Before an Independent Hearings Panel
Appointed by Waimakariri District Council

under:	the Resource Management Act 1991
in the matter of:	Submissions and further submissions on the Proposed Waimakariri District Plan
and:	Hearing Stream 12: Rezoning requests (larger scale)
and:	**Carter Group Property Limited** *(Submitter 237)*
and:	**Rolleston Industrial Developments Limited** *(Submitter 160)*

Statement of evidence of Eoghan O'Neill (Stormwater and Wastewater) on behalf of Carter Group Limited and Rolleston Industrial Developments Limited

Dated: 5 March 2024

Reference: J M Appleyard (jo.appleyard@chapmantripp.com)
LMN Forrester (lucy.forrester@chapmantripp.com)

chapmantripp.com
T +64 3 353 4130
F +64 4 472 7111

PO Box 2510
Christchurch 8140
New Zealand

Auckland
Wellington
Christchurch

chapman tripp

STATEMENT OF EVIDENCE OF EOGHAN O'NEILL ON BEHALF OF CARTER GROUP LIMITED AND ROLLESTON INDUSTRIAL DEVELOPMENTS LIMITED

INTRODUCTION

1 My full name is Eoghan Michael O'Neill.

2 I am a Technical Director with Pattle Delamore Partners Ltd and have been employed in that capacity since October 2012. I am a Chartered Professional Engineer with more than 20 years' experience in the planning and design of wastewater, water supply and stormwater infrastructure.

3 I hold Bachelor of Engineering and Master of Engineering Science degrees awarded by University College Dublin. Much of my experience is related to the planning of infrastructure to facilitate development. I have prepared and presented evidence to Plan Change Hearings, Resource Consent Hearings and the Environment Court on numerous occasions. I have performed this role both as a Council employee and as a consultant on behalf of applicants.

4 I am familiar with the Submitters' request to rezone land bound by Mill Road, Whites Road, Bradleys Road (the *Site*).

5 I was involved in private plan change 31 (*PC31*) to rezone this land under the operative District Plan.

CODE OF CONDUCT

6 Although this is not an Environment Court hearing, I note that in preparing my evidence I have reviewed the Code of Conduct for Expert Witnesses contained in Part 9 of the Environment Court Practice Note 2023. I have complied with it in preparing my evidence. I confirm that the issues addressed in this statement of evidence are within my area of expertise, except where relying on the opinion or evidence of other witnesses. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed.

SCOPE OF EVIDENCE

7 My evidence will address:

7.1 Description of the management of stormwater within the proposed development; and

7.2 Assessment of options available for servicing the Site for stormwater and wastewater.

8 In preparing my evidence, I have reviewed:

8.1 The Outline Development Plan (*ODP*);

8.2 Statement of Evidence (Infrastructure) prepared by **Mr Tim Mcleod** of Inovo Ltd;

8.3 Statement of Evidence (Hydrology) prepared by **Mr Bas Veendrick** of Pattle Delamore Partners Ltd;

8.4 Statement of Evidence (Ecology) prepared by **Ms Laura Drummond** of Pattle Delamore Partners Ltd;

8.5 Statement of Evidence (Flooding) prepared by **Mr Ben Throssell** of Pattle Delamore Partners Ltd;

8.6 Further submissions relevant to my expertise relating to the rezoning of the Site; and

8.7 The relevant documents from PC31.

SUMMARY OF EVIDENCE

9 The management of stormwater quantity, including hydraulic continuity between the upstream and downstream catchments, can be managed by means of the following:

9.1 Formalised flow paths to be installed to connect the upstream and downstream catchments.

9.2 Attenuation and flood storage to be provided within the Site to manage runoff up to the 2% AEP (50-yr ARI) by the use of basins, compensatory storage, and rain tanks. Stormwater detention basins will be designed to be constructed along the fall of the Site with minimal excavation to ensure storage can be provided without intercepting highest groundwater at the Site. Low bunding shall be gradually formed along the fall of the contour to retain floodwaters.

10 Water quality treatment can be provided as follows:

10.1 Residential and retirement village/school runoff to be predominantly treated by means of filtration via high infiltration rate raingardens or swales and bioscapes which will be designed to treat 90% of rainfall runoff from the Site. Raingardens and bioscapes, being approximately 1m deep, will likely be constructed into seasonal groundwater. They will be fully lined so as to avoid any active drainage of groundwater that may be intercepted at their base.

10.2 Up to 2ha of stormwater wetlands or wet ponds can be constructed at the Site as a permitted activity under Rule 5.114 of the Canterbury Land and Water Regional Plan (*LWRP*). This provision allows greater flexibility for the location of potential treatment and storage facilities in wetter parts of the Site during detailed design. For the purposes of this proposal, all storage and treatment is provided without the use of wetlands or wet ponds.

10.3 Large lot residential stormwater runoff to be treated by means of swales, high-infiltration raingardens and bioscapes.

10.4 Stormwater runoff from business areas to be treated by means of rain gardens or proprietary filtration devices.

10.5 All stormwater treatment infrastructure will be designed to limit potential groundwater take to within permitted activity status under requirements of the LRWP.

11 Wastewater for the proposed development can be managed by way of a new wastewater pump station located within the Site pumping to Rangiora Wastewater Treatment Plan (*WWTP*) via a new rising main.

12 To facilitate the initial build out of lots, and mitigate any odour issues which would occur with a small number of lots connected to the new wastewater main, the new pump station could connect to the existing Mandeville/ Ōhoka wastewater pressure main to facilitate the development of an initial 250 lots before the new pressure main was constructed to the WWTP.

REZONING REQUEST SUMMARY

13 The majority of the Site is located at 535 Mill Road and is roughly trapezoidal in shape bounded for the most part by Whites, Mill and Bradleys roads, Ōhoka. The Site is typically gently sloping (1:180) to flat, sloping from west to east towards Whites Road. The current land use of the Site is a dairy farm with the farmhouse and farm buildings in a cluster towards the western corner and an additional cluster of farm buildings near the boundary of 531 Mill Road. Open paddocks predominate, but the Site comprises a variety of mature trees and shelterbelts. A relatively high water table extends over the Site and several waterways, including Ōhoka Stream and the Ōhoka South Branch, flow in an easterly direction across the Site.

14 The proposed residential development will comprise of up to 850 residential units, a potential primary school and a potential retirement village. If a school is not developed, approximately 42 additional residential units could be developed. The new commercial area (Local Centre Zone) will provide for approximately 2,700m^2 of commercial floor space as well as car parking.

EXISTING SITE STORMWATER CHARACTERISTICS

15 The proposed Site is zoned as rural and is approximately 156 ha in area. The existing land-uses on-site consist of large undeveloped

paddock areas. Existing impervious areas are limited to unsealed roads, buildings and associated sealed areas (< 1% of existing area). The general fall across the Site is northwest to southeast and elevation ranging between RL 29 m to RL 20 m. The average slope across the Site is approximately 0.5% (1V:200H).

16 The Site has limited stormwater infrastructure and runoff from the Site generally drains via land drains or as sheet flow from the Site. These existing land drains collect and drain stormwater and high groundwater away from the contributing catchment areas to the main waterways crossing the Site. As shown in **Attachment 1**, a tributary of Ōhoka Stream crosses the northern part of the Site, and several branches of South Ōhoka Stream cross the southern half of the Site. Two springs are mapped on the Site in the Canterbury Regional Council (*ECan*) online database. A groundwater seep is located on the Site closer to Whites Rd. These springs are discussed in detail in the evidence of **Ms Drummond**.

17 Potential flooding of the Site and the surrounding land is covered in detail in the evidence of **Mr Throssell**. The stormwater management proposals for the Site have been developed in close collaboration with the flood modelling and flood mitigation work to ensure that the development can progress without increasing the flood risk to properties upstream or downstream of the development. As noted in Paragraph 88 of **Mr Throssell's** evidence, *"modelling of the 200-year event shows the flood hazard is still low for areas south of Mill Road/downstream of Whites Road and moderate for areas north of Mill Road. I note the PDP model predicts generally limited effects greater than 10 mm for areas north of Mill Road and no increase greater than 20 mm for habitable dwellings elsewhere within the PDP model."* He therefore concludes that the effect of the development on flooding outside of the Site are less than minor.

18 In general, the groundwater flows to the southeast, towards the coast. Groundwater discharges into spring fed streams, including the Ōhoka Stream and the Cam River/Ruataniwha. The groundwater is typically shallow and subject to seasonal fluctuations. Groundwater at the Site is estimated, using the record from bore M35/0596, to be an average of 0.64 m below ground level (bgl) with the highest recorded groundwater level at 0.14 m bgl (June 2018). Seasonal fluctuations in this bore are relatively small, commonly being 0.5 – 0.8 m. As expected, groundwater levels are generally highest in winter/spring and lowest in summer/autumn. It is noted that bore M35/0596 is close to spring M35/7485 (mapped location is 20 m away), and so may be in an area of the Site that has particularly high groundwater levels.

19 It is noteworthy that extensive test pitting undertaken by Tetra Coffey Ltd at the Site in May 2021 encountered a range of groundwater depths, these are shown on **Attachment 2**. The shallowest groundwater level recorded during this testing was

1.15m bgl close to Spring M35/7485, the deepest groundwater was encountered at 1.85m bgl at the Mill Road end of the Site. The recorded water depth at monitoring bore M35/0596 at the time of these investigations was approximately 0.9m bgl. Detailed knowledge of maximum ground levels across the Site will be crucial to inform the placement and depth of stormwater detention ponds at the Site. The stormwater concept has conservatively assumed that stormwater detention basins will be constructed with minimal excavation (less than 0.2 m) to avoid interception of groundwater. Detailed groundwater monitoring at the Site will be undertaken prior to development to inform the detailed design of these basins and ensure no interception of groundwater occurs.

20 The downstream catchment has comparable properties to the pre-development Site. The downstream catchment is undeveloped rural land (paddocks) with land drains collecting runoff and intersecting shallow groundwater. The downstream catchment drains towards the Ōhoka Stream and eventually to the Kaiapoi River.

FLOW CONTINUITY

21 The continuity of pre-and post-development flows from upstream of the Site to downstream of the Site will be provided by way of the three main formalised flow path corridors through the proposed development. The management of these flow corridors will convey flow from upstream of the Site to downstream of the Site without increasing the flood risk outside of the Site are discussed in in the evidence of **Mr Throssell**.

PROPOSED STORMWATER MANAGEMENT

22 The pre- and post-development stormwater catchments for the proposed development were delineated using the following information:

22.1 Available LiDAR information.

22.2 Existing stormwater infrastructure as per the Waimakariri District Council (*WDC*) GIS.

22.3 Flow paths as determined by the WDC 200-year flood modelling results.

22.4 Proposed development plans.

23 The following design criteria was used as the basis assessment of the stormwater effects:

Table 1: Design Criteria for Stormwater Treatment Devices

Item	Design Criteria	References

Primary flows	• 5-year return event (20% AEP)	WDC CoP 2020
Secondary flows	• 50-year return event (2% AEP)	WDC CoP 2020
Attenuation requirement	• Post-development peak flows for all intensities to be less than pre-development flows	WDC CoP 2020
Rainfall	• HIRDS V4 RCP 8.5 (2081 – 2100)	WDC CoP 2020
Runoff coefficient	• As per Table 5.2 & Table 5.3 of WDC Engineering Code of Practice	WDC CoP 2020
Water Quality Flow	• 5 mm/hr	CCC Onsite Stormwater Mitigation Guide

24 Any modification of the main flow paths (i.e. Ōhoka Stream tributaries) across the Site will be designed to maintain hydraulic connectivity between the upstream and downstream catchments including baseflow. They also will collect and convey controlled outflows of treated stormwater, from the attenuation basins associated with the proposed development catchments, towards the downstream environment. All stormwater treatment infrastructure and stormwater attenuation basins will be located outside of the 50-year flood level of the main flow paths.

25 Primary stormwater runoff from the residential development areas (i.e. flows from up to a 20% AEP/5-yr ARI) within the Site will be collected along roads via swales. This flow will be conveyed to be discharged into either raingardens or bioscapes for treatment.

26 Rain gardens are a "closer to source" treatment system consisting of engineered gardens designed to harness the natural ability of vegetation and soils to treat stormwater. They are typically built in the berm to the side of the kerb and channel, or in this instance would be in the swale or the berm next to the swale. Treatment occurs through sedimentation, filtration, adsorption through the soil media and uptake by vegetation. A proprietary high infiltration rate engineered media (trade name Filterra®) can also be used within rain gardens to reduce the required footprint or increase the treatment flow over a certain raingarden infiltration area. Raingarden media will absorb and filter contaminants before stormwater flows into a slotted or perforated pipe underdrainage system located within a granular drainage layer at the base of the rain garden structure. The raingardens will be located within a

concrete structure or appropriately lined excavation to prevent the drainage of groundwater into the system. The underdrainage system connects to a piped stormwater network which conveys treated stormwater along with any first flush overflow to detention storage. A typical cross section though a rain garden is shown in Figure 1 below. Note the figure below allows for partial soakage to ground, whereas the systems within the Site will be fully sealed.

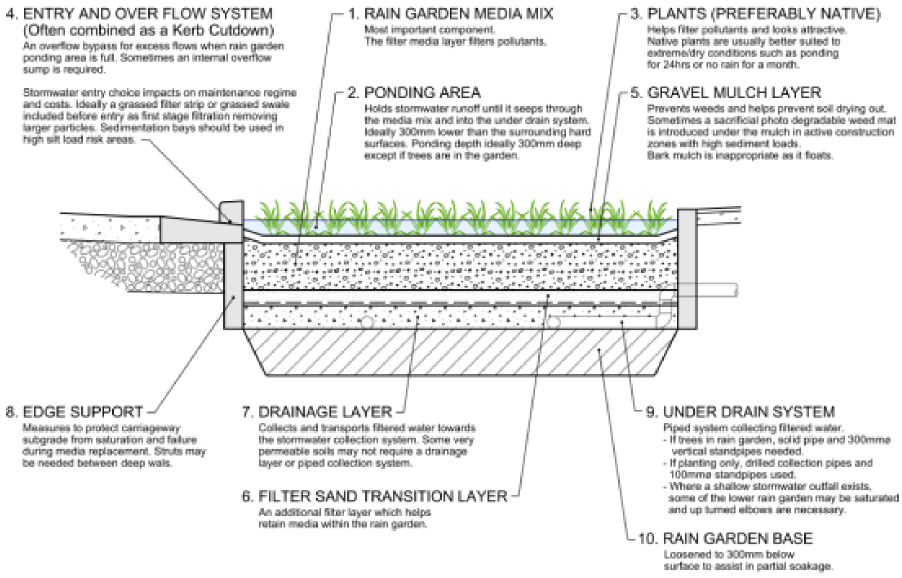


Figure 1 - Typical Rain Garden Cross- Section (Source: CCC Rain Garden design, construction and maintenance manual)

27 Bioscapes are effectively a larger form of rain garden which can be located to receive larger cumulative flows from a catchment or sub-catchment. Bioscapes are proposed to be constructed using Filterra® engineered media. As with raingardens, the treated stormwater along with any by-pass flow will be collected by underdrains to a piped stormwater network and conveyed to detention facilities. Bioscapes have been used extensively internationally and a number have been in installed in New Zealand. Christchurch City Council is in the process of designing a number of bioscapes for treatment of stormwater in the Avon River corridor and other locations.

28 As with raingardens, bioscapes will be fully lined and no groundwater will be able to enter the system, therefore no groundwater take consent will be triggered. Depending on the time of year, the construction of both rain gardens and bioscapes have the potential to intercept groundwater during construction. This can be managed by undertaking construction in the summer months or via temporary construction dewatering consents for the development which will also be required for pipeline construction and other activities.

29 Treated stormwater and bypass stormwater in excess of the Water Quality flow will enter the piped stormwater network which shall be designed to convey primary flows up to the 20% AEP. Flows in excess of the primary flows, i.e. flows in excess of a 20% AEP, will be directed towards the detention basins via roads and dedicated easements/swales. The commercial area will have conveyance pipelines sized for the 10% AEP with treatment via raingardens, bioscapes or proprietary filter devices in cark parks or other green space. Flows will then be conveyed to detention areas.

30 Attenuation is to be provided across the development for management of the post-development discharge to the waterways to ensure that this does not exceed the pre-development runoff from the development. Formalised attenuation will be provided for up to the 2% AEP event by means of attenuation basins located at the end of the stormwater pipe network for sub-catchments. These basins will have controlled outlets discharging into the main flow path corridors. The total peak discharge flow from these outlets will be equal to or less than the peak pre-development. The balance volume will be stored within the detention ponds and released over an extended period of time as the storm recedes. Attenuation tanks are also proposed to capture and attenuate roof runoff for rural-residential areas, however the benefits provided by these storage volumes have not been considered as part of the detention basin volume calculations.

31 The required stormwater attenuation for each catchment up to the 2% AEP (50-year ARI) has been calculated. The volume is based on matching (post-development) the pre-development runoff for the Site during the critical storm duration and taking into account the change in land use. The pre-development peak runoff for each catchment has been calculated using the Rational Method along with appropriate runoff coefficients and time of concentration for the pre-development situation. This is then applied to a calculation of a hydrograph for catchment runoff for each event duration. The pre-developed hydrograph was created using the Standard Rational method. The post-developed hydrograph was developed using the Modified Rational method and using the peak flow of the pre-developed situation. After that, a detention pond for each catchment was sized using the pre-developed peak flow as the target detention pond outflow. The post-developed Modified Rational hydrograph was the pond's inflow hydrograph. This is considered to be a very conservative approach as the pre-development flow calculated is low relative to the equivalent pre-development flow calculated by an alternative methodology using the WDC District flood model hydrology. As the pre-development flow is used to set the controlled outlet flow from the detention basins, a higher pre-development flow would result in a lower volume of storage.

32 The total attenuation required for the development using the Rational Method Hydrograph approach is 26,464 m^3, this will be provided within a number of stormwater detention basins across the

development totalling approximately 52,195 m^2 of basin area. Stormwater detention basins are proposed to be constructed outside of the 2% AEP (50-year ARI) flood level of the flow path corridors. The indicative locations of the proposed storage attenuation basins are indicated in **Attachment 3**.

33 Given the uncertainty regarding highest groundwater depths across the Site as discussed in Paragraphs 18 and 19 of my evidence above, the detention basins have been conceptually designed so as to require no more than 200mm depth of excavation. Stormwater detention basins will be designed to be constructed along the fall of the Site with minimal excavation being undertaken to ensure that storage can be provided without intercepting highest groundwater at the Site. The Site is generally well graded in a west to east direction at a fall of approximately 1 in 200. Low bunding shall be gradually formed along the fall of the topography to retain floodwaters within the basins. Maximum bund height at the downgradient end of the proposed basins shall be approximately 1m in height. The maximum water depth in the pond shall therefore vary from 0m at the upgradient end of the basin to a maximum water depth of 0.8m at the downgradient end of the basin. Maximum depth shall be controlled by a backwater overflow at the upgradient end of the basin which will direct sheet flow to the main flow corridor on the outlet site of the basins. These shall be incorporated into the landscape with appropriate planting treatments. Each basin shall have a controlled outlet, with the total outlet flow from all basins to be no more than the calculated pre-development flow.

34 Some small areas of development at the Whites Road end of the Site will likely be difficult to attenuate in the above manner. In such an event, it is proposed that the treated flow from these areas will discharge directly to the main flow path corridors with no attenuation, but this will ideally be kept to a minimum. To ensure hydraulic neutrality is maintained between pre- and post-development flows, additional compensatory storage will be provided within other stormwater detention basins within the development, along with a reduced basin outflow at those locations, to compensate for those areas that will not be attenuated. The overall impact will be neutrality between pre-development and post-development flows.

35 The total area that can drain to basins is estimated to be approximately 126.4 ha and the area which cannot drain to a basin is approximately 26.4 ha. The area that cannot feasibly drain towards the proposed basins is parallel to Whites Road and the corridor width ranges from 150 m in the south to 220 m towards the north (Ōhoka end), as shown in **Attachment 3**. It should be noted that a significant proportion, approximately 10 ha, of this 26.4 ha will not be subject to increased impervious development due to the protection of key flow paths and the allowance for a large riparian strip along Whites Road and the presence of stormwater detention

basins. However, for the sake of conservatism, the full 26.4 ha was used as unattenuated area in the overall storage volume assessment.

36 The recommended volume of 26,464 m^3 was calculated by considering all 4 sub-catchments (shown in **Attachment 3**) of the Site individually and calculating the storage required for their respective catchment peak event (2% AEP rainfall event). The storage calculation considered the unattenuated areas by subtracting the post development runoff flow for the unattenuated areas (outlined in Table 3) from the pre-development catchment flows (outlined in Table 2) to provide an allowable attenuated outflow for each catchment. Based on this outflow, basin attenuation volumes were calculated and are presented in Table 4.

Table 2: Pre-Development Catchment Flows

Catchment	C-Coeff	Area (Ha)	Tc (min)	2% AEP Flows (m^3/s)
1	0.35	30.68	85	0.88
2	0.35	54.16	43	2.27
3	0.35	51.1925	36	2.41
4	0.35	16.7678	40	0.74
Total		**152.8**		**6.29**

Table 3: Post-Development Catchment Flows

Catchment	C-Coeff	Total Catchment Area (Ha)	Unattenuated Area (Ha)	Catchment Tc (min)	Unattenuated 2% AEP Flows (m^3/s)
1	0.69	40.9	4.4	33	0.44
2	0.70	43.6	4.9	30	0.50
3	0.50	61.7	16.2	26	1.16
4	0.78	6.6	0.9	10	0.19
Total		**152.8**	**26.4**		**2.29**

Table 4: Attenuation Volumes				
Catchment	**C-Coeff**	**Attenuated Area (Ha)**	**Max Outflow (m^3/s)**	**Catchment Attenuation Volume (m^3)**
1	0.69	36.52	0.44	16,547
2	0.70	38.75	1.78	4,527
3	0.50	45.43	1.24	4,861
4	0.78	5.70	0.55	530
Total		**126.4**	**4.0**	**26,464**

37 The above approach was determined to be the most appropriate approach while still retaining a degree of conservatism. As a comparison, the model parameters of the WDC District Model were used to test what the 2% AEP volume difference at the outflow from the Site would be during a 6-hr event. The estimated change in volume was approximately 10,000 m^3 which is significantly less than the 26,464 m^3 of storage proposed.

38 **Attachments 4** and **5** provide a concept and long section for the potential area to be serviced by the lowest basins in Catchments C1 and C2 respectively. The long sections indicate the potential location of rain gardens and how the flow from these will be collected and conveyed through to the respective attenuation basins.

39 In the event that groundwater levels are determined to be deeper in certain parts of the Site during detailed groundwater monitoring, the detention basins can be excavated slightly deeper into the ground. It should also be noted that the basins are generally located alongside existing drains or streams through the Site. These drains are likely to be having a significant local dewatering effect which would be expected to lower the local groundwater levels in the vicinity of the proposed basins. It is considered, the above concept provides a robust and conservative solution that can adequately manage stormwater on the Site and maintain hydraulic neutrality between pre-development and post-development flows. This can also be achieved without the need for basins which actively intercept and take groundwater.

40 As a general rule stormwater treatment areas will be located an appropriate distance away from springs and streams. Untreated stormwater will be managed such that it cannot come into contact with a spring or discharge into a stream. Stormwater detention areas, which will receive treated stormwater and first flush bypass flow will be located away from springs and above the 50-year flood level.

41 It should be noted that up to 2ha of stormwater wetlands or wet ponds can be constructed at the Site as a permitted activity under Rule 5.114 of the LWRP. This provision allows additional flexibility for the location of potential treatment and storage facilities in wetter parts of the Site during detailed design. For the purposes of this proposal, all storage and treatment are provided without the use of wetlands or wet ponds.

PROPOSED WASTEWATER SERVICING

42 The Site is not currently serviced for wastewater. The Site is located between Mandeville to the south and Ōhoka to the north. These areas are serviced by the existing Mandeville Ōhoka wastewater scheme. This consists of two sub catchments. The main catchment, Mandeville, consists of a network of Septic Tank Effluent Pumping (*STEP*) systems which discharge to a central pump station on Bradleys Rd. From here primary effluent is pumped through a pressure main to the Rangiora wastewater treatment plant (*WWTP*). Wastewater from the Ōhoka catchment is collected via a pumped sewer network which connects directly into the Mandeville pressure main as it passes through Ōhoka.

43 It is my understanding from **Mr McLeod's** evidence that the servicing of the Site for wastewater is proposed to be either via conventional gravity or pressure sewer to a new pump station near Mill Road. I concur with **Mr McLeod's** evidence that a pressure sewer network at the Site would be preferable. A pressure sewer network is such that each property has a single pump station with progressive cavity grinder pumps. These pump into a network of welded polyethylene pipes of relatively small diameters which would provide far less opportunity for ingress of groundwater as opposed to a conventional gravity sewer system across the Site. The pressure sewer network would connect to a main pump station within the Site which would in turn pump to the Rangiora WWTP. In an earthquake situation, pressure sewer networks are known to be significantly more resilient and far easier to repair than conventional gravity pipes. In Christchurch they are being used extensively in greenfield areas, particularly South East Halswell, where ground water levels are similarly high.

44 The proposed new pump station for the Site is to connect via a new separate pressure main to the Rangiora WWTP. Assuming that the existing pipeline will follow the general route of the existing pressure main, the proposed pipeline would be approximately 7.1 km long. It would initially have a falling grade to the Cust Drain and would then rise again to the WWTP site. There are a number of obstacles along the route which would need to be considered in design. These include three crossings of Ōhoka Stream Tributaries, a crossing of the Cust Main Drain, a railway crossing at Lineside Road and the ultimate connection to the WWTP. All of these obstacles have been overcome previously in the design and construction of the existing pipeline and, in my view, similar design solutions would be equally

successful for the proposed pipeline and therefore show this is a viable option.

45 In his evidence, **Mr McLeod** raises the possibility of initially connecting the proposed development into the existing pressure main from Bradleys Road pump station until such a time as a reasonable number of lots within the Site have developed. I would concur that there are advantages to this approach. Managing the initial build out of flows from new lots through a new long pipeline would present challenges with respect to odour management at the discharge location and any air valve locations along the pipe route. Connecting the new pump station to the existing pipeline would provide a dual benefit of reducing the hydraulic retention time for the existing pipe situation, as well as allowing for a sufficient level of development to occur within the Site prior to connection of that flow to a new 7.1 km long pipeline. I have carried out some independent analysis to determine if, in my view, there is capacity available in the existing pressure main to cater for this flow.

46 Analysis of historical Bradleys Road pumped wastewater flows from 2021 to 2023 (SCADA data at 1-minute intervals supplied by WDC) indicate that this pump station and pressure main are generally running significantly below the capacity of the system for the majority of the time. The average daily flow over the period assessed was 269 m^3/day compared to a theoretical flow capacity of approximately 2340 m^3/day with a pump at Bradleys Road running continuously through the pressure main, this is just 11.5% of the theoretical flow capacity.

47 However, during certain conditions when groundwater levels in the area west of Mandeville are particularly high, the wastewater flows into the Bradleys Road pump station have been noted to increase dramatically. This was evident in the wastewater record for July/August 2022 and similar groundwater conditions are also noted to have occurred in June 2014.

48 Following a significant series of rainfall events in July 2022, the wastewater flows into Mandeville pumps station were significantly elevated for an extended period of time. Flows from the Bradleys Road pump station exceeded 1,000 m^3/day for approximately 12 days, with the highest daily flow recorded on 31st July 2022 at 1,740 m^3/day. On this peak day, the pump station was operational for more than 17 hours. Therefore, even in very extreme circumstances within the wastewater network, ultimately caused by historically high groundwater levels, there were still approximately 7 hours when the main Bradleys Road pump station was not operational.

49 It can therefore be concluded that, the existing Bradleys Road Pump Station pressure main does have some limited spare capacity under peak flow conditions to receive limited pumped flows from further development in Ōhoka. The utilisation of this capacity would require

direct communication between Bradleys Road pump station and a new pump station at the proposed development. The control logic for both pump stations would need to be established such that both stations cannot pump at the same time and the new pump station would only operate when Bradleys Road pump station storage tank volume was below a certain tank level at the pump station. For the majority of time, there would be little to no restriction on the ability for each pump station to operate.

50 Following prolonged wet periods, such as occurred in June 2014 and July 2022, there is a smaller window of available capacity within the pipeline. Based on the peak day flow from 31st July 2022 there is approximately 6.5 hours of pumping time available into the pressure main. Taking a conservative approach, it is assumed that 4 hours of this would be available to the new pump station on the Site. Assuming a pressure sewer network was established to service the development, the proposed new pump station would have a design flow of approximately 12.4 L/s. At 4 hours of pumping, this would equate to approximately 179 m^3 of available wastewater capacity in the pipeline on a peak day. This equates to approximately 260 lots worth of pumping capacity.

51 In order to avail this capacity, sufficient buffering capacity would need to be available within the Site's wastewater system to store and buffer flows while the Bradleys Road pump station is operating. The proposed new pump station will have a standard requirement for emergency storage to be provided in order to allow a minimum response time for power failures or major mechanical issues. This requirement is typically 8 hours of storage at average flows, for the proposed Site pump station this equates to approximately 233 m^3 of storage which would need to be constructed at the same time as the pump station.

52 While this total emergency storage allowance would be available from the date of construction, the total storage provided would be significantly greater than the actual emergency storage required as the development commences and progresses. For example, at 100 lots of development the actual 8-hour emergency storage requirement is 23 m^3. As development progresses, the emergency storage requirement gradually increases to meet the total storage provided at the time of construction. At 250 lots worth of development completed, the emergency storage requirement is 57 m^3 of storage, leaving an additional 176 m^3 of storage potentially available to buffer flows at the pump station while Bradley Road pump station is operating. This available storage of 176 m^3 is more than 24 hours of total daily wastewater volume for 250 lots. This would be more than sufficient to buffer flows into Bradleys Road during peak periods of flow. I would therefore conclude that there is capacity within the Bradleys Road pressure main for a minimum of 250 lots from the Site during peak periods of flow. Above 250 lots, a new pressure main to Rangiora WWTP would need to be constructed.

53 The WWTP has been upgraded over the last 10 years to create significant additional capacity for growth within the district. Upgrades include the construction of a new Aeration Basin as well as construction of a new inlet works structure which was designed to receive up to 960 L/s as an ultimate design flow. I understand that, at present, the inlet works have the capacity to receive approximately 750 L/s which can be increased to the ultimate design flow of 960 L/s via modifications to the inlet screens within the inlet structure. It is recognised that the existing treatment capacity of the WWTP does not match this ultimate design flow. My understanding from the evidence of **Mr McLeod** is that the current treatment capacity of the WWTP is 33,000 Population Equivalent.

54 Further upgrades to the WWTP and the downstream disposal infrastructure are planned within the current LTP and have been earmarked for future LTP's. At this trunk infrastructure level, upgrades are considered on a district wide growth basis. It is not anticipated that the rezoning of this Site would have a significant impact on the district wide rate of growth within the Waimakariri District particularly given the development will be staged. Upgrades to the Rangiora WWTP, and the timing of such to cater for district wide growth, will be considered by WDC though the three yearly LTP review process and timed accordingly.

CONCLUSION

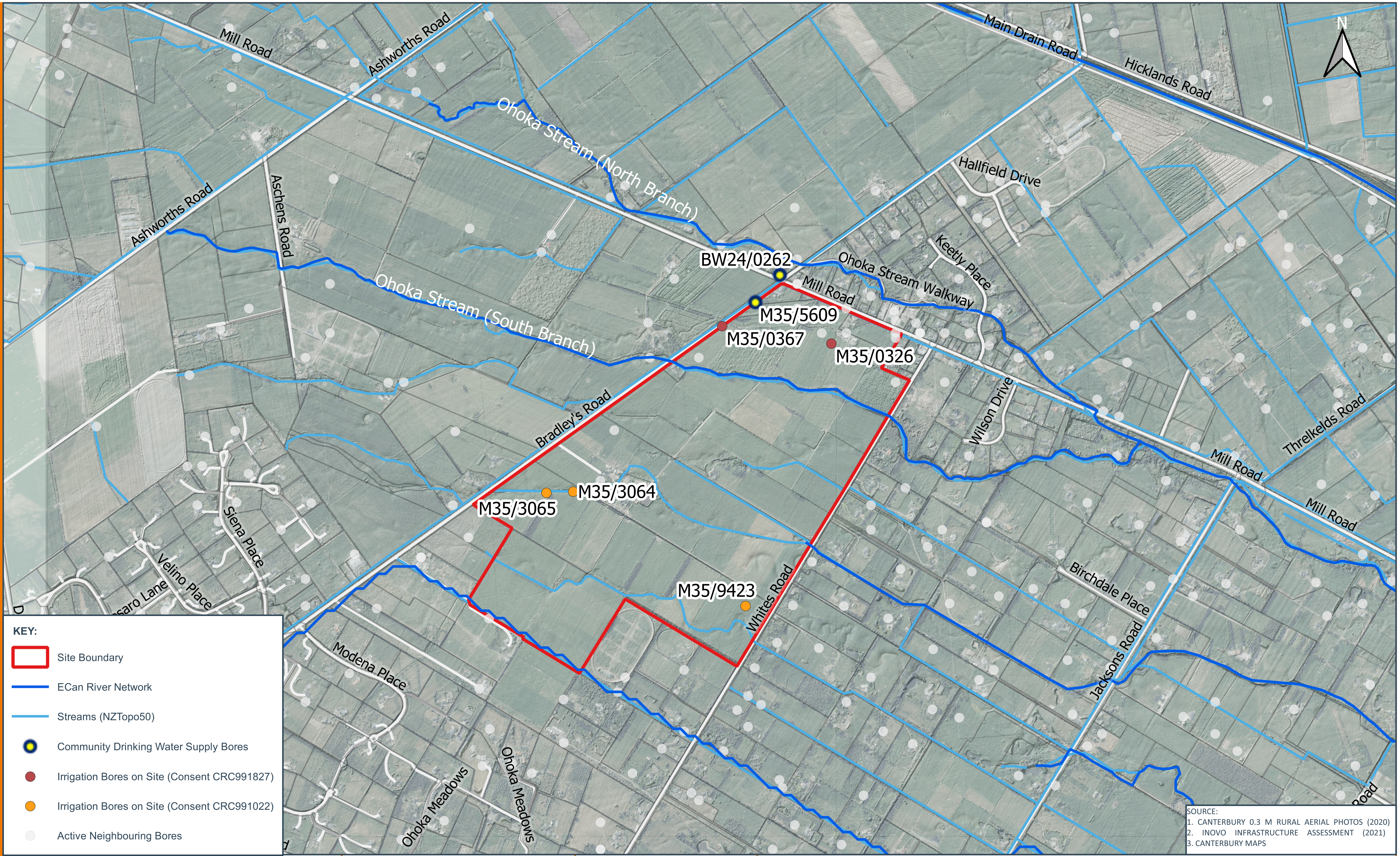
55 I am of the view that viable stormwater and wastewater concepts exist for the servicing of the proposed rezoning request.

Dated: 5 March 2024

____

Eoghan O'Neill

ATTACHMENT 1



pdp

0 200 400 600 m

METRES

SCALE : 1:20,000 (A4)

NO.	REVISION	DATE	BY
A	FINAL	FEB 24	TG

CLIENT

ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

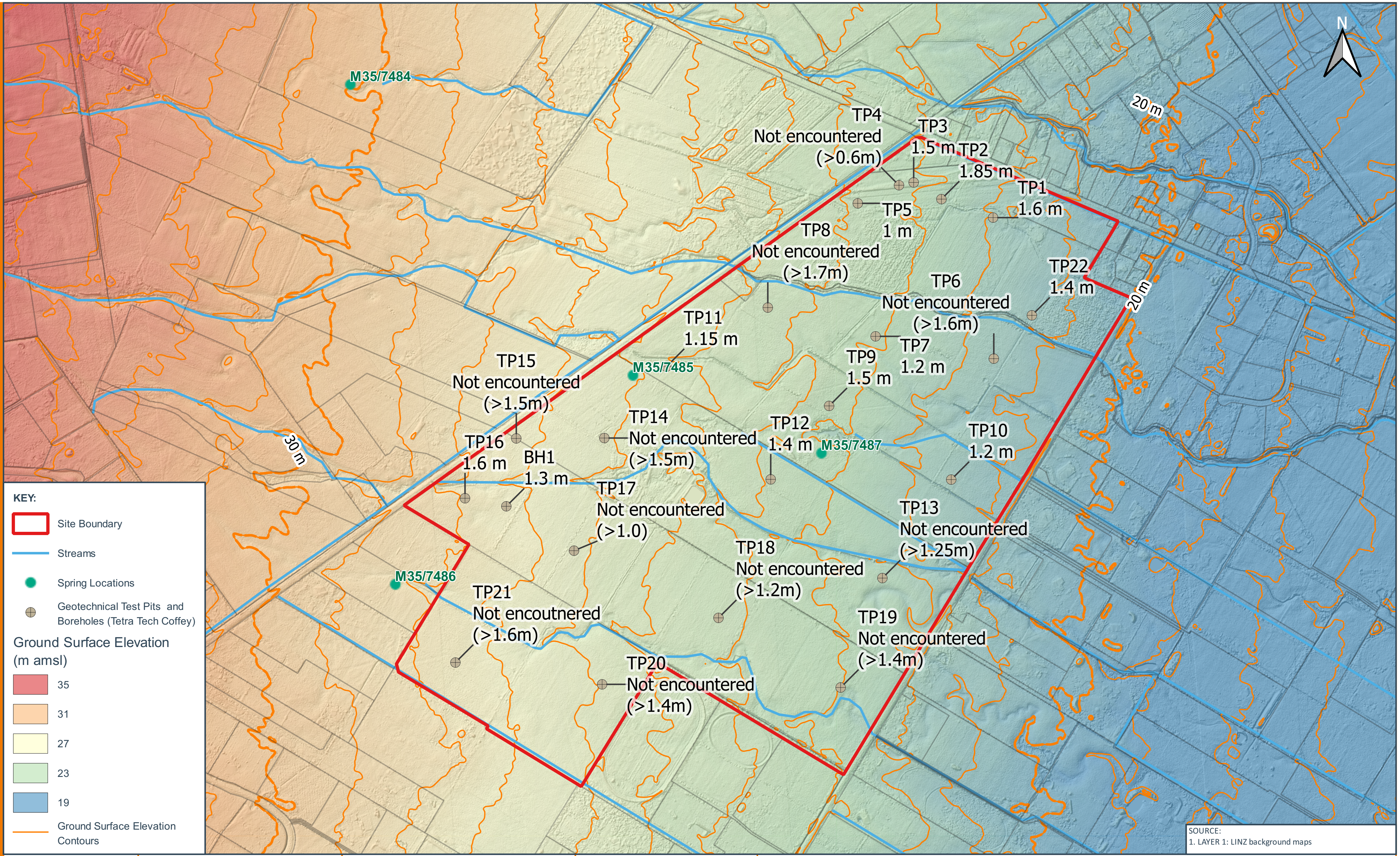
FIGURE

ATTACHMENT 1: SITE OVERVIEW

PROJECT

PRELIMINARY WATER SUPPLY ASSESSMENT - OHOKA PLAN CHANGE

ATTACHMENT 2



S:\C04518\500\Projects\C04518500_Z004_GW_levels.qgz

pdp

0 100 200 m

METRES

SCALE : 1:12,000 (A4)

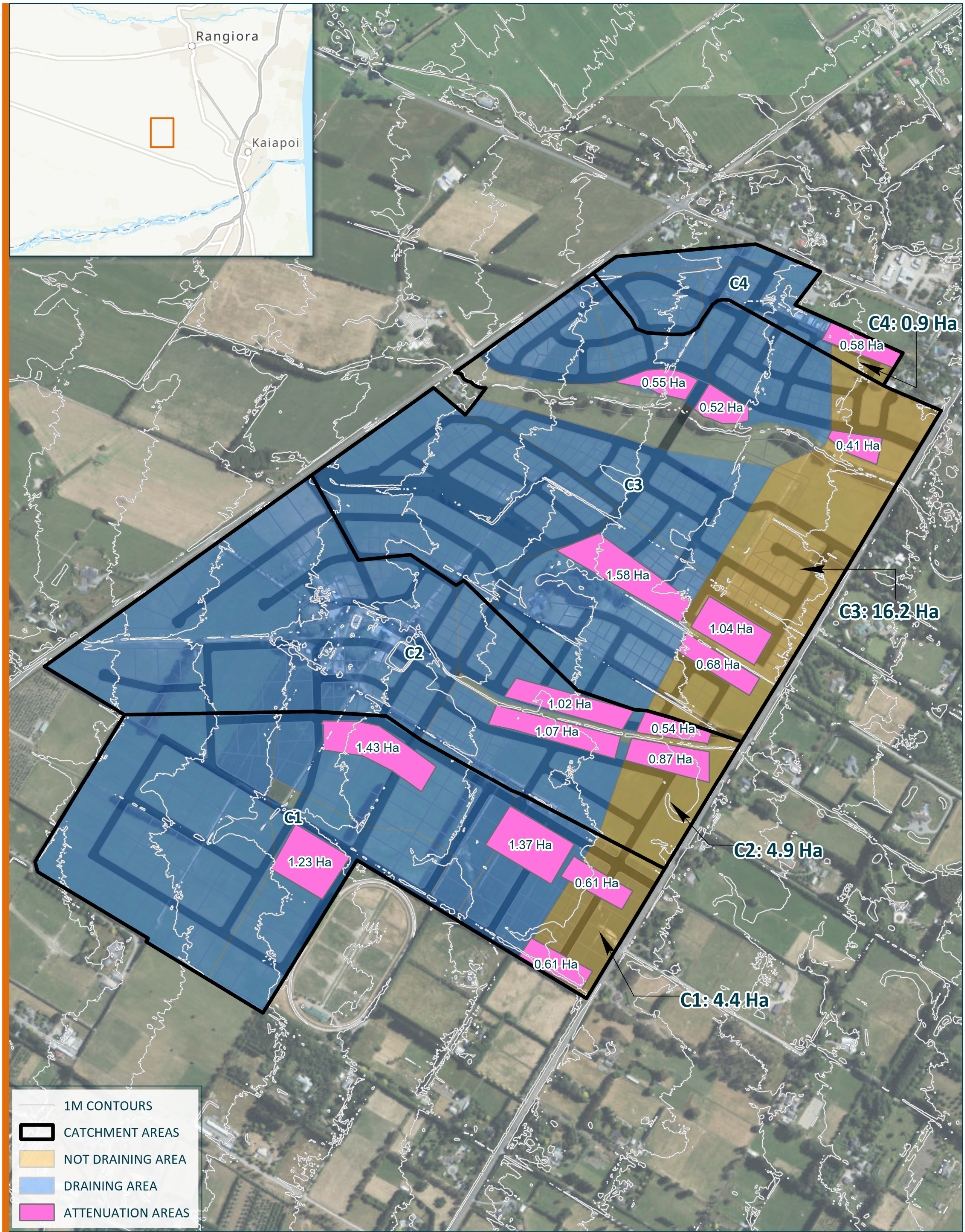
NO.	REVISION	DATE	BY
A	FINAL	JAN 22	TG

CLIENT
ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

FIGURE
ATTACHMENT 2: SITE TOPOGRAPHY & GROUNDWATER LEVELS

PROJECT
HYDROLOGICAL ASSESSMENT - OHOKA PLAN CHANGE

ATTACHMENT 3



pdp

ATTACHMENT 3: ATTENUATION AREAS AND BASIN LOCATIONS

OHOKA STORMWATER INVESTIGATION

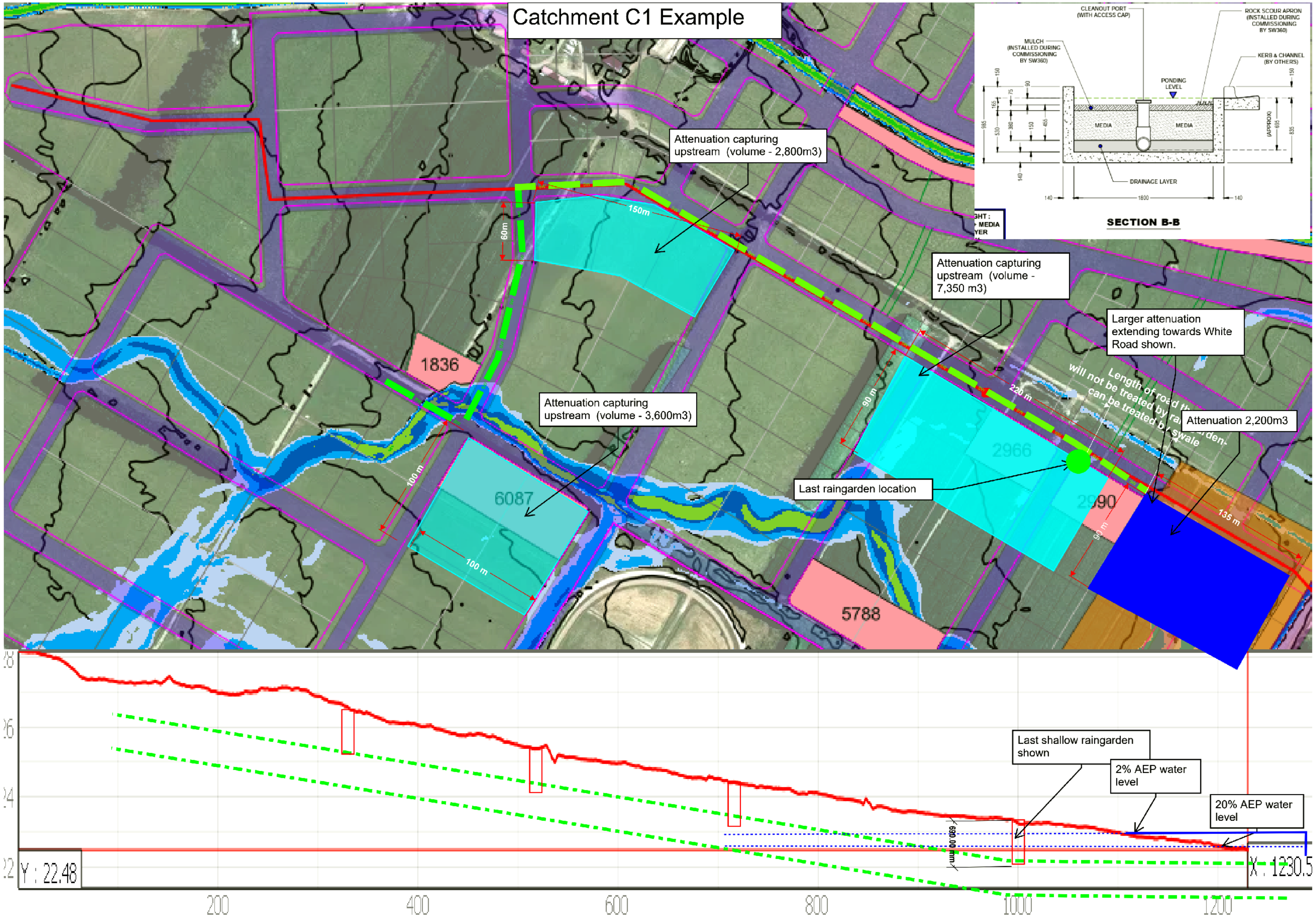
SOURCE:
1. AERIAL IMAGERY: EAGLE TECHNOLOGY, LINZ, STATSNZ, NIWA, NATURAL EARTH, © OPENSTREETMAP CONTRIBUTORS., DEPRECATED BASEMAP - EAGLE TECHNOLOGY, LAND INFORMATION NEW ZEALAND, EAGLE TECHNOLOGY, LAND INFORMATION NEW ZEALAND, GEBCO, COMMUNITY MAPS CONTRIBUTORS.

0 150 300
METRES
SCALE : 1:10,000 (A4)
N

REVISION: 001 | DATE: AUG 23 | BY:CB
CLIENT: ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

Figure 1

ATTACHMENT 4



ATTACHMENT 4: CATCHMENT C1 EXAMPLE

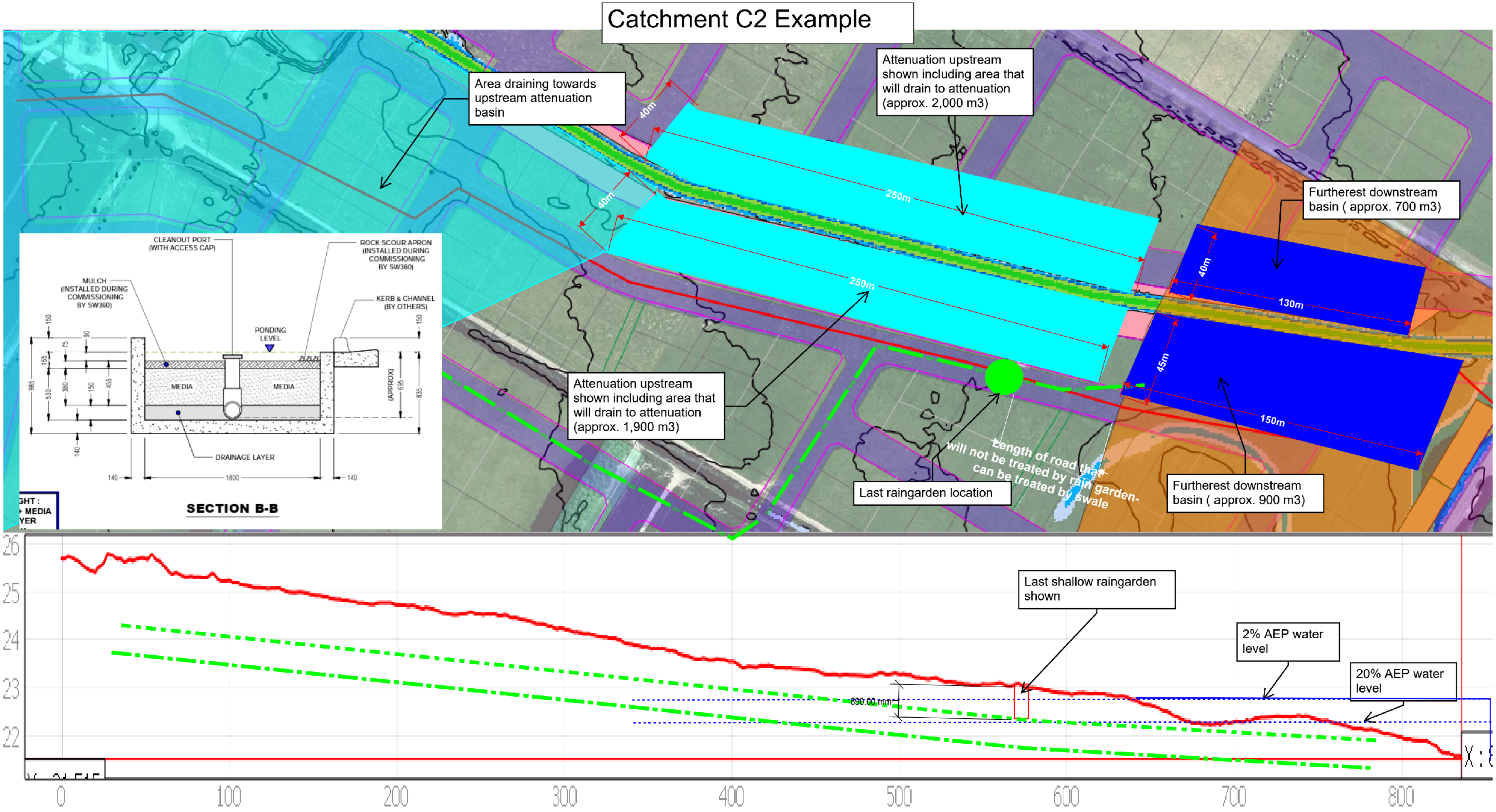
OHOKA STORMWATER INVESTIGATION

N

REVISION: 01 | DATE: FEB 24 | BY: JR
CLIENT: ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

C04518300_GIS008

ATTACHMENT 5



ATTACHMENT 5: CATCHMENT C2 EXAMPLE

OHOKA STORMWATER INVESTIGATION

N

REVISION: 01 | DATE: FEB 24 | BY: JR

CLIENT: ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

C04518300_GIS007