's Cambridge C8/C9 Master Plan – Water Supply Report.

No leakage was allowed in the Harrison Grierson's assessment. Therefore, leakage was assumed as 280 L/person/day for Cambridge (equivalent to 0.097 L/s/ha for the proposed site), which is consistent with the pervious master planning work.

Table 4-1 below summarises the demand for the proposed Kama Trust development

Table 4-1: Calculated Demands for Kama Trust Site

Area (ha)	Peak Hour Demand (L/s/ha):	Leakage Demand (L/s/ha):	Total Demand (L/s):
13	0.435	0.097	6.916

b) **Required Minimum Working Pressure:** LoS of 300 kPa (30 m) pressure at every connection point.

c) **Fire Flow:** New Zealand Fire Service Code of Practice; SNZ PAS 4509:2008 and subsequent amendments, to the satisfaction of the New Zealand Fire Service. Commercial requirement FW3 – 50 L/s.

Table 4-2: Fire Flow Requirements

Code:	Description:	Requirements:	
		Minimum Fire Flow (L/s):	Minimum Residual Pressure at Required Fire Flow (m):
FW2	Residential	25	10.0
FW3	Industrial	50	10.0

5 Methodology

- Add the C8 and C9 growth cell's water supply network in the 2022 base model. The twin DN200 mains are used to service the C8 and C9 growth cells and the proposed site from a modified booster pump at the existing Hautapu Reservoir site.
- The proposed site is modelled as a demand node and the full demand of C8 and C9 growth cells are allocated in the model.
- Run the model and confirm if the proposed site can achieve the required water pressure LoS.
- Carry out a fire flow analysis to check if the proposed site meets the fire flow and residual pressure criteria set out in New Zealand Fire Flow standards (SNZ PAS 4509:2008).

- Based on the simulation result, provide recommendations on network upgrades to meet the WDC's Level of Service and fire flow requirements.

6 Results

6.1 Model Network

The Waipa water supply network models are modelled as a 2D model in Inforworks WS Pro software. Table 6-1 below summarises the proposed pipe dimensions specified in RITS and AS/NZS 4130.

Table 6-1: Pipe Parameters

Pipe Product	Nominal Size (mm)	Mean Internal Diameter (mm)
PVC-0, PN12.5	150	166.8
PVC-0, PN12.5	200	218.4

Figure 6-1 below illustrates the modelled network. The proposed site was presented as a demand node with the development demand and was supplied via the C8 and C9 network from the existing network (twin DN200 on Victoria Street). The full demands of C8 and C9 growth cells were allocated to the C8 and C9 network.

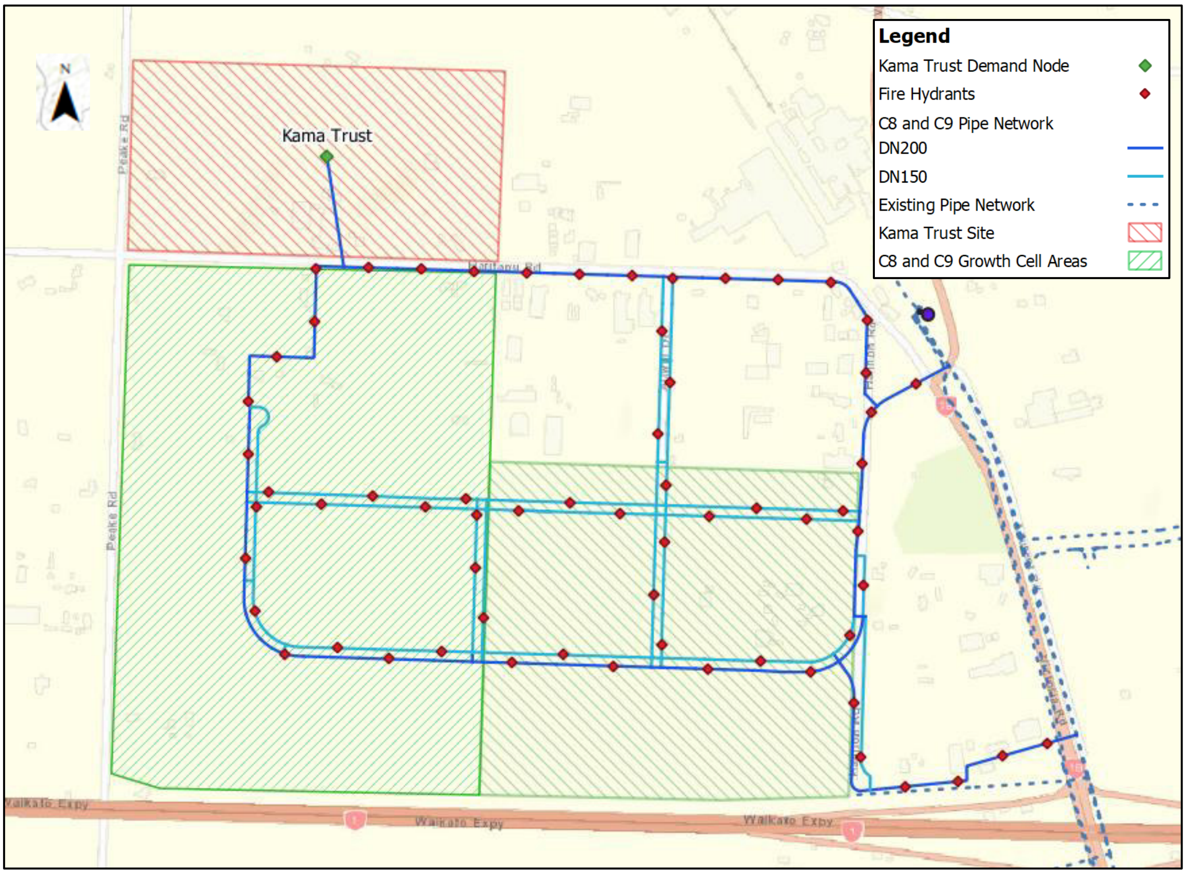


Figure 6-1: Modelled Water Supply Network

6.2 Model Results

The following sections detail the simulation results.

6.2.1 Working Pressure

The hydraulic model used the peak day demand to simulate the worst-case scenario within the water supply network. The Kama Trust site can be serviced with the proposed Hautapu booster pump and pipelines to service C8/C9 growth cells.

6.2.2 Fire Flow Testing

The fire flow capacity of the Kama Trust site was assessed by adding the required FW3 fire flow demand (50 L/s) to the Kama Trust demand node and checking if the minimum residual pressure (10 m) can be achieved during the fire event.

The required fire flow is to be provided by the new Hautapu booster pump and to service C8/C9 growth cells.

7 Conclusions and Recommendations

WSP has carried out a high-level water supply modelling assessment to verify if the Kama Trust site can have sufficient water supply if it becomes an industrial area from the current rural area. Waipa District Council provided the Cambridge C8/C9 Mater Plan – Water Supply Report by Harrison Grierson and WSP used this report as the basis of this assessment.

The assessment was conducted in the Waipa 2022 base model and 2050 growth model with the full C8 and C9 growth cell demands in place. The following WDC Level of Service and the New Zealand Fire Flow Standards (SNZ PAS 4509:2008) criteria are used in this assessment:

- Required Minimum Working Pressure: LoS of 300 kPa (30 m) pressure at every connection point.
- Fire Flow: New Zealand Fire Service Code of Practice; SNZ PAS 4509:2008 and subsequent amendments, to the satisfaction of the New Zealand Fire Service. Commercial requirement FW3 – 50 L/s. The minimum residual pressure at the required fire flow is 10 m.

The proposed Hautapu booster pumps and pipelines are planned to provide additional supply in the North Cambridge and Hautapu network and service the C8/C9 growth cells as part of the wider development plan. The Kama Trust site can be incorporated as part of the future development and the timing of these upgrades is to be confirmed by Waipa District Council as it is subject to the staging of the other developments in this area and implementation of existing infrastructure upgrades. Kama Trust Limited will need to work with Waipa District Council to confirm the staging of holistic capacity and supply upgrades to the Hautapu area and at a detail level the timing of development in C8/9 such that the internal reticulation in those growth cells is available .

APPENDIX B STORMWATER MANAGEMENT GENERAL LAYOUT PLANS

374 PEAKE ROAD, CAMBRIDGE
STORMWATER CONCEPT DESIGN
DEAN HAWTHORNE

DRAWING REGISTER

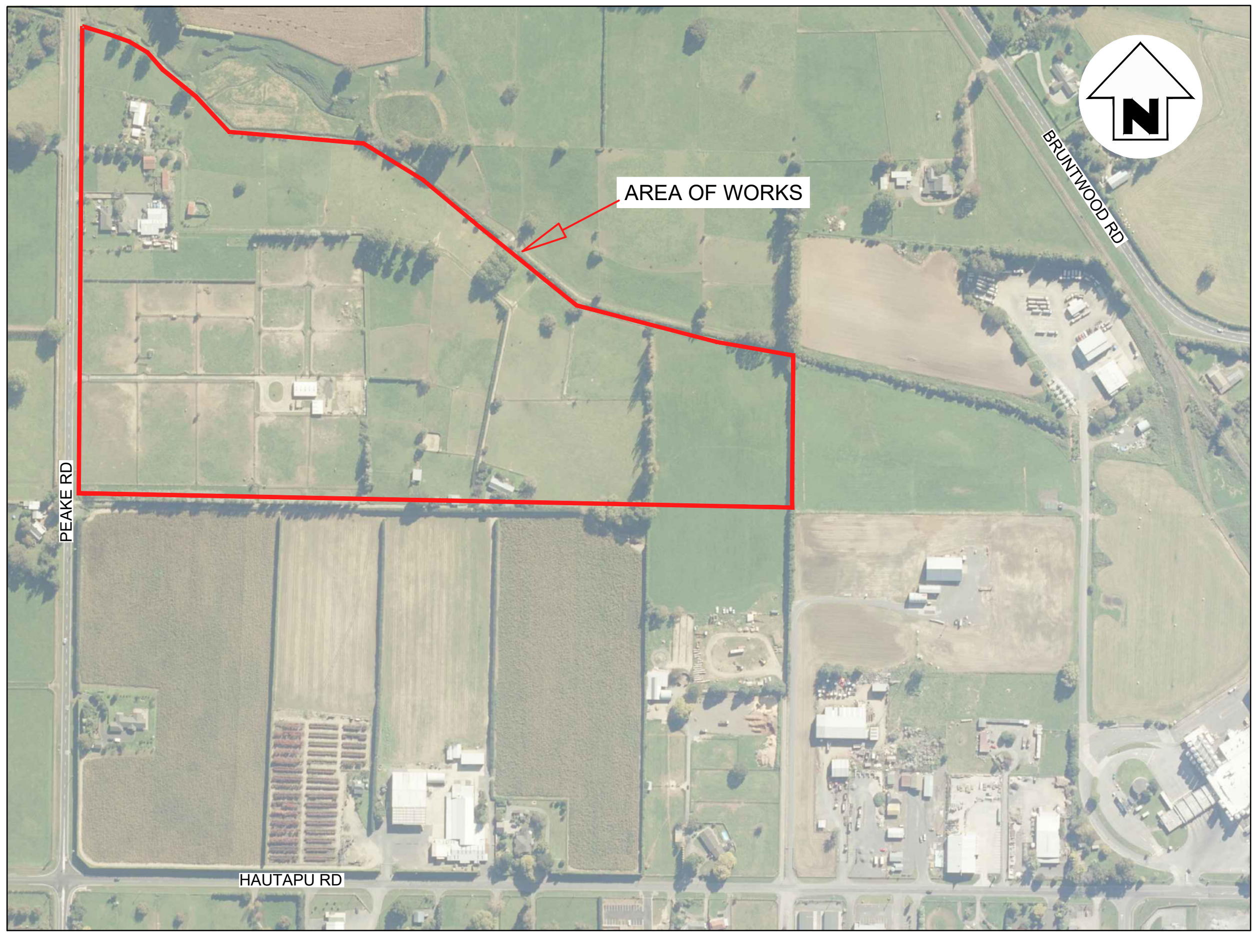
DRAWING	SHEET	TITLE	DATE OF ISSUE / REVISION				
			A				
221096.01	C00	COVER SHEET	01/02/23				
221096.01	C01	ONSITE STORMWATER MANAGEMENT - OPTION 1	01/02/23				
221096.01	C02	ONSITE STORMWATER MANAGEMENT - OPTION 2	09/03/23				
221096.01	C03	INTEGRATED SOLUTION WITH KAMA TRUST	01/02/23				

KEY

WD	WORKING DRAFT	C	TENDER
A	APPROVAL	0	CONSTRUCTION
B	CONSENTS	AS	AS BUILT

TRANSMITTAL

TO	ATTENTION	DATE OF ISSUE / REVISION				
		A				
OWNER / DEVELOPER	DEAN HAWTHORNE	01/02/23				
ARCHITECT / DESIGNER						
QUANTITY SURVEYOR						
BUILDER / CONTRACTOR						
TERRITORIAL AUTHORITY						



PROJECT LOCATION PLAN
SCALE N.T.S

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File Name: C:\12d\Sdata\BTW12D\221096 - Dean Hawthorne_5448\07 Drawings\221096-01 Hawthorne.dwg - C00 Plot Date: 09/03/2023 Plot Time: 09:50

GENERAL NOTES

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3. EXISTING TOP OF BANK AND STREAM LINE SHOWN ON THE PLAN ARE INDICATIVE ONLY.
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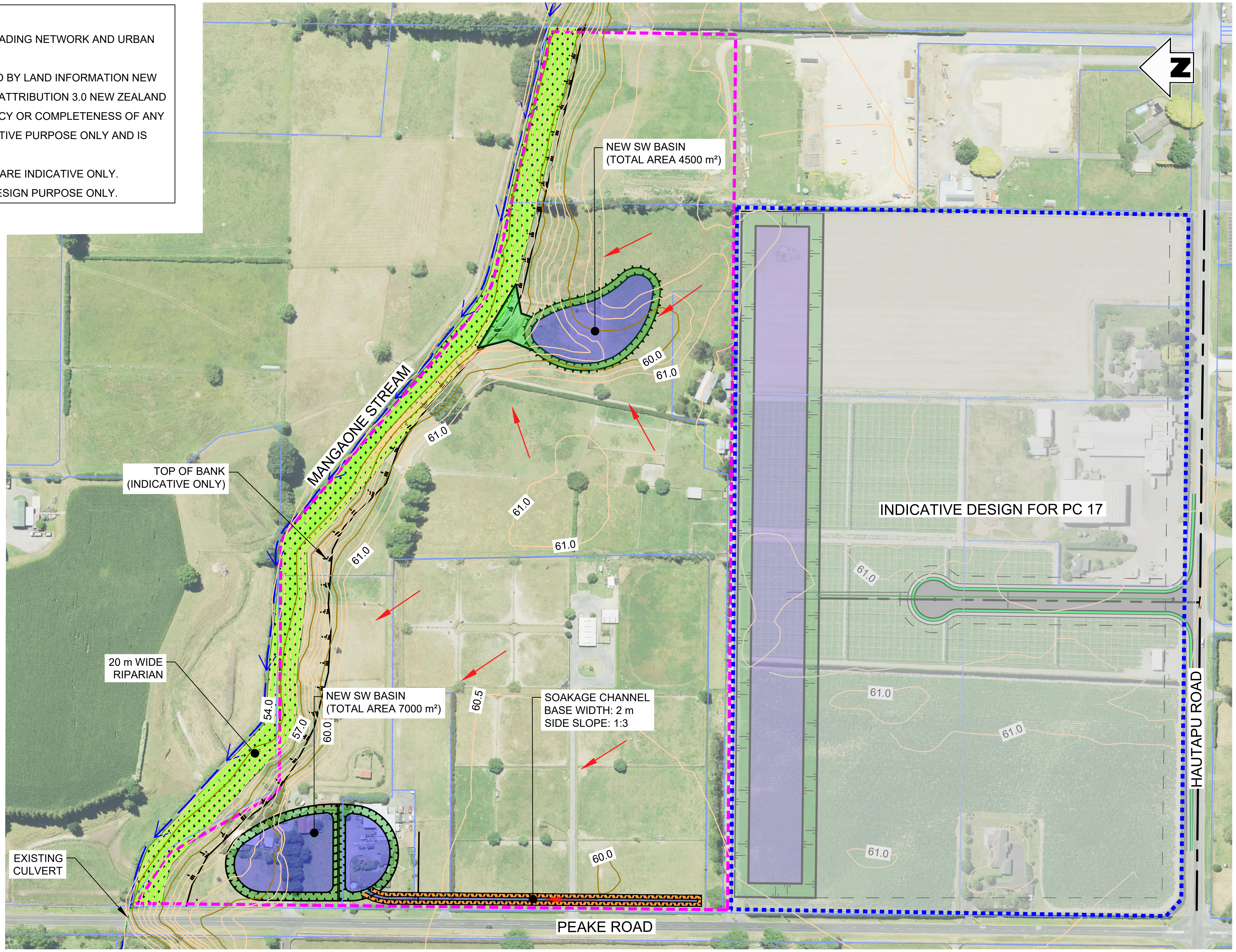
LEGEND

- BOUNDARIES
- TOP OF BANK
- SITE BOUNDARY
- SITE BOUNDARY (KAMA TRUST)
- STREAM
- EXISTING CULVERT
- FLOW DIRECTION
- 20m WIDE RIPARIAN BUFFER



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SCALE 1 : 3000

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PLAN
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NO	DATE	BY	CHKD.	APPR.	OPER.	DESCRIPTION
A	02/23	KA	SH	SH	.	ISSUED FOR APPROVAL

REVISIONS

NUMBER	TITLE

REFERENCE DRAWINGS

GENERAL NOTES
1. Coordinates in terms of : NA
2. Elevations in terms of : NA
3. Contour interval is : 0.5m

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PROJECT No.	221096	
A3 SCALE	AS SHOWN	
SURVEYED	-	-
DRAWN	K.AUNG	27.01.2023
CHECKED	S.HUSBAND	27.01.2023

TITLE
DEAN HAWTHORN
STORMWATER CONCEPT DESIGN
ONSITE STORMWATER MANAGEMENT - OPTION 1

ORIGINAL SIZE	DRAWING No	SHEET	REVISION
A3	221096-01	C01	A

File Name: C:\12d\S\data\BTW12D\221096 - Dean Hawthorne_5448\07 Drawings\221096-01 Hawthorne.dwg - C01 Plot Date: 09/03/2023 Plot Time: 09:50

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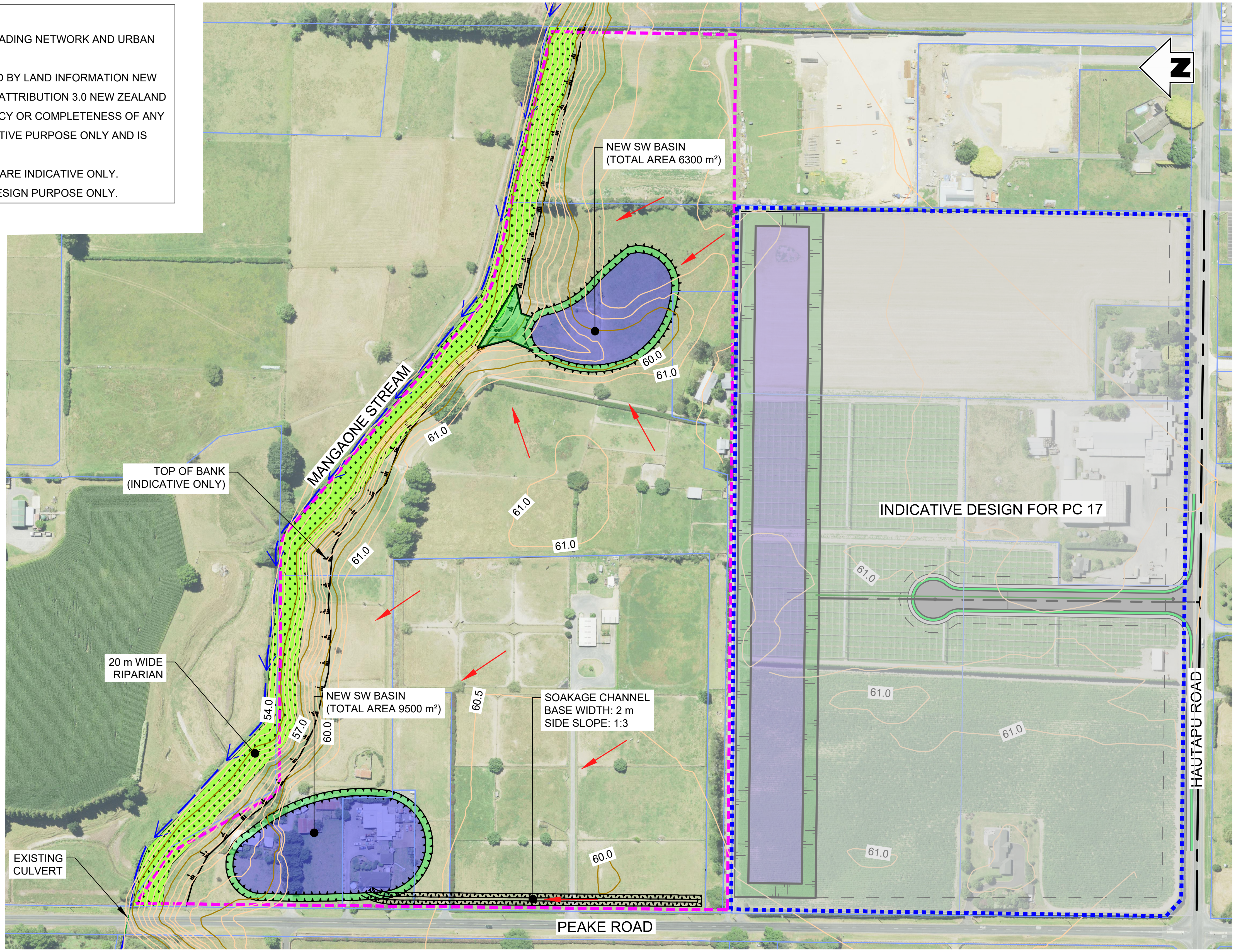

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REVISIONS

NUMBER	TITLE

REFERENCE DRAWINGS

GENERAL NOTES
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2. Elevations in terms of : NA
3. Contour interval is : 0.5m

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PROJECT No.	221096	
A3 SCALE	AS SHOWN	
SURVEYED	-	-
DRAWN	E.MURRAY	09.03.2023
CHECKED	S.HUSBAND	09.03.2023

TITLE
DEAN HAWTHORN
STORMWATER CONCEPT DESIGN
ONSITE STORMWATER MANAGEMENT - OPTION 2

ORIGINAL SIZE	DRAWING No	SHEET	REVISION
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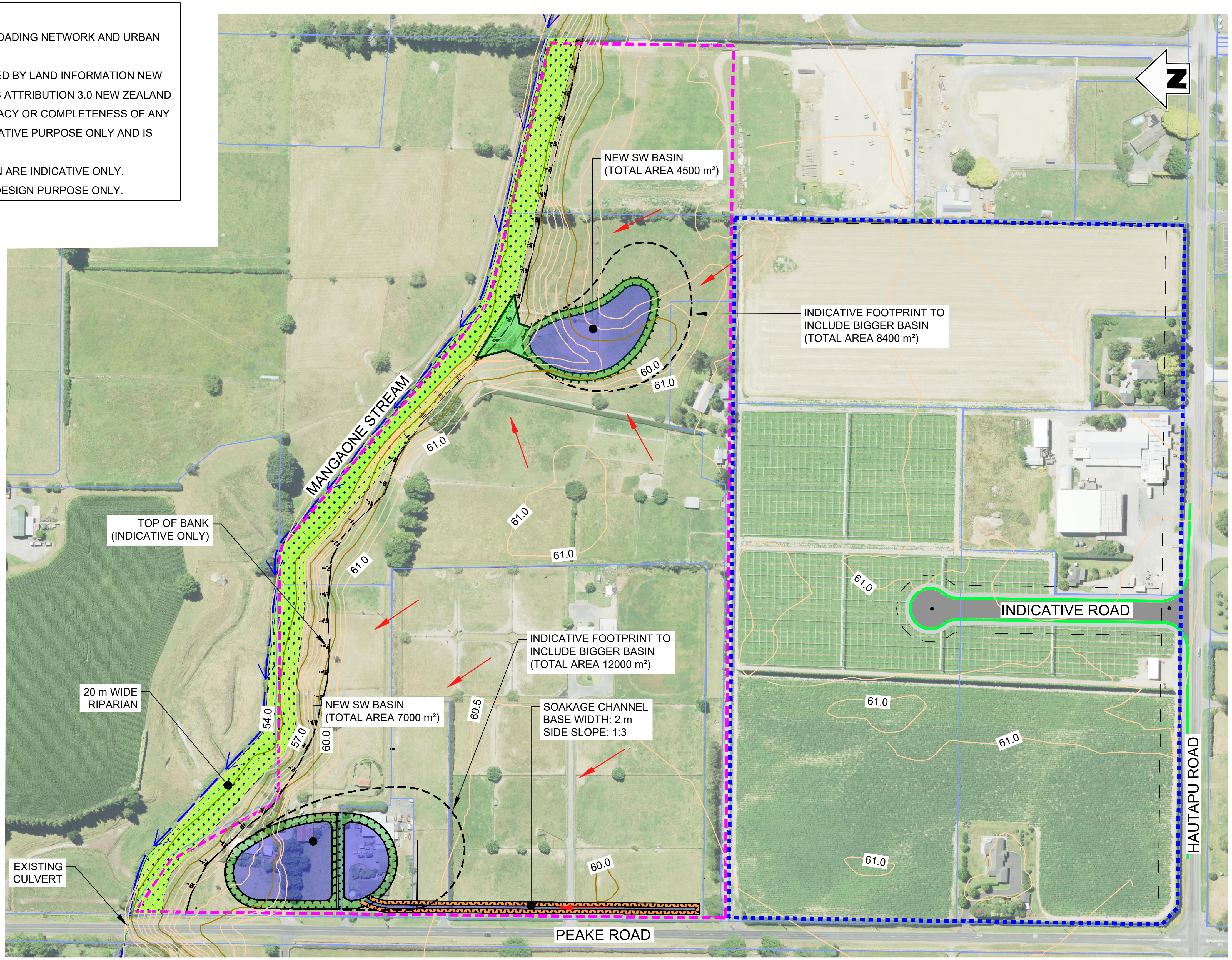

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PLAN
SCALE 1:3000

ISSUED FOR APPROVAL

BTW COMPANY

SURVEYING
ENGINEERING
PLANNING
ENVIRONMENT

NO	DATE	BY	CHKD.	APPR.	OPER.	DESCRIPTION
A	02/23	KA	SH	SH	.	ISSUED FOR APPROVAL

REVISIONS

NUMBER	TITLE

REFERENCE DRAWINGS

GENERAL NOTES
1. Coordinates in terms of : NA
2. Elevations in terms of : NA
3. Contour interval is : 0.5m

LOCATION	374 PEAKE RD, CAMBRIDGE	
PROJECT No.	221096	
A3 SCALE	AS SHOWN	
SURVEYED	-	-
DRAWN	K.AUNG	27.01.2023
CHECKED	S.HUSBAND	27.01.2023

TITLE
DEAN HAWTHORN
STORMWATER CONCEPT DESIGN
INTEGRATED SOLUTION WITH KAMA TRUST

ORIGINAL SIZE	DRAWING No	SHEET	REVISION
A3	221096-01	C03	A

File Name: C:\12dS\data\BTW12D\221096 - Dean Hawthorne_5448\07 Drawings\221096-01 Hawthorne.dwg - C03 Plot Date: 09/03/2023 Plot Time: 09:56

APPENDIX C STORMWATER MODELLING ASSESSMENT – OPTION 1

A stormwater modelling assessment was undertaken with the preparation of this report to develop a pre and post development hydrograph during the 50%, 10% and 1% AEP Design storm (Historical and RCP 6.0 rainfall) for the development site for input into the catchment wide hydrological and hydraulic modelling assessment using the stormwater system developed within this report.

This assessment defined the pre and post development hydrographs shown and summarised below.

The post development scenario utilised a stormwater basin with an impermeable liner with a soakage forebay with a piped outlet to the Mangaone Stream discharging at pre-development flow rates to collect stormwater and maintain the peak discharge rate of the predevelopment scenario for the stream.

Table 7.1: 1% AEP Post Development Hydraulic Modelling Results (24 hour design storm)

Catchment	Pre-Development Peak discharge (m^3/s) historic rainfall	Post Development Peak Discharge (m^3/s) RCP 6.0 2081-2100 rainfall
Project Site	1.54	1.54

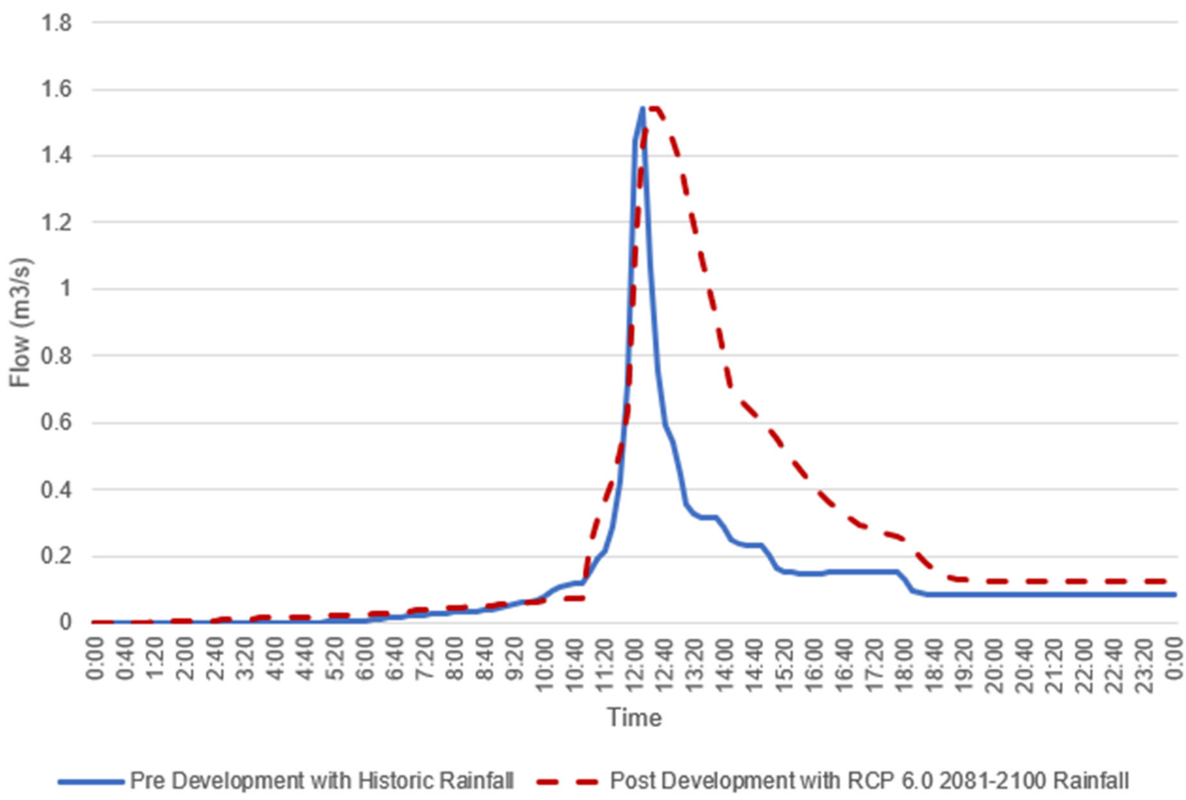

Figure C 1: Pre v Post Development Hydrograph for 1% AEP 24 hour Design Storm

C.1 Methodology

Peak run-off flows were calculated using the SCS unit hydrograph method.

The hydrological assessment was completed using the HIRDS historical rainfall data and repeated using the HIRD predicted future rainfall with RCP6.0 climate change allowance for the period 2081 – 2100.

A hydrological model was developed using the SCS Method in HEC HMS and the normalised 24 hour design storm temporal rainfall pattern specified within the WRC TR2020/06.

On-site soakage systems and stormwater treatment facilities were included in the post development model to simulate the storage/attenuation provided in these basins.

Stormwater modelling was performed utilising a common basin with the results being split in the planning process. This allowed estimations of basin sizing at early stages of development.

C.2 Catchment Analysis

Catchment analysis was undertaken to calculate design parameters for use in hydrological modelling and sizing of stormwater treatment facilities.

A summary of the analysis undertaken is provided below.

- Catchment area based on the area of the project site
- Impervious areas calculated via first principals based on the existing and proposed land use, taking the impervious percentage in an industrial area as at 80%.
- Catchment slope assessed using the end area method
- Time of concentration calculated utilising several methodologies with selection of a sensible median value
- Lag calculated for HEC-HMS input
- Selection of appropriate curve numbers, calculation of weighted curve number value, storage and initial abstraction in accordance with TR55 methodology. The Waikato Regional Council Technical Report 2020/06 Waikato stormwater run-off modelling guideline (WRC TR2020/06) Table 5.2 Run-off curve numbers for most urban and rural lands from the Waikato Region has been used for the selection of curve number as the soil categories are more appropriate than that included in the ARC TP108.
- The normalised temporal rainfall pattern and nested design storm has been adopted from the WRC TR 2018/02.
- Rainfall depths and intensities were selected from the appropriate V4 NIWA HIRDS rainfall data. Historical rainfall data was used in the selection of the pre-development rainfall intensity. Representative Concentration Pathway 6.0 2081-2100 (RCP 6.0) was used for the selection of post-development rainfall intensity.

Key outputs from the catchment analysis are included in Table C 1.

Table C 1: Catchment Analysis Summary

Catchment Description	Catchment Area (ha)	Curve Number (Weighted Average)	Initial Abstraction (mm)	Lag Time (mins) *
Pre-Development	17	59.5	8.6	10.3

Catchment Description	Catchment Area (ha)	Curve Number (Weighted Average)	Initial Abstraction (mm)	Lag Time (mins) *
Post-Development	17	90.2	1.4	7.0

*Lag time calculated as 2/3 of the time of concentration

C.3 Hydrological Modelling Results (Pre-Development)

Hydrological modelling was undertaken to understand the pre-development run-off from the existing site and its surrounding catchment.

C.3.1 HEC-HMS Model

The pre-development run-off was calculated using HEC-HMS software using the assumptions provided in Appendix C.2.

7.1.1 Results

The run-off volumes and peak flows calculated during the analysis in the 50%, 10% 1% AEP design storm events are summarised below in Table C 2.

Table C 2: Pre-Development Hydrological Modelling Results during 24 hour design storm (Historical, 2018)

Catchment Description	50% AEP Design Storm		10% AEP Design Storm		1% AEP Design Storm	
	Run-off Volume (m^3)	Peak Flow (m^3/s)	Run-off Volume (m^3)	Peak Flow (m^3/s)	Run-off Volume (m^3)	Peak Flow (m^3/s)
Pre-Development	2376	0.307	5340	0.712	11309	1.540

C.4 Hydraulic Modelling Results (Post-Development)

Hydraulic modelling was undertaken to calculate the run-off volumes from the developed site and the resulting post development peak flow rates following storage and attenuation within the stormwater basin.

C.4.1 HEC-HMS Model

A HEC-HMS model was constructed to model the post development run-off and the attenuation and outflow from the proposed basin.

Development run-off was modelled with a soakage forebay and a main basin with outflow to the Mangaone Stream at pre-development flow rates.

The basin dimensions have been optimised to provide the required soakage, attenuation and storage whilst minimising the underground footprint.

The model setup is provided in Figure C 2 and further information on the stormwater management system design is provided in Section 5.

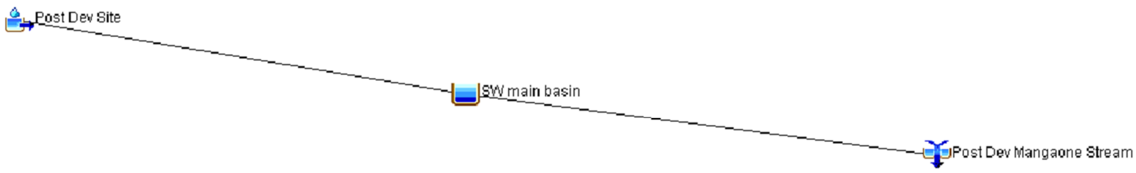

Figure C 2: HEC-HMS Model Setup

C.4.2 Results

The post development hydrographs at the basin outlet and the storage-elevation relationships for the proposed post development scenario are provided in Figure C 3, Figure C 4 and Figure C 5.

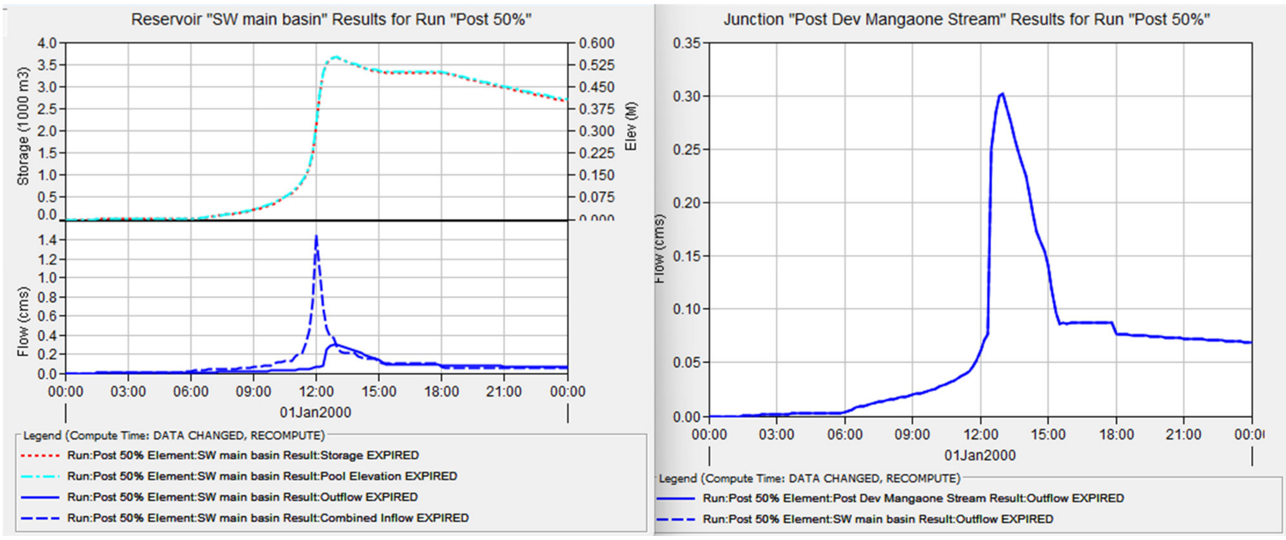

Figure C 3: Main Basin and Total Discharge to Stream, 50% AEP Design Storm Hydrograph

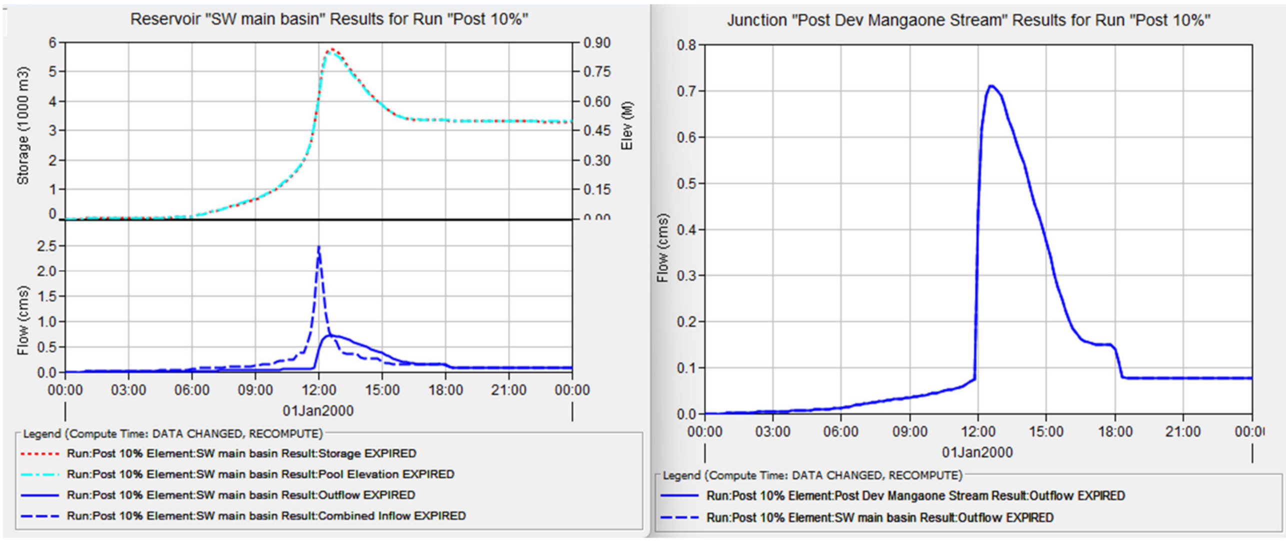

Figure C 4: Main Basin and Total Discharge to Stream, 10% AEP Design Storm Hydrograph

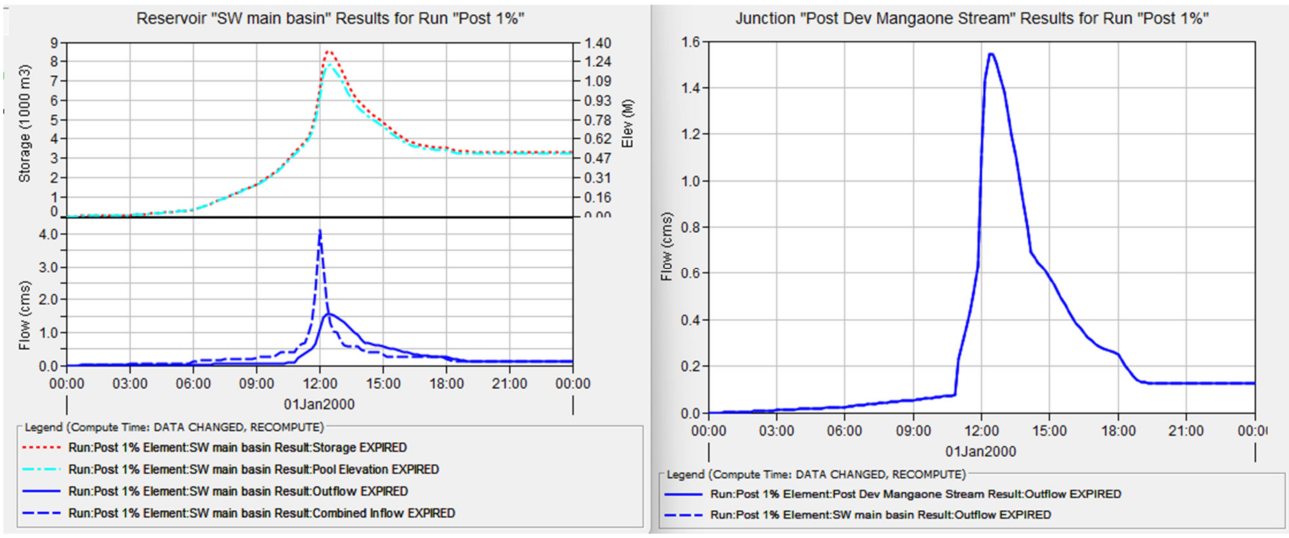

Figure C 5: Main Basin and Total Discharge to Stream, 1% AEP Design Storm Hydrograph

The run-off volumes and peak flows calculated during the analysis in the 50%, 10% and 1% AEP design storm events are summarised in Table C 3.

Table C 3: Post Development Hydraulic Modelling Results (24 hour design storm, RCP6.0, 2081-2100)

Catchment Description	Run-off Volume (m^3)	Peak Discharge (m^3/s)
Post Development Site 50% AEP	5350	0.302
Post Development Site 10% AEP	11072	0.710
Post Development Site 1% AEP	21403	1.544

The results show the peak discharge from the site during a 1 % design storm is restricted to the pre-development discharge rates despite significant increases in the post-development run-off volumes and peak inflows during all storm events. During the detailed design phase, the orifice will be designed as a staged outlet to match flows from a 50% AEP and 10% AEP storm to pre-development flows as well as the 1% AEP scenario.

It is concluded that the proposed mitigation measures discussed in Section 5 ensure that the development is having no impact on flows reaching the Mangaone Stream.

APPENDIX D STORMWATER MODELLING ASSESSMENT – OPTION 2

A stormwater modelling assessment was undertaken with the preparation of this report to develop a pre and post development hydrograph during the 50%, 10% and 1% AEP Design storm (Historical and RCP 6.0 rainfall) for the development site for input into the catchment wide hydrological and hydraulic modelling assessment using the stormwater system developed within this report.

This assessment defined the pre and post development hydrographs shown and summarised below.

The post development scenario utilised a stormwater soakage basin with a spillway to the Mangaone Stream discharging in a 1% AEP design storm at 80% of pre-development flow rates to retain and soak runoff during 50% and 10% design storms while soaking and discharging only partially from a 1% AEP design storm.

Table 7.2: 1% AEP Post Development Hydraulic Modelling Results (24 hour design storm)

Catchment	Pre-Development Peak discharge (m^3/s) historic rainfall	Post Development Peak Discharge (m^3/s) RCP 6.0 2081-2100 rainfall
Project Site	1.54	1.23

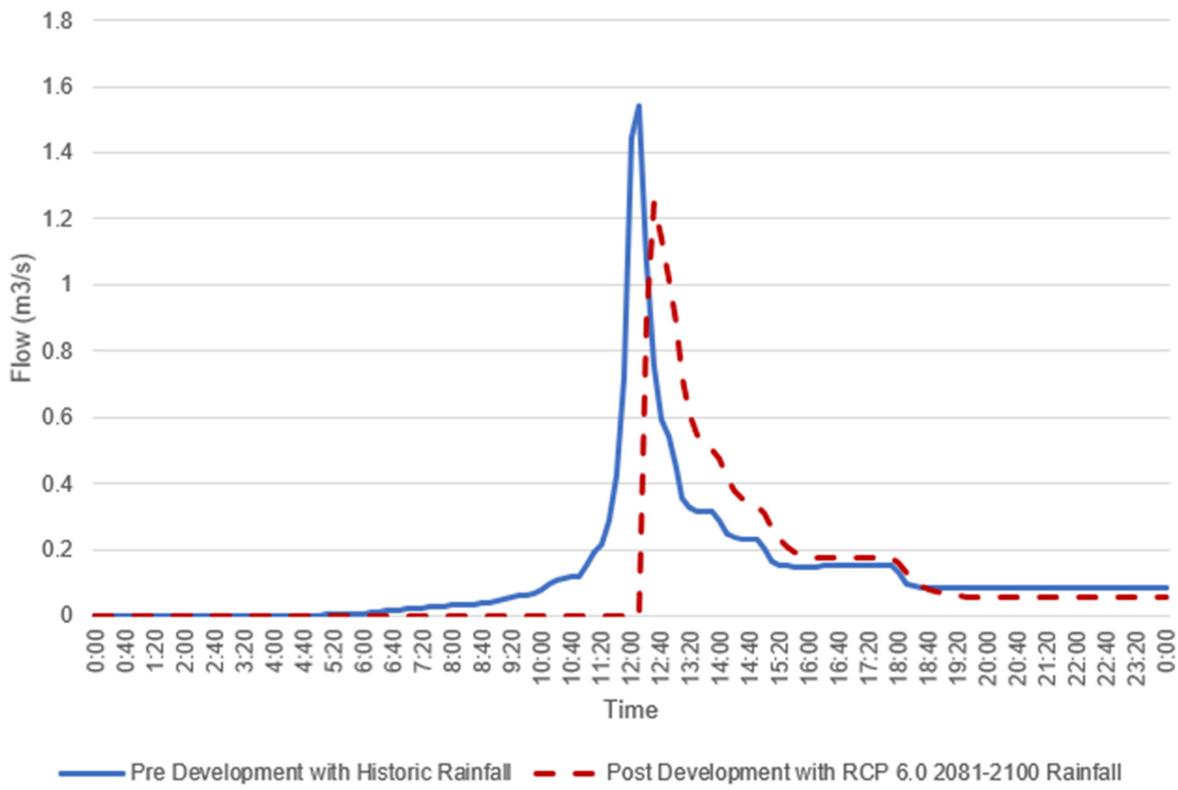

Figure D 6: Pre v Post Development Hydrograph for 1% AEP 24 hour Design Storm – Discharge to stream

D.1 Methodology

Peak run-off flows were calculated using the SCS unit hydrograph method.

The hydrological assessment was completed using the HIRDS historical rainfall data and repeated using the HIRD predicted future rainfall with RCP6.0 climate change allowance for the period 2081 – 2100.

A hydrological model was developed using the SCS Method in HEC HMS and the normalised 24 hour design storm temporal rainfall pattern specified within the WRC TR2020/06.

On-site soakage systems and stormwater treatment facilities were included in the post development model to simulate the storage/attenuation provided in these basins.

Stormwater modelling was performed utilising a common basin with the results being split in the planning process. This allowed estimations of basin sizing at early stages of development.

D.2 Catchment Analysis

Catchment analysis was undertaken to calculate design parameters for use in hydrological modelling and sizing of stormwater treatment facilities.

A summary of the analysis undertaken is provided below.

- Catchment area based on the area of the project site
- Impervious areas calculated via first principals based on the existing and proposed land use, taking the impervious percentage in an industrial area as at 80%.
- Catchment slope assessed using the end area method
- Time of concentration calculated utilising several methodologies with selection of a sensible median value
- Lag calculated for HEC-HMS input
- Selection of appropriate curve numbers, calculation of weighted curve number value, storage and initial abstraction in accordance with TR55 methodology. The Waikato Regional Council Technical Report 2020/06 Waikato stormwater run-off modelling guideline (WRC TR2020/06) Table 5.2 Run-off curve numbers for most urban and rural lands from the Waikato Region has been used for the selection of curve number as the soil categories are more appropriate than that included in the ARC TP108.
- The normalised temporal rainfall pattern and nested design storm has been adopted from the WRC TR 2018/02.
- Rainfall depths and intensities were selected from the appropriate V4 NIWA HIRDS rainfall data. Historical rainfall data was used in the selection of the pre-development rainfall intensity. Representative Concentration Pathway 6.0 2081-2100 (RCP 6.0) was used for the selection of post-development rainfall intensity.

Key outputs from the catchment analysis are included in Table C 1.

Table D 4: Catchment Analysis Summary

Catchment Description	Catchment Area (ha)	Curve Number (Weighted Average)	Initial Abstraction (mm)	Lag Time (mins) *
Pre-Development	17	59.5	8.6	10.3

Catchment Description	Catchment Area (ha)	Curve Number (Weighted Average)	Initial Abstraction (mm)	Lag Time (mins) *
Post-Development	17	90.2	1.4	7.0

*Lag time calculated as 2/3 of the time of concentration

D.3 Hydrological Modelling Results (Pre-Development)

Hydrological modelling was undertaken to understand the pre-development run-off from the existing site and its surrounding catchment.

D.3.1 HEC-HMS Model

The pre-development run-off was calculated using HEC-HMS software using the assumptions provided in Appendix C.2.

7.1.2 Results

The run-off volumes and peak flows calculated during the analysis in the 50%, 10% 1% AEP design storm events are summarised below in Table C 2.

Table D 5: Pre-Development Hydrological Modelling Results during 24 hour design storm (Historical, 2018)

Catchment Description	50% AEP Design Storm		10% AEP Design Storm		1% AEP Design Storm	
	Run-off Volume (m^3)	Peak Flow (m^3/s)	Run-off Volume (m^3)	Peak Flow (m^3/s)	Run-off Volume (m^3)	Peak Flow (m^3/s)
Pre-Development	0	0	0	0	9728	1.230

D.4 Hydraulic Modelling Results (Post-Development)

Hydraulic modelling was undertaken to calculate the run-off volumes from the developed site and the resulting post development peak flow rates following storage and attenuation within the stormwater basin.

D.4.1 HEC-HMS Model

A HEC-HMS model was constructed to model the post development run-off and the attenuation and outflow from the proposed basin.

Development run-off was modelled with a soakage base and spillway with outflow to the Mangaone Stream at 80%of pre-development flow rates during a 1% AEP design storm.

The basin dimensions have been optimised to provide the required soakage, attenuation and storage whilst minimising the underground footprint.

The model setup is provided in Figure D 7 and further information on the stormwater management system design is provided in Section 5.

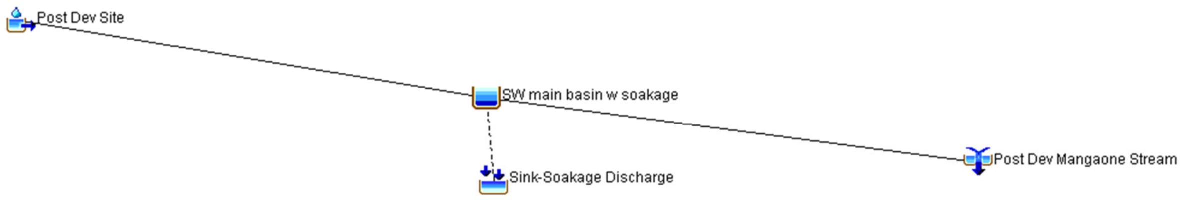

Figure D 7: HEC-HMS Model Setup

D.4.2 Results

The post development hydrographs at the basin outlet and the storage-elevation relationships for the proposed post development scenario are provided in Figure D 8, Figure D 9 and Figure D 10.

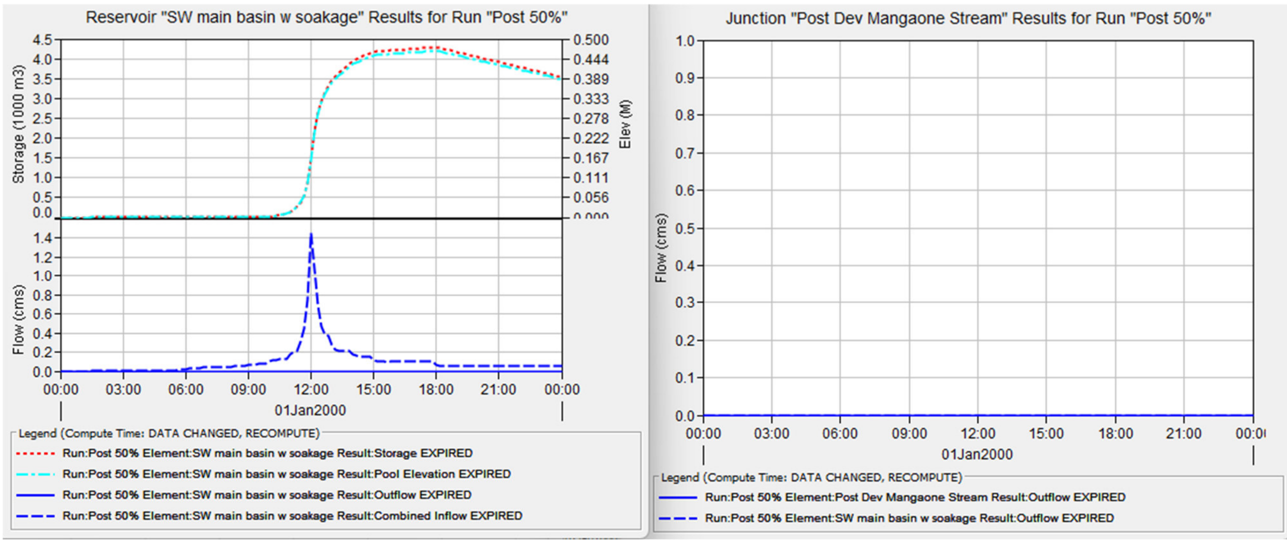

Figure D 8: Main Basin and Total Discharge to Stream, 50% AEP Design Storm Hydrograph

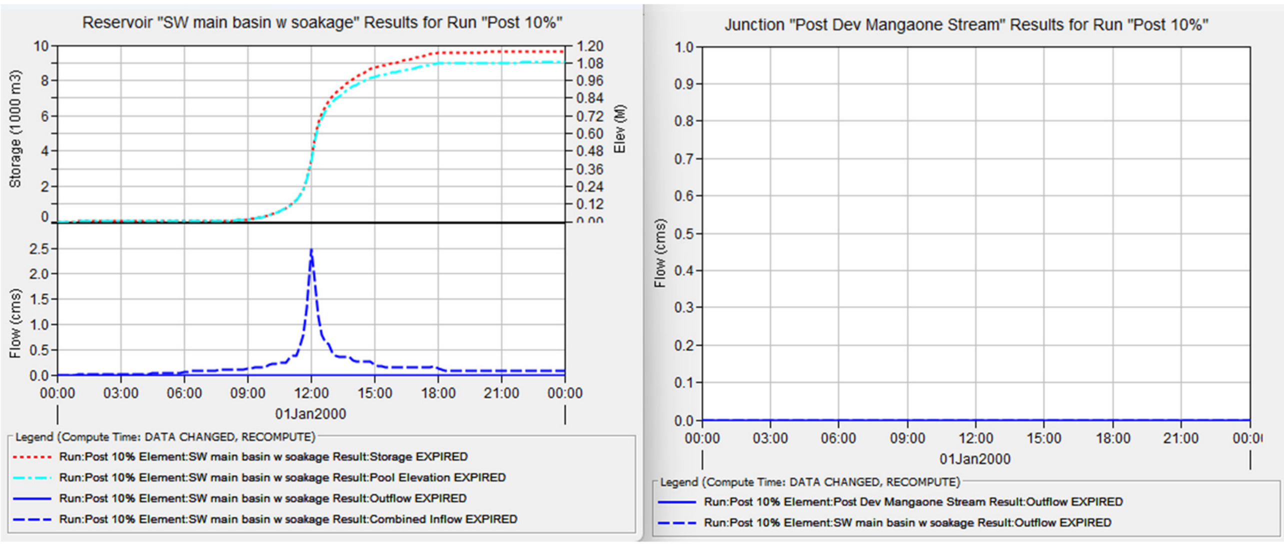

Figure D 9: Main Basin and Total Discharge to Stream, 10% AEP Design Storm Hydrograph

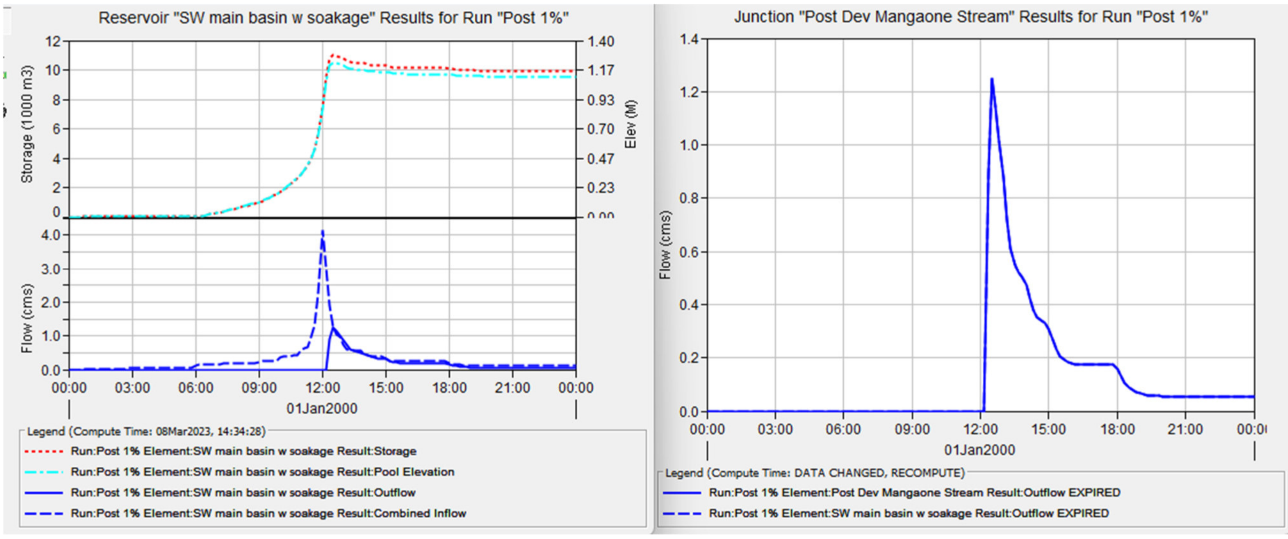

Figure D 10: Main Basin and Total Discharge to Stream, 1% AEP Design Storm Hydrograph

The run-off volumes and peak flows calculated during the analysis in the 50%, 10% and 1% AEP design storm events are summarised in Table C 3.

Table D 6: Post Development Hydraulic Modelling Results (24 hour design storm, RCP6.0, 2081-2100)

Catchment Description	Run-off Volume (m^3)	Peak Discharge (m^3/s)
Post Development Site 50% AEP	0	0
Post Development Site 10% AEP	0	0
Post Development Site 1% AEP	9728	1.230

The results show the peak discharge from the site during a 1 % design storm is restricted to 80% of the pre-development discharge rate despite significant increases in the post-development run-off volumes and peak inflows during all storm events.

It is concluded that the proposed mitigation measures discussed in Section 5 ensure that the development is having a positive impact on flows reaching the Mangaone Stream.

APPENDIX E WETLAND DESIGN PARAMETERS

Each wetland will consist of two modules, the forebay and the main basin. Stormwater will enter the basin, where it will be held for water quality treatment and discharged to the Mangaone Stream to meet the pre-development flow rate.

The main components of the wetland include:

- Forebay with a volume of 15% of the water quality volume (WQV) based on RITS Table 4-22.
- Holding a permanent pool, a portion of the water quality volume as defined by RITS Table4-20.
- Holding the extended detention volume (EDV) to be released over 24 hours.
- The remainder of the run-off volume held by the wetland will be released via staged orifice to the Mangaone Stream at pre-development flow rates of the 50%, 10% and 1% AEP design storms.

Wetland design parameters and can be seen in Table E 7, location of the wetlands and drainage channel is shown in Appendix B and modelling calculations shown in Appendix C.

Table E 7: Stormwater Basin Design Parameters and Assumptions – Subject Site

Parameter	Design Wetland 1	Design Wetland 2
Catchment area	10.7ha	6.3
Post development Percentage Impervious	80%	80%
Total Wetland Surface Area	7000m²	4500m²
Boundary Standoff for Planting and Access	5m	5m
Total Volume	7800m³	4800m³
EDV Depth	0.5m	0.5m
Dead storage Depth	0.3m	0.3m
Total Wetland Depth	1.5m	1.5m
Wetland Batter Slope Gradient	1:3	1:3

The project site constructed wetlands can be increased in size to accommodate the Kama Trust area run-off in a series of shared constructed wetlands.

Wetland design parameters and can be seen in Table E 7, location of the wetlands and drainage channel is shown in Appendix B and modelling calculations shown in Appendix C.

Table E 8: Stormwater Basin Design Parameters and Assumptions – Integrated with Kama Trust

Parameter	Wetland 1 Design	Wetland 2 Design
Attenuation & flood control	5800m³	4000m³
Stream protection (EDV)	3800m³	2600m³
Water Quality Treatment (WQV)	2100m³	1400m³
Total volume	7000m³	4700m³
Site area	19.7ha	13.1ha
Post development Percentage Impervious	80%	80%
Total Wetland Surface Area	12000m²	8000m²
Boundary Standoff for Planting and Access	5m	5m
EDV Depth	0.5m	0.5m
Dead storage Depth	0.3m	0.3m
Total Wetland Depth	1.5m	1.5m
Wetland Wall Gradient	1:3	1:3

APPENDIX F SOAKAGE BASIN DESIGN PARAMETERS

Each soakage pond will contain the main basin. Stormwater will enter the basin, where it will be held for water quality treatment and discharged to the Mangaone Stream to meet the pre-development flow rate.

The main components of the soakage basin include:

- Capturing runoff from a 50% AEP design storm for discharge through soakage
- Capturing runoff from a 10% AEP design storm for discharge through soakage
- Capturing runoff from a 1% AEP design storm for combined discharge through soakage and run off via a spillway at 80% of pre development flow rates

Wetland design parameters and can be seen in Table F 9, location of the soakage basins and drainage channel is shown in Appendix B and modelling calculations shown in Appendix D.

Table F 9: Stormwater Basin Design Parameters and Assumptions – Subject Site

Parameter	Design Soakage Basin 1	Design Soakage Basin 2
Catchment area	10.7ha	6.3
Post development Percentage Impervious	80%	80%
Total Surface Area	5800m²	3700m²
Boundary Standoff for Planting and Access	5m	5m
Total Volume	4000m³	2500m³
Total Depth	1.2m	1.2m
Batter Slope Gradient	1:3	1:3