BEFORE THE INDEPENDENT HEARINGS PANEL
APPOINTED BY WAIMAKARIRI DISTRICT COUNCIL

UNDER	the Resource Management Act 1991
AND	
IN THE MATTER	of Submissions on the Waimakariri District Plan Review – Hearing Stream 12

STATEMENT OF EVIDENCE OF JAMES MATTHEW PHELPS HOPKINS

for **Rainer Hack and Ursula Hack (Submission Number 201), relating to 110 Parsonage Road**

Services

Dated: 4 March 2024

EXPERIENCE

1. My name is James Matthew Phelps Hopkins.

2. I have a Bachelor of Technology degree in Environmental Engineering from Massey University.

3. I am a Chartered Professional Engineer with 25 years' experience, predominantly in land development.

4. My work experience includes comprehensive involvement in land development projects from plan change, through consenting, detailed engineering design and civil construction.

5. I have previously prepared of evidence for a number of notified resource consents and private plan changes in Canterbury and Nelson.

6. I have been engaged for this development to provide civil engineering services during the plan change phase, focussing particularly on stormwater.

7. In preparing this statement of evidence I have considered the following documents:

    a. Waimakariri District Council Plan Change

    b. ECan LWRP

    c. Waterways Wetlands and Drainage Guide

    d. ECan 200 year flood maps (accessed on ECan online GIS system)

    e. Pre-application meeting minutes and advice

    f. Preliminary consultation advice from Te Ngāi Tūāhuriri Rūnanga

    g. Outline Development Plan (ODP) – prepared by Align.

8. My evidence focuses primarily on stormwater management elements for the site, but also addresses wastewater, water and roading/access.

9. Unless stated otherwise, all statements in my evidence are my own opinion.

CODE OF CONDUCT

10. Although these proceedings are not before the Environment Court, I have read, understood, and will comply with the Code of Conduct for Expert Witnesses contained in the Environment Court's Practice Note 2023. This evidence has been prepared in accordance with this Practice Note and I agree to comply with it. Except where I state that I am relying on the evidence of another person, I confirm that this evidence is within my area of expertise. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed in this evidence.

SCOPE OF EVIDENCE

11. I have prepared evidence in relation to:

    a. Stormwater quantity

    b. Stormwater quality

    c. Flooding

    d. Water Supply

    e. Wastewater Disposal

    f. Roading and Access

SITE DESCRIPTION AND POTENTIAL DEVELOPMENT

12. The subject site for the submission (110 Parsonage Rd) is comprised of land that is currently zoned rural and utilised as a lifestyle agricultural and/or grazing purposes associated with the existing historic dwelling at 110 Parsonage Road. The total plan change area is approximately 3.7 ha.

13. The indicative layout proposed for the site would yield approximately 31 new lots in a mixture of medium density townhouse/terrace form, 280 m^2 - 350 m^2 lots), low density residential (approx. 520 - 660 m^2) and low density/rural fringe lots (approx. 1,400 – 2,000 m^2). It is proposed that the existing dwelling will be retained on a balance lot of approximately 7,400 m^2.

14. The concept plan, prepared by Align, is attached to the planning evidence of Victoria Edmonds.

15. The site currently has a single dwelling and associated ancillary buildings generally located in the northeastern corner of the site. The remainder of the site is a mixture of pasture and mature trees. The site is quite flat, with the only appreciable level differences being between the area around the existing dwelling (typical RL 9.0 - 9.5) with the remainder of the site being very flat, typically between RL 7.6 (southwest corner) and RL 8.5 (the majority of the site.

16. The site topographic survey, completed by Graham Surveying, is included in **Appendix A.**

WAIMAKARIRI DISTRICT PLAN REVIEW

17. As Victoria Edmonds has provided good background on the Waimakariri District Plan review and the submission associated with 110 Parsonage Road, I will not provide any commentary on this.

PRE-APPLICATION MEETING

18. I have noted the information provided in the pre-application meeting notes provided by Waimakariri District Council.

VALUES OF TANGATA WHENUA

19. Ngāi Tahu, as tangata whenua, holds ancestral and contemporary connections with Canterbury, governed by the Te Rūnanga o Ngāi Tahu Act 1996 and Ngāi Tahu Claims Settlement Act 1998.

20. Preliminary Cultural advice has been received from Te Ngāi Tūāhuriri Rūnanga, which provides guidance to avoid, remedy or mitigate any effects on cultural values.

21. The advice document also provides a number of specific recommendations, the key specific recommendations are:

    - Recommendations include surveys for indigenous species and springs.
    - Treatment of stormwater before discharge is advised.
    - Low-impact design methods, such as rain and greywater collection, are encouraged.
    - Planting indigenous species is recommended to enhance habitat.

22. The guidance and advice has been incorporated into the preliminary stormwater design detailed below.

STORMWATER QUANTITY

PRELIMINARY STORMWATER TECHNICAL REPORT

23. A technical stormwater report was prepared in September 2023 and is attached as **Appendix B.** Some of the findings of that report have been refined further in this evidence.

PRE-DEVELOPMENT CATCHMENTS AND DRAINAGE

24. The site is currently predominately pasture, with a number of large mature trees.

25. There are no visible on-site stormwater drainage, channels or pipework.

26. Test pits excavated on site in 2023 revealed that the site is underlain with silts, to at least 2 m depth, resulting in relatively poorly draining soils, which typically result in a relatively high runoff coefficient. The measured infiltration rate of the already damp soils was between 10 and 20 mm/hr. This silt layer will mean that discharge to ground (soakage) will not be practical.

27. Photos from my site visit and soakage test pits completed on 29 August 2023 are included in **Appendix C.**

28. The topographic survey completed by Graham Surveying shows that the exiting ground level on site varies between RL 7.5 in the southwest corner, and RL 9.5 around the existing dwelling.

29. Piezometric contours (Waimakariri June 2010 Shallow Wells) on canterbury maps indicate that groundwater could be expected at around RL 5.0 to RL 6.0.

30. The site naturally grades towards the south (i.e. towards the extension of Parsonage Road), where any surface runoff is collected by the existing Parsonage Road roadside drain, which in turn is collected by a sump adjacent to the entrance to number 100 Parsonage Road. This sump discharges into an un-named open drain between numbers 97 and 107 Parsonage Road. This drain flows south, connects with Eders Road drainage, then crosses Gladstone Road and enters the circa 2018 open drainage network through Ranby Place and Fearne Drive. The drain then crosses Petries Road, where it becomes known as McIntosh Drain. The McIntosh Drain flows generally

southwards for approximately 7 km where it discharges into the Kaiapoi River, near the confluence with the Waimakariri River. For the rest of this evidence the entire downstream network, from where the open drain connects to Parsonage Road, will be referred to as the McIntosh Drain.

31. The McIntosh Drain is stated by WDC as having no spare capacity. Accordingly, the proposed development must achieve stormwater neutrality for events up to and including the 50 year (2% AEP) event.

32. The McIntosh Drain is several kilometres long, with a flat gradient, is heavily tidally influenced and Waimakariri Council have advised that the drain has a Time of Concentration at the point of discharge (Tc) = 2 hours.

33. There is no spare capacity in the recently constructed stormwater management area to the west, thus stormwater management for any development of the subject site will need to be managed on-site.

34. The pre-development catchment summary is as follows:

    a. Pasture/Permeable 35,940 m²

    b. Hardstand/pavement 535 m²

    c. Roof 426 m²

    d. TOTAL AREA 36,900 m²

POST-DEVELOPMENT CATCHMENTS AND DRAINAGE

35. Once developed (according to the indicative masterplan layout) the site will contain modern best engineering practice stormwater solutions, which is proposed to include some "low impact" design features such as swales and raingardens. Critically the system will need to contain stormwater attenuation, which is proposed in the form of stormwater management pond (or ponds) with restricted outlets to manage the discharge flow rate to ensure it is less than or equal to the predeveloped scenario.

36. In general the stormwater system is proposed to comprise of:

    a. Roof and hardstand runoff from individual lots will discharge via pipes to either kerb outlet in the street, or an underground pipe network.

b. Road runoff will be collected in road side kerb and channel, where it is collected in sumps and discharged to an underground pipe network.

c. The underground pipe network(s) will discharge to a treatment device (such as a proprietary filter, swale, raingarden or similar).

d. Treated stormwater will then progress to a retention pond(s) with a restricted outlet that will limit the flow rate being discharged to the McIntosh Drain. These ponds will fill up throughout the duration of rain events, but designed to drain within 48 hours of cessation of rain.

e. In rain events that exceed the design capacity of the ponds (i.e. events exceeding the 50 year design criteria), flows will spill via spillway and discharge un-restricted to the McIntosh Drain.

f. The utilisation of rainwater tanks on individual dwellings is also a consideration. It is proposed that the adoption of these, and the analysis of the performance as part of the overall system would be considered at the subdivision consent stage. The use of rainwater tanks on individual sites may help to reduce the amount of land required for a stormwater management pond whilst also providing for some beneficial re-use of water. However the stormwater system can be designed to meet the design criteria either with or without rainwater tanks.

37. For the purposes of this analysis, it has been assumed that the entire plan change area would be managed via a centralised stormwater attenuation area.

38. At the time of subdivision consent and or engineering design it may be determined that the site would be better managed using more than one stormwater management area.

39. The post-development catchment summary is as follows:

a. Permeable/Landscape 23,280 m²

b. Hardstand/pavement west 7,560 m²

c. Roof west 6,060 m²

d. TOTAL AREA 36,900 m²

40. The proposed development layout and recommended stormwater management area is shown in the Stormwater Concept Plant in **Appendix D.**

41. It should be noted that the detailed modelling (at consent and engineering design stage) will need to account for the fact that a full stormwater pond has 100% impermeable equivalence, as all rain that lands on the pond contributes directly to and increases its volume. For simplicity of the model pond areas have been added to the roof area (at C = 0.9).

RAINFALL INTENSITY

42. The HIRDS V 4.0 rainfall for both historic + 16% and RCP 8.5 (2081-2100) were considered, and due to the similarity in these the more conservative (higher) rainfall figures in RCP 8.5 (2081-2100) were adopted for this analysis. A table of these HIRDS rainfall intensities is included in **Appendix E.**

RATIONAL METHOD RUNOFF CALCULATIONS.

43. Rational method calculations provide a good high-level indication of the scale of the effects of development in terms of runoff flow rates and volumes.

44. For the purposes of this analysis, it has been assumed that the pre-developed runoff coefficient for pasture/grass areas is 0.35. This is derived from (NZBC E1/VM1 Table 1 – Heavy clay soil types – pasture and grass cover C = 0.4), with a -0.05 adjustment for flat sites, making the net runoff coefficient of C = 0.35 (for Landscape areas).

45. Allowing for the higher runoff from the existing dwelling (roof at C = 0.9 and unsealed hardstand at C = 0.6) the weighted average runoff coefficient for the undeveloped site, C = 0.36.

46. The post-developed runoff co-efficient has been calculated, for the concept layout plan, using the weighted C method to be 0.535. This represents a mixture of medium density (townhouse multi-unit), residential density and low density/transition allotments, roading and reserve and aligns closely with the common average of C 0.55 for residential developed areas being specified in the Christchurch City Council Waterways Wetlands and Drainage Guide (WWDG).

47. This implies that, when calculated via the rational method, the peak runoff in the developed scenario will be approximately 48% greater than pre-development scenario.

48. The time of concentration within the site has been estimated using NZBC E1 VM1 (2.3.2/Figure 1) to be 25 minutes, (although note this is of little relevance to the

attenuation calculations where the critical storm duration at the point of discharge is 2 hours).

49. The pre-developed runoff (peak flowrate) has been calculated to be 97.1 L/s for the 50 year (2% AEP) 2 hour event, with a runoff volume of 699 m^3.

50. The post-developed runoff (peak flowrate) has been calculated to be 144.1 L/s for the 50 year (2% AEP) 2 hour event, with a runoff volume of 1,037 m^3.

HEC HMS HYDROLOGICAL MODEL RUNOFF CALCULATIONS.

51. Hydrological models, such as HEC HMS, can provide a more detailed analysis of the runoff from the site, and includes allowances for ponded water, initial losses, better consideration of runoff infiltration to ground (based on soil type) and arguably provides a more realistic analysis of the runoff from the site. The preliminary design of the stormwater management (attenuation) requirements has therefore been completed using HEC-HMS modelling. Care is required when comparing Rational Method figures with HEC-HMS figures, I have noted this where necessary during my following discussion points.

52. For the HEC HMS model the following Parameters have been assumed:

a. A lag time of 0.6 x Tc = 0.25 hr (15 minutes).

b. Pasture areas have been assumed to have a SCS curve number of 74, derived from Auckland Regional Council TP108 for group C soils.

c. Rainfall Hyetograph as per CCC WWDG fig 21-8

d. Antecedent rain of 15 mm over 24 hours (at 0.625 mm/hr)

e. Catchment parameters as per Table 1 and Table 2 overleaf.

53. **TABLE 1**

Catchment	Area	SCS	Impermeable	Lag
Pavement	535 m^2 (0.000535 km^2)	98	100%	10 min
Roof	426 m^2 (0.000426 km^2)	98	100%	10 min

Permeable	35,940 m^2 (0.03594km^2)	74	0%	10 min
TOTAL	36,900 m^2			

TABLE 2

Catchment	Area	SCS	Impermeable	Lag
Pavement	7,560 m^2 (0.00756 km^2)	98	100%	10 min
Roof	6,060 m^2 (0.00606 km^2)	98	100%	10 min
Permeable	23,280 m^2 (0.02328m^2)	74	0%	10 min
TOTAL	36,900 m^2			

54. Using the SCS curve method the Pre-Development runoff in the 2% AEP (50 year) 2 hour event is calculated to be 238 L/s, with a total runoff volume of 757 m^3.

55. This substantially higher peak runoff (flowrate) compared to the rational method is due to the use of a hyetograph in the SCS model where the peak rainfall is twice the average, as per WWDG figure 21-8. This is a more realistic representation of a rain event than the rational method, which in essence assumes the rain event comprises of the average rainfall intensity evenly throughout the rain event.

56. Using the SCS curve method the unattenuated Post-Development peak runoff in the 2% AEP (50 year) 2 hour event is calculated to be 326 L/s, a 37% increase in peak flow rate.

57. Introducing stormwater attenuation areas to the model enables the peak outflow flow rate to be reduced to less than the predeveloped flow rate. The volume of attenuation required is never as simple as comparing the predeveloped volume and post developed volume, it is a complex calculation that generally requires models such as HEC-HMS which account for the multiple dynamic inputs.

58. A suitable attenuation solution is dry storage ponds with restrictive outlets.

59. The pond has been modelled at 1100 m^2 (at maximum depth of 1.3 m) providing an operational volume of 636 m^3. This maximum depth of 1.3 m is is 100mm less than the

limit set by the invert of the McIntosh Drain at the discharge point, adjustments to the area, depth and orifice diameter would enable further optimisation of the pond.

60. Table 3 below sets out the performance of the attenuation pond with a 190 mm orifice and a maximum operational depth of 1.3 m.

61. **TABLE 3**

AEP(ARI)	Pre-dev Q (L/s)	Post-dev Q (L/s)	Attenuated Q (L/s)	% reduction in Q	Peak Vol V (m^3)
20% (5 yr)	82	145	85	+3.7%	159
10% (10 yr)	119	191	95	-20%	263
5% (20 yr)	164	244	105	-36%	391
2% (50 yr)	238	326	117	-51%	636

62. The 5 year (2 hour) pre-developed runoff flow rate is important as this has been selected to be the design event for which flow neutrality will need to be set. All events from this event, up to and including the 2% (50 year) 2 hour event will be attenuated. While it might be noticeable that the flow has increased by 3.7% for this event, simple detailed design adjustments to the pond size and/or orifice can ensure this is corrected.

63. Since there is no soakage to ground available due to underlying soil types, the net runoff volume will not be able to be decreased by discharging stormwater to ground. However, an attenuation pond will ensure that the increased volume is discharged at a lower flow rate but over a longer duration, thus mitigating potential to worsening flooding effects in the McIntosh Drain in critical duration events.

ATTENUATION VOLUME AND LAND AREA REQUIRED

64. In order to provide an appropriate land area to achieve this it is suggested that the development will need to set aside a total land area in the order of 1,100 m^2 (approximately 3% of the total land area) plus ancillary areas for access, maintenance and landscaping. This may be provided as a combination of ponds, raingardens, swales and rainwater tanks.

65. It is noted that the reserve area between the two terrace housing blocks is approximately 1,750 m² and therefore provides more than enough area to accommodate the stormwater attenuation needs for the development.

66. The concept stormwater plan in **Appendix D** shows that significantly more land than this area is available without modification to the number of lots possible. Some of the stormwater management land may be mixed use (i.e. recreation reserve during dry weather, and stormwater management during wet weather).

STORMWATER QUALITY

67. The quality of stormwater must also be managed so that the quality of the receiving waters is not adversely affected by the proposed development.

68. Guidance from Te Ngāi Tūāhuriri Rūnanga includes the use of low impact design methods.

69. It is proposed that all runoff from hardstand areas will be treated by rain gardens planted with indigenous species, or similar low impact designs. The provision of space suitable for stormwater treatment via low impact systems has been included in the Stormwater Management Concept Plan.

70. Low impact features such as raingardens also provide a volumetric attenuation benefit, and these may help reduce the volume of attenuation required. Preliminary modelling has not included raingarden volumes in the attenuation calculations, thus making this preliminary analysis conservative.

71. The likely locations of suitable rain gardens are shown in the stormwater concept plan in **Appendix D.**

72. The quality of runoff from the site during earthworks during construction often provides the biggest risk of contaminants in the form of silts entering the receiving waters. At the time of subdivision consent a detailed stormwater management plan, including erosion and sediment control during the earthworks phase will be developed. This will be completed either in accordance with ECan permitted activity rules, or if necessary, via a resource consent in a way that mitigates the risks of sediment discharges.

STORMWATER MANAGEMENT CONCLUSION

73. The quality of stormwater must also be managed so that the quality of the receiving waters is not adversely affected by the proposed development.

74. Guidance from Te Ngāi Tūāhuriri Rūnanga includes the use of low impact design methods.

75. Stormwater Quantity and Quality management can be achieved by best engineering practice designs in general accordance with the stormwater concept plan in **Appendix D.**

FLOODING AND FLOOR LEVELS

76. ECan 1:200 flood modelling (accessed on ECan online GIS system) was reviewed and does not show any significant flooding on the site, so therefore area wide flooding of the site is not a concern.

77. The recent development to the west (upgradient) has a stormwater management area that will manage runoff from the upgradient land.

78. The New Zealand Building Code (NZBC) requires that floor levels are set 100 mm above the 50 year flood level

79. Where the 50 year flood level may result in water greater than 100 mm deep in a road (where vehicles may drive through the water creating bow waves) and additional 300 mm freeboard is required, making a total of 400 mm.

80. Standard road design, with adjacent lots that grade towards the kerb and channel, in combination with dwellings that have a floor level of at least 150 mm above existing ground (more depending on dwelling floor and cladding design) easily exceeds the requirement of the 100 mm freeboard requirement, with finished floor levels (concrete floors with brick cladding) typically 200 – 250 mm above the crown of road.

81. The placement of excavated materials from cutting to road subgrade, excavation of trenches and excavation of a stormwater management area could provide additional material that can be placed as engineered fill to further increase the finished ground level of any lots in areas that require it. This could be to, say, 250mm above the crown of road. This would make compliance with a freeboard of 400 mm easier at the time of dwelling construction since the minimum FFL above finished ground is 150 mm.

82. The proposed stormwater management area (pond) and associated secondary flow paths would be the most likely cause of water exceeding 100mm depth, however given the relatively small scale of the development it is highly unlikely that secondary flows will actually exceed 100mm.

83. Secondary flow paths both to and from the stormwater management area can easily be accommodated within road reserve, ROW areas and pedestrian link paths. There is no anticipated need for secondary flow paths within private property.

84. Regardless, it is practicable to ensure that finished floor levels of dwellings are 400 mm above the 100 year flood level where this becomes a requirement.

85. If a requirement were to be set regarding finished floor levels this would need to be created so that it could accommodate both scenarios where 100 mm freeboard would be adequate (i.e. dwellings adjacent to a Road or ROW where there is no flooding in the 100 year event) and 400 mm (i.e. where a dwelling is adjacent to a Road or ROW where flooding may exceed 100 mm and vehicles are likely to drive through that water).

86. Therefore, the proposed change of land use from rural to residential is not inhibited by existing flood conditions, nor will it create hazardous flood conditions within the site, or on adjacent/downstream land.

WASTEWATER

87. The site is not currently served by a reticulated wastewater network, and it is understood that the existing dwelling is served by an on-site wastewater treatment and disposal facility in the form of a septic tank.

88. There is a relatively new wastewater pump station on Parsonage Road, approximately 200m to the west of the site, with an invert of sufficient depth to facilitate a gravity connection from the site.

89. It is practicable to lay a new gravity wastewater network, with individual laterals serving each new allotment, and discharging to the pump station (or into one of the gravity pipes that already discharges into the pump station).

90. Therefore, wastewater management for the site can be managed via reticulated gravity wastewater network as a matter of routine engineering design at the time of subdivision consent.

91. Using flow estimation methods in the Waimakariri Engineering Code of Practice Part 6 it is anticipated that the flows will be as follows:

- Average Dry Weather Flow (ADWF) = 32 lots x 250 l/person/day x 2.7 persons per lot

= 19,000 L/day = 0.25 L/s

- Maximum Flow = ADWF x PWWF x PDWF
- Maximum flow = 0.25 x 4 x 2.5 = 2.5 L/s

92. A concept wastewater plan is included in **Appendix F.**

WATER AND FIREFIGHTING

93. Waimakariri GIS does not show any water reticulation serving the site, or along the site frontage.

94. The nearest watermain is a DN180 (150mm) PE main located at the corner of McQuillan Avenue and Parsonage Road. This water main terminates with a fire hydrant just to the east of a DN180/DN180 Tee.

95. A DN63 submain extends east on the northern berm of Parsonage Road, serving 6 lots on Parsonage Road and the wastewater pump station.

96. The DN63 submain is not adequate to provide the water supply needs of the proposed development as it is not capable of providing firefighting flows, so an extension of the DN180 main will be required.

97. A DN180 main extending to, and into the site would be able to provide the potable water requirements and firefighting requirements of the proposed development, provided Council network modelling confirmed that the existing network has adequate flow/pressure to do so.

98. Therefore, water supply to the site for potable and firefighting purposes is practicable and can be managed via reticulated water network as a matter of routine engineering design at the time of subdivision consent.

99. The water supply demand has been calculated based on the Waimakariri District Council Engineering Code of Practice Part 7 as:

- 32 dwellings x 0.1 L/s/dwelling = 3.2 L/s peak hourly flow
- Fire demand = 2 hydrants at minimum 12.5 L/s each = 25 L/s

100. Assuming 2 hydrants and 50% of the peak hourly residential flow, totalling 28.2 L/s the pressure loss in a 150mm (internal diameter) pipe is 1 metre head loss per 48m pipe

length. Therefore, the approximately 450m of DN180 pipe would result in 9.4 m head loss.

101. In order to achieve the required 100 kPa residual pressure at the last point of supply with a total demand of 28.2 L/s, the source at the existing hydrant in McQuillan Ave, where the extension will connect, will need to be at least 19.4 m, which it is understood that the existing supply will exceed, although this would need to be confirmed by WDC asset engineers.

102. A concept water supply/fire hydrant concept plan is included in **Appendix G.**

ROADING AND ACCESS

103. Parsonage Road legally terminates approximately halfway across the frontage of the site, at the location of the existing entrance to the existing dwelling.

104. Upgrading the formation of Parsonage Road, from the entrance to 100 Parsonage Road, for approximately 130 m, up to the entrance to the existing dwelling (which will become the proposed ROW to the eastern low density/transition lots) will be required.

105. It is anticipated that this upgrade will be to the same standard as the recent upgrade to Parsonage Road for the recent subdivision to the west.

106. A new road and ROW's serving the high density and residential density lots will need to be formed. The new road would also provide access to the main proposed stormwater management area for maintenance purposes.

107. These proposed upgrades could easily be managed via standard engineering design at the time of subdivision consent to provide the required access to the development.

108. Access to Woodend township, as well as north and south via SH1 can be achieved via the existing established routes, namely:

- McQuillan Ave (northbound/SH1),
- Parsonage Rd (Woodend and southbound/SH1).
- An additional link to Woodend township (southeast quadrant) and alternative link southbound/SH1 is available via Stopforth St and Eders Rd or Gladstone Rd.

109. A concept roading plan is included in **Appendix H.**

OUTLINE DEVELOPMENT PLAN (ODP)

110. The above evidence has been prepared based on the Outline Development Plan (ODP) prepared by Align Group, as presented in Victoria Edmond's evidence.

CONCLUSION

111. The above evidence demonstrates that the requested change to the site zoning as proposed in Victoria Edmonds Evidence is not inhibited by the ability to serve the site with the required civil services.

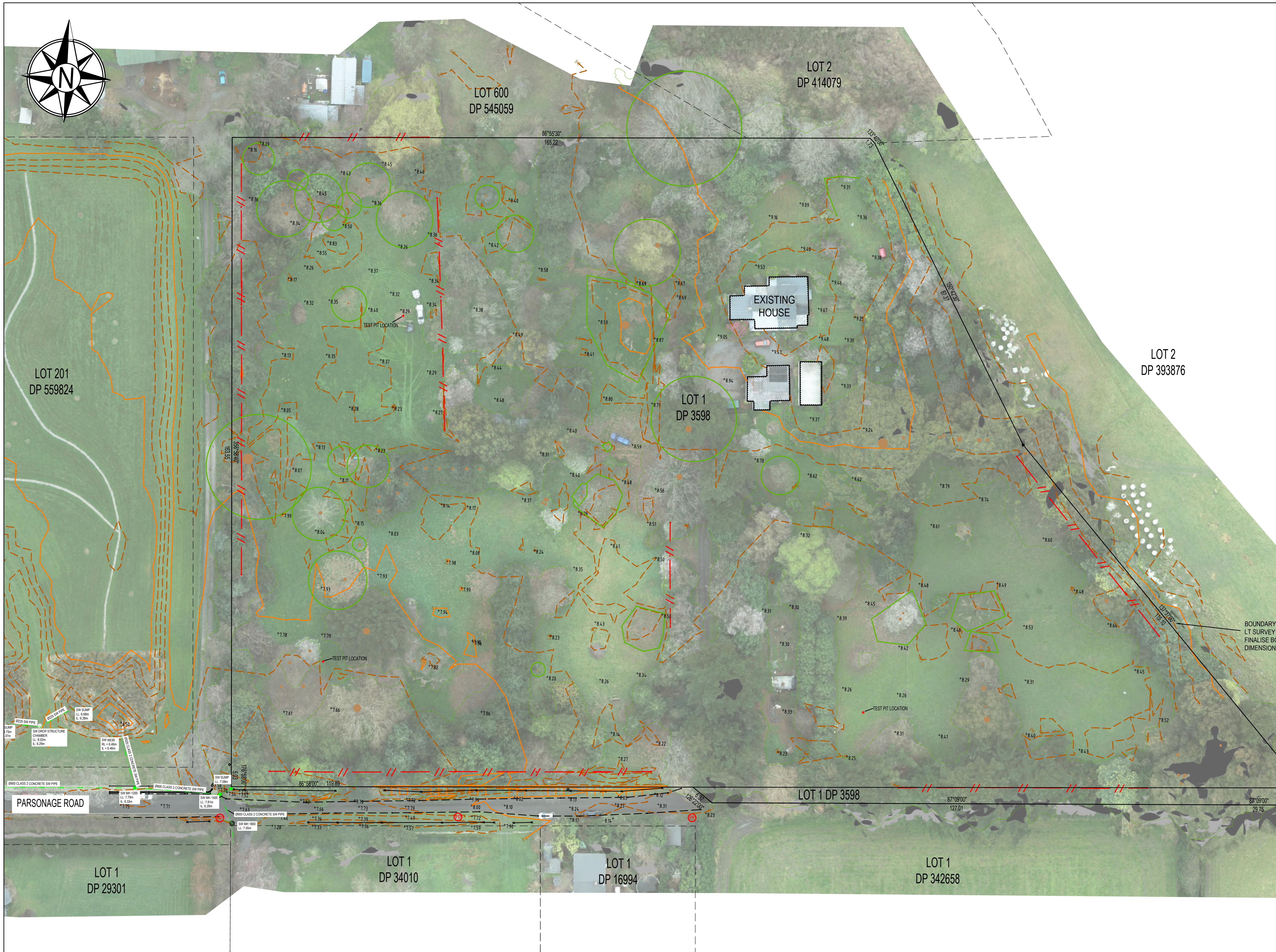
APPENDICES

Appendix A – Graham Surveying Topographic Survey Plan
Appendix B – Stormwater Technical Report
Appendix C – Soakage test pit photos
Appendix D – Stormwater Concept Plan
Appendix E – HIRDS V4 data
Appendix F – Wastewater Concept Plan
Appendix G – Water supply/fire hydrant concept plan
Appendix H – Roading Concept Plan

JAMES HOPKINS
4 March 2024

APPENDIX A

Graham Surveying Topographical Survey Plan



NOTES:

1. COORDINATES ARE IN TERMS OF NZGD 2000 MOUNT PLEASANT CIRCUIT 2000. ORIGIN OF COORDINATES:IT II DP 39896 (EMGD) 395054.550mE, 829944.178mN
2. LEVELS ARE IN TERMS OF NZVD 2016 ORIGIN OF LEVELS V 90 (B4AL) RL 7.2701m
3. DRAINAGE STRUCTURES HAVE BEEN MEASURED ON SITE AND CHECKED AGAINST COUNCIL GIS DATA. PIPE ALIGNMENT IS INDICATIVE ONLY. TRUE ALIGNMENT ON SITE MAY DIFFER FROM ALIGNMENT SHOWN.
4. BOUNDARIES HAVE BEEN CALCULATED FROM LOCAL CADASTRAL MARKS. A BOUNDARY DEFINITION SURVEY SHOULD BE CARRIED OUT TO ESTABLISH EXACT BOUNDARY POSITIONS ON SITE.
5. THE AERIAL WAS SURVEYED FROM A UAV ON THE 5TH OF SEPTEMBER 2023.

LEGEND:

- EXISTING BOUNDARY
- CONTOURS (109.0)
- BUILDING
- TOP OF BANK
- INVERT OF DITCH
- FENCE
- FOOTPATH
- EDGE OF SEAL
- KERB
- SPOT HEIGHT
- CESSPIT
- STORMWATER MANHOLE
- STORMWATER PIPE
- POWER POLE
- TELECOM PLINTH
- TREE (TRUNK, DRIPLINE)
- OLD SURVEY PEG

Rev	Date	Description	Dwn.	Chkd.	Appd.
A	13/09/23	DRAWING ISSUED	NT	RG	RG

GRAHAM SURVEYING

© Graham Surveying Limited

This drawing has been prepared by Graham Surveying Limited solely for the benefit and use by our client in accordance with the terms of our engagement and client instructions. Graham Surveying Limited does not and shall not assume any responsibility or liability whatsoever to any third party arising out of any use or reliance by any third party on the content of this drawing.

	Name	Date
Drawn	NT	08/09/2023
Surveyed	NT + AC	05/09/2023 - 06/09/2023
Checked	RG	13/09/2023
Approved	RG	13/09/2023

Client: RAINER AND URSULA HACK

Project: 110 PARSONAGE ROAD, WOODEND, WAIMAKARIRI

Title: TOPOGRAPHICAL SURVEY

Status: AS SURVEYED

Horizontal Datum: MOUNT PLEASANT 2000

Vertical Datum: NZVD 2016

Scale: 1:1000m @ A3

Size: A3

Drawing Number: GSL23156-SU-1000

Revision: A

NOTES:

1. COORDINATES ARE IN TERMS OF NZGD 2000 MOUNT PLEASANT CIRCUIT 2000. ORIGIN OF COORDINATES:IT II DP 39896 (EMGD) 395054.550mE, 829944.178mN
2. LEVELS ARE IN TERMS OF NZVD 2016 ORIGIN OF LEVELS V 90 (B4AL) RL 7.2701m
3. DRAINAGE STRUCTURES HAVE BEEN MEASURED ON SITE AND CHECKED AGAINST COUNCIL GIS DATA. PIPE ALIGNMENT IS INDICATIVE ONLY. TRUE ALIGNMENT ON SITE MAY DIFFER FROM ALIGNMENT SHOWN.
4. BOUNDARIES HAVE BEEN CALCULATED FROM LOCAL CADASTRAL MARKS. A BOUNDARY DEFINITION SURVEY SHOULD BE CARRIED OUT TO ESTABLISH EXACT BOUNDARY POSITIONS ON SITE.
5. THE AERIAL WAS SURVEYED FROM A UAV ON THE 5TH OF SEPTEMBER 2023.

LEGEND:

EXISTING BOUNDARY
CONTOURS 109.0
BUILDING
TOP OF BANK
INVERT OF DITCH
FENCE
FOOTPATH
EDGE OF SEAL
KERB
SPOT HEIGHT
CESSPIT
STORMWATER MANHOLE
STORMWATER PIPE
POWER POLE
TELECOM PLINTH
TREE (TRUNK, DRIPLINE)
OLD SURVEY PEG

Rev	Date	Description	Dwn.	Chkd.	Appd.
A	13/09/23	DRAWING ISSUED	NT	RG	RG

GRAHAM SURVEYING

© Graham Surveying Limited

This drawing has been prepared by Graham Surveying Limited solely for the benefit and use by our client in accordance with the terms of our engagement and client instructions. Graham Surveying Limited does not and shall not assume any responsibility or liability whatsoever to any third party arising out of any use or reliance by any third party on the content of this drawing.

Drawn: NT	Date: 08/09/2023	
Surveyed: NT + AC	Date: 05/09/2023 - 06/09/2023	
Checked: RG	Date: 13/09/2023	
Approved: RG	Date: 13/09/2023	

Client: RAINER AND URSULA HACK

Project: 110 PARSONAGE ROAD, WOODEND, WAIMAKARIRI

Title: TOPOGRAPHICAL SURVEY

Status: AS SURVEYED

Horizontal Datum: MOUNT PLEASANT 2000	Vertical Datum: NZVD 2016
Scale: 1:1000m @ A3	Size: A3
Drawing Number: GSL23156-SU-1001	Revision: A

APPENDIX B

Stormwater Technical Report

(September 2023)

PHELPS CIVIL

STORMWATER REPORT

110 Parsonage Road - Plan Change and Subdivision

Project Summary

Site Address:	110 Parsonage Road, Woodend
Site Legal Description:	Lot 1 DP Held in CT CB453/268
Site Area:	~3.7 Ha
Consent References:	ECan - n/a
	Local Council - n/a
	Building - n/a
Client Name:	R & U Hack

Report Summary

Project Ref:	230821
Document Ref:	230821.02
Project Title:	110 Parsonage Road
File Name:	Stormwater Report (Short)
Revision:	2.0
Date:	4 March 2024
Author:	James Hopkins

www.phelpscivil.nz | james@phelpscivil.nz | 022 026 0504 | 37 Terako Terrace, Lyford, RD1, Waiau 7395 | SERVING ALL OF CANTERBURY

1 Executive summary

Please note that this stormwater report has been superseded by more detailed analysis in the Plan Change hearing evidence prepared by James Hopkins.

The proposed rezoning and subdivision development of the 3.7 Ha property at 110 Parsonage Road, Woodend will require comprehensive stormwater management to ensure that the runoff from the site to the downstream stormwater system (in particular the McIntosh Drain) does not exceed development levels for all rain events up to and including the 2% AEP (50 year) 2 hour duration event.

The site has no measurable soakage, so stormwater flow management will need to be provided by storage tanks or ponds with restrictive orifices/weirs. Test pits on site and topographic survey completed by **Graham Survey** indicate that the practical depth limit for ponds will be approximately 1.5 m.

Overall it is estimated that the proposed development will generate a need for approximately 1,712 m³ of retention storage with discharge via a restrictive weir/orifice so that the net peak discharge flow rate from the fully developed site will not exceed the pre-developed flow rate of 8.1 L/s (24 hour, 20% ARP (5 year) event.

The provision of approximately 1,200 m³ of pond storage is possible within the communal recreation area (identified as (5) on the **Align** Site Master Plan. If this volume can be achieved fully then there will be no further need to attenuate runoff from the western portion of the site. If constraints limit the volume this may need to be augmented by tanks or other similar solutions. The existing Site Master Plan does not appear to provide for any additional area in the western portion of the site for pond style storage. If the preference is for pond storage (rather than tanks) then additional area should be provided in the Site Master Plan.

The provision of approximately 341 m³ of pond based storage, in conjunction with roofwater tank based attenuation (capable of restricting roof runoff to a maximum of 0.31 L/s). If roofwater tank based attenuation is not provided then approximately 451 m³ of storage will be required. There is approximately 1,300m² of greenspace either side of the access to the existing dwelling and new low density lots which appears to be adequate to provide all the required attenuation storage for the eastern portion of the site.

Wastewater, water supply and roading do not provide any issues that cannot be easily managed at the detailed design (subdivision) stage, so these are not discussed in detail in this report.

2 Introduction

2.1 Site Description

The site is located at 110 Parsonage Road, Woodend and is approximately 3.7 Ha. The site currently has a single dwelling and associated ancillary buildings generally located in the northeastern corner of the site. The remainder of the site is a mixture of pasture and mature trees. The site is very flat, with the only appreciable level differences being between the area around the existing dwellint (typical RL 9.0 - 9.5) with the remainder of the site being very flat, typically between RL 7.6 (southwest corner) and RL 8.5 (the majority of the site. The preliminary site topographic survey, completed by Graham Survey, is included in **Appendix A**.

The site is bound:

- To the west by an access to 100 Parsonage Rd, beyond which lies the recently constructed stormwater ponds for the nearby subdivision.
- To the north by 100 Parsonage Road (rural residential property)
- To the east by 160 Gladstone Road (rural property, evidently used for agricultural purposes
- To the south by Parsonage Road (western half of southern frontage only) and by 124 Gladstone Road (rural property, evidently used for agricultural (cropping) purposes.

Figure 1 - Site

(Note boundary shown is approximate)

2.2 Proposed Development Summary

It is proposed that the site will be included in the upcoming Waimakariri Plan Change to be rezoned for residential development. If the rezoning proceeds it is proposed that the site will be subdivided to generate approximately 31 new lots. This is proposed to be comprised of 5 lower density lots (1,400 – 2,000 m² each), 14 residential lots (520 – 640 m² each) and 12 high density multiunit type lots (280 – 350 m² each, in four blocks of three units). Access to the Residential units and high density units will be via a new road and associated private lanes (ROW). Access to the 5 lower density lots is proposed to be via a new ROW which will also serve the existing dwelling (which will be on a balance lot of approximately 7,400 m^2). While the actual number of lots and configuration may change through the plan change and subdivision processes, the above has been used for the purposes of demonstrating that there is a practicable stormwater solution available for the plan change area.

3 Existing Site Stormwater

3.1 Existing on-site stormwater systems

The site does not currently have any defined stormwater features inside the property boundaries. An existing roadside drain on the north side of Parsonage road drains runoff from the site.

3.2 Downstream storwmater network

The roadside drain in Parsonage Road drains to the west where it enters a sump adjacent to the entrance to number 100 Parsonage Road. This sump discharges into an un-named open drain between numbers 97 and 107 Parsonage Road. This drain flows south, connects with Eders Road drainage, then crosses Gladstone Road and enters the circa 2018 open drainage network through Ranby Place and Fearne Drive. The drain then crosses Petries Road, where it becomes known as McIntosh Drain. The McIntosh Drain flows generally southwards for approximately 7 km where it discharges into the Kaiapoi River, near the confluence with the Waimakariri River.

The McIntosh Drain is stated by WDC as having no spare capacity, accordingly the proposed development must achieve stormwater neutrality. The McIntosh Drain is several kilometres long, very flat, heavily tidally influenced and has a Time of Concentration (Tc) = 2 hours. WDC require that discharges from all developments are mitigated for all events up to and including the 2 hour event.

3.3 Existing site stormwater runoff

The site is best described as generally flat with a mixture of pasture and mature vegetation. Existing stormwater runoff has been estimated utilising 2 methods.

1) NZBC / Rational Method
2) HEC HMS / SCS curve method. PENDING

3.3.a Time of Concentration (Tc)

NZBC E1 VM1 2.3.2 / Figure 1:

($Tc = 100nL^{0.33} / S^{0.2}$) – Where:

n = 0.045 (Average Grassed Surface)

L = 200 m

S = 0.01 (m/m)

Tc = 25 mins (more or less)

3.3.b Rational Method Runoff Estimate

Q = C x i x A x 2.77

Assumptions:

Tc = 30 mins (nearest rainfall figure)

i = 49 mm/hr (NIWA HIRDS V4.0, 2% AEP, 30 min RCP 8.5 (2081-2100)

C = 0.213

A = 3.7 Ha

Q = 107 L/s (at siteTc)

3.3.c Soil Curve Runoff Estimate (HEC HMS)

PENDING – REFER PLAN CHANGE HEARING EVIDENCE

4 Proposed Stormwater Management

4.1 Reduction in Time of Concentration

As the site does not currently have any defined watercourses or channelisation the time of concentration is quite high. Once developed the addition of roof area, hardstand areas, roads and pipes will decrease the time of concentration.

The introduction of stormwater management facilities such as rain gardens and ponds will help extend the time of concentration, but for the purposes of this report it has been assumed that the reduced Tc for the developed site will not be directly mitigated (extended) in any way.

In reality any change in the time of concentration is irrelevant in terms of stormwater management for this site as discharges into the receiving stormwater network is required to be attenuated for all events up to and including the 2% AEP 24 hour event.

4.1.a Time of Concentration (Tc) (DEVELOPED)

NZBC E1 VM1 2.3.1 / Figure 2:

(Tc = Te + Tf) – Where:

Te = 10 minutes (residential with 36 – 50% coverage)

Tf = 1.8 min (gutter flow) + 100 m * 0.9 m/s = 90 s = 1.5 min (pipe flow)

Tc = 12.3 min

By interpolation of the NIWA HIRDs data the rainfall intensity associated with the Rc for the developed site (12.3 mins) is 84 mm/hr (vs 49 mm/hr undeveloped)

4.2 Increased Runoff

4.2.a Peak flow rate Q (L/s)

With the reduction in Tc and corresponding increase in rainfall intensity for the reduced Tc the runoff from the site has been conservatively estimated utilising the rational method to increase from approximately 107 L/s to approximately 380 L/s. This makes it clear that there will need to be some form of attenuation, given the downstream receiving environment (McIntosh Drain) has been clearly documented as having no additional capacity.

Despite this marked increase in peak flow rate the critical factor in designing the stormwater management system for this site will be longer duration events. Any stormwater management (attenuation) system capable of managing longer duration events will significantly over attenuate the shorter duration events. Since the critical duration for the receiving environment is the 24 hour event, the target peak discharge flow rate will be approximately **8.1 L/s (TBC depending on WDC advice re McIntosh Drain critical duration).**

4.2.b Retention Volume

In order to ensure that the site achieves flow neutrality for all duration events up to and including the 24 hour event the proposed stormwater attenuation system will need to be developed to ensure it has enough volumetric capacity to avoid any increase in flow rate up to (and including) the 24 hour event.

To provide a conservative estimation of the required attenuation volume the site runoff for the developed site was compared to the predevelopment runoff for the 24 hour 2% AEP event (8.1L/s). This identified an estimated retention volume of 1,781 m³ (for the 24 hour 2% AEP event).

4.3 Attenuation Options

4.3.a Discharge to Ground (Soakage)

Testpits advanced to approximately 1.8m on site revealed sandy SILTS, which when tested for soakage were found to not provide any appreciable soakage (approximately 30mm in the first hour, which decreased markedly in the second half hour). Thus soakage to ground cannot be relied on to provide stormwater flow/volume management.

4.3.b Rainwater Tanks with restrictive outlets

The utilisation of rainwater tanks with a restrictive outlet can reduce the peak flow rate from a dwelling.

An option is to utilise rainwater tanks in a split re-use and attenuation mode. With a controlled outlet at approximately half depth resulting in the top half of the tank providing attenuation, with the bottom half storing water for non-potable re-use on site.

It is estimated that a single 20,000 rainwater tank with a 12mm restrictive orifice would provide 10 m³ or storage with a peak outflow rate of 0.31 L/s (which could be achieved by a 12mm orifice with 1.5m of storage head above the orifice). This would be able to serve a dwelling of 120 m² roof area. A pair of interconnected tanks with only having a single outlet (or larger single tank), would be able to serve larger dwellings. As the required storage head above the orifice is 1.5m a typical tank of 3.0 m depth could also be used to retain some roof water for irrigation purposes.

It should be noted however that the inclusion of roofwater tanks as an attenuation device is adopted their performance will have to be carefully included in any stormwater modelling and the exact parameters of the tank installed and its operation will need to be protected by a consent notice on the new title.

4.3.c Permable surfaces

Given the identified very low soakage in the location the use of any permeable pavements is not recommended, and even if used will not likely provide any appreciable benefit to stormwater management.

4.3.d Raingardens

Raingardens are a common way of providing stormwater treatment (particularly for trafficable hardstand areas). Careful design of these raingardens will mean that they operate in conjunction with the attenuation ponds, and the raingardens can provide some of the required attenuation volumes.

At the time of detailed design (subdivision consent) the raingardens will be designed, and their volume included in the calculations for the overall site retention volume/discharge.

4.3.e Retention Ponds

The most practical and lowest cost method of providing flow attenuation is the construction of stormwater ponds with a restrictive orifice outlet. The biggest cost associated with this method is the land area required to construct the pond.

It is anticipated that these could be constructed to an approximate depth of 1.5m below existing ground (the actual depth will likely be governed by the invert of the McIntosh Drain at the discharge point, which has been surveyed to RL 6.5, and it is anticipated that the developed site will be finished at or above RL 8.5). The proposed greenspace area is 1,727 m², but that area has been indicated as containing some trees which are to be retained. This may limit the available, space and any specific tree requirements are needed to be known before this method is adopted.

All stormwater management ponds require a bank slope of 1:4 (1:5 preferred and better for any recreation use and mowability). In addition to this a minimum width around all sides that are accessible for maintenance – usually at least 5 metres (this will need to be confirmed by WDC). Factoring these in the maximum area at the top of the bank will be 1,220 m². Assuming a 1:4 side slope and a 1.5 m depth the approximate pond volume is 1,200 m³ (approximately 500 m³ less than the total required). Therefore, there is not enough space in the proposed green space to manage all the stormwater in one location (at a depth of 1.5 m). Additional space will need to be provided, or another means of retention adopted to augment the performance of the main retention pond. If the available working depth of the retention pond is less than 1.5 m the required area will increase further, necessitating more augmentation.

It should also be noted that it is prudent to ensure that additional land is allowed for stormwater management over and above preliminary calculations. This is so that the development of a working stormwater solution is not constrained by a lack of land area. Ensuring that the indicated density and the provision of suitable space in the Site Master Plan is critical to ensuring this occurs. It is suggested that a minimum 10% buffer is provided to any areas identified as being need for ponds.

4.4 Low Density Area Stormwater Management

The eastern portion of the proposed development is shown as including 5 new low density lots (as a transition to the rural zone to the east). It is proposed that this portion of the site, including the existing dwelling, could be managed separately to the standard and high density development on the western portion of the site, thus sharing the attenuation volume and space requirements more evenly across the site.

Calculations indicate that the attenuation volume required will be approximately 450 m³ with a peak outflow of 3.7 L/s.

www.phelpscivil.nz | james@phelpscivil.nz | 022 026 0504 | 37 Terako Terrace, Lyford, RD1, Waiau 7395 | SERVING ALL OF CANTERBURY

The use of roofwater tanks to provide attenuation could result in the adequate management off roof water from the new dwellings, with only the hardstand areas needing to be managed in a pond. If roofwater from each of the 5 dwellings was entirely managed via a tank with restrictive orifice (each achieving a peak outflow of 0.31 L/s – refer to 4.3b above) then the required pond volume would reduce to approximately 341 m^3. The generous landscaping space either side of the ROW/Access to the eastern lots provides in excess of 1300 m^2 or potential storage space. This would mean that a shallow stormwater retention zone of, say, 0.5m depth would utilise approximately half of this area. The utilisation of this area as ponding area would not prevent the planting of trees, provided any trees were tolerant of having their root zone submerged in water for approximately 24-48 hours.

Some suggested locations for attenuation are shown on the plan in **Appendix X.**

4.5 Residential and High Density Area Stormwater Management

The western portion of the proposed development, comprising of 14 residential lots and 12 high density units, as well as the majority of the required roading and ROW will generate the majority of the increased runoff.

Calculations indicate that the volume required for the western portion of the site separate to the east side will be approximately 1,152 m^3, with a peak outflow of 4.2 L/s. As the available pond volume in the reserve is 1,200 m^3 there might not need to be any supplementary means of attenuation for the western portion of the site. However given the recommendation that there is over provision of areas suitable for stormwater management it is recommended that either more area, or additional measures are allowed for.

The berm area indicated on the eastern side of the road could be utilised for raingardens which will provide both treatment and attenuation, however the need for access to each lot across this will significantly reduce the useable area and increase the cost. If it was possible it is suggested that the parking is moved to the eastern side of the road, thus allowing a continuous raingarden along the western side of the road.

5 Wastewater

Waimakariri District Council have indicated in pre-application meeting minutes that there is some capacity available in the recently constructed wastewater pump station, approximately 250m to the west of the site. Assuming the pump station is at a reasonable depth it is considered most likely that the site can be served via a gravity network discharging into the pump station (or into one of the existing pipes that already discharge into the pump station).

6 Water

Waimakariri District Council has indicated in pre-application meeting minutes that the existing water reticulation in Parsonage Road can be extended to serve the site. While an upgrade to the existing WDC network is required WDC have already programmed this upgrade to mitigate head losses, and this is due to be constructed in 2024, which will likely precede the first dwelling being completed and any additional demand being actually generated by the proposed development.

7 Roading/Access

Some upgrading to the parsonage road carriageway will be required form the end of the existing upgraded road (at the entrance to 100 Parsonage Road). It is anticipated that the upgraded formation will match the recent upgrades to the west.

New roading and ROW access has been indicated on the Align “Site Master Plan” and this can easily be designed in detail at the subdivision stage.

www.phelpscivil.nz | james@phelpscivil.nz | 022 026 0504 | 37 Terako Terrace, Lyford, RD1, Waiau 7395 | SERVING ALL OF CANTERBURY

APPENDIX C

Site Soakage Test Photos

APPENDIX D

Stormwater Concept Plan

PHELPS CIVIL
022 026 0504
james@phelpscivil.nz

Stormwater Management Concept

Scale (@ A3) 1:1000

230821

110 Parsonage Road, Woodend - Plan Change

230821 - SM01-B

Plotted by: James Hopkins on 28-Feb-24 at 10:02. Filename:230821 - 110 Parsonage Road.dwg

APPENDIX E

HIRDS V4.0 data

HIRDS V4 Intensity-Duration-Frequency Results
Sitename: woodend
Coordinate system: WGS84
Longitude: 172.6774
Latitude: -43.3196

DDF Mode Parameters	c	d	e	f	g	h	i	
Values:	-0.0082	0.591768	-0.00267	-0.0065	0.342784	-0.01574	2.189054	
Example:	Duration (h	ARI (yrs)	x	y	Rainfall Rate (mm/hr)			
	24	100	3.178054	4.600149	5.929931			

Rainfall intensities (mm/hr) :: Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	19.1	14	11.8	8.92	6.7	4.1	2.88	1.93	1.21	0.892	0.706	0.583
2	0.5	21.7	16	13.4	10.1	7.57	4.62	3.24	2.16	1.35	0.997	0.788	0.65
5	0.2	31.5	23	19.3	14.4	10.7	6.48	4.51	2.99	1.86	1.37	1.08	0.887
10	0.1	39.4	28.6	24	17.8	13.2	7.93	5.5	3.63	2.25	1.65	1.3	1.07
20	0.05	48.1	34.8	29.1	21.5	15.9	9.46	6.54	4.3	2.66	1.94	1.52	1.25
30	0.033	53.5	38.6	32.2	23.8	17.5	10.4	7.17	4.7	2.9	2.11	1.66	1.36
40	0.025	57.4	41.4	34.5	25.4	18.7	11.1	7.62	4.99	3.07	2.24	1.75	1.44
50	0.02	60.6	43.6	36.3	26.8	19.7	11.6	7.98	5.22	3.21	2.33	1.83	1.5
60	0.017	63.3	45.5	37.9	27.9	20.4	12.1	8.27	5.41	3.32	2.41	1.89	1.55
80	0.013	67.5	48.5	40.3	29.6	21.7	12.8	8.74	5.7	3.49	2.54	1.99	1.63
100	0.01	70.9	50.8	42.2	31	22.7	13.3	9.1	5.93	3.63	2.63	2.06	1.69
250	0.004	85	60.7	50.2	36.7	26.7	15.5	10.6	6.85	4.17	3.02	2.36	1.93

HISTORICAL + 16%

	Duration	10	20	30	60	120	360	720	1440	2880	4320	5760	7200
ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	22.16	16.24	13.69	10.35	7.77	4.76	3.34	2.24	1.40	1.03	0.82	0.68
2	0.5	25.17	18.56	15.54	11.72	8.78	5.36	3.76	2.51	1.57	1.16	0.91	0.75
5	0.2	36.54	26.68	22.39	16.70	12.41	7.52	5.23	3.47	2.16	1.59	1.25	1.03
10	0.1	45.70	33.18	27.84	20.65	15.31	9.20	6.38	4.21	2.61	1.91	1.51	1.24
20	0.05	55.80	40.37	33.76	24.94	18.44	10.97	7.59	4.99	3.09	2.25	1.76	1.45
30	0.033	62.06	44.78	37.35	27.61	20.30	12.06	8.32	5.45	3.36	2.45	1.93	1.58
40	0.025	66.58	48.02	40.02	29.46	21.69	12.88	8.84	5.79	3.56	2.60	2.03	1.67
50	0.02	70.30	50.58	42.108	31.09	22.85	13.46	9.26	6.06	3.72	2.70	2.12	1.74
60	0.017	73.43	52.78	43.96	32.36	23.66	14.04	9.59	6.28	3.85	2.80	2.19	1.80
80	0.013	78.30	56.26	46.75	34.34	25.17	14.85	10.14	6.61	4.05	2.95	2.31	1.89
100	0.01	82.24	58.93	48.95	35.96	26.33	15.43	10.56	6.88	4.21	3.05	2.39	1.96
250	0.004	98.60	70.41	58.23	42.57	30.97	17.98	12.30	7.95	4.84	3.50	2.74	2.24

Rainfall intensities (mm/hr) :: RCP8.5 for the period 2081-2100

ARI	AEP	10	20	30	60	120	360	720	1440	2880	4320	5760	7200
1.58	0.633	24.9	18.4	15.5	11.7	8.63	5.09	3.47	2.27	1.39	1.01	0.791	0.65
2	0.5	28.6	21	17.7	13.3	9.85	5.78	3.95	2.56	1.57	1.14	0.892	0.731
5	0.2	41.9	30.6	25.7	19.2	14.1	8.23	5.58	3.59	2.19	1.58	1.24	1.01
10	0.1	52.7	38.3	32.1	23.9	17.5	10.1	6.84	4.39	2.66	1.92	1.5	1.22
20	0.05	64.5	46.7	39	28.9	21.1	12.2	8.17	5.21	3.15	2.27	1.77	1.44
30	0.033	71.9	51.9	43.3	32	23.3	13.4	8.98	5.71	3.44	2.48	1.93	1.57
40	0.025	77.3	55.7	46.4	34.2	24.9	14.3	9.57	6.07	3.65	2.63	2.04	1.66
50	0.02	81.7	58.8	49	36.1	**26.2**	15	10	6.35	3.82	2.74	2.13	1.73
60	0.017	85.3	61.3	51.1	37.6	27.3	15.6	10.4	6.59	3.95	2.84	2.2	1.79
80	0.013	91.2	65.5	54.4	40	29	16.5	11	6.95	4.17	2.99	2.32	1.88
100	0.01	95.7	68.6	57	41.8	30.3	17.2	11.5	7.25	4.33	3.1	2.41	1.95
250	0.004	115	81.9	67.9	49.5	35.7	20.1	13.3	8.37	4.98	3.56	2.75	2.23

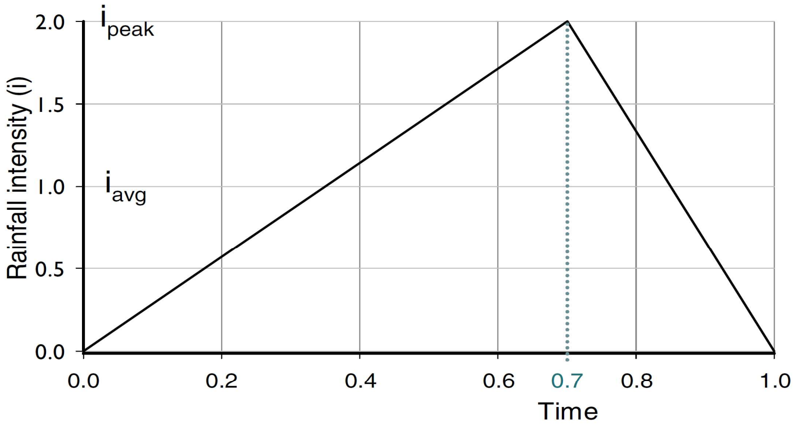


Figure 21-7: A standard dimensionless hyetograph for rainfall intensity.

APPENDIX F

Wastewater Concept Plan

PHELPS CIVIL
022 026 0504
james@phelpscivil.nz

Wastewater Concept

Scale (@ A3) 1:1000

230821

110 Parsonage Road, Woodend - Plan Change

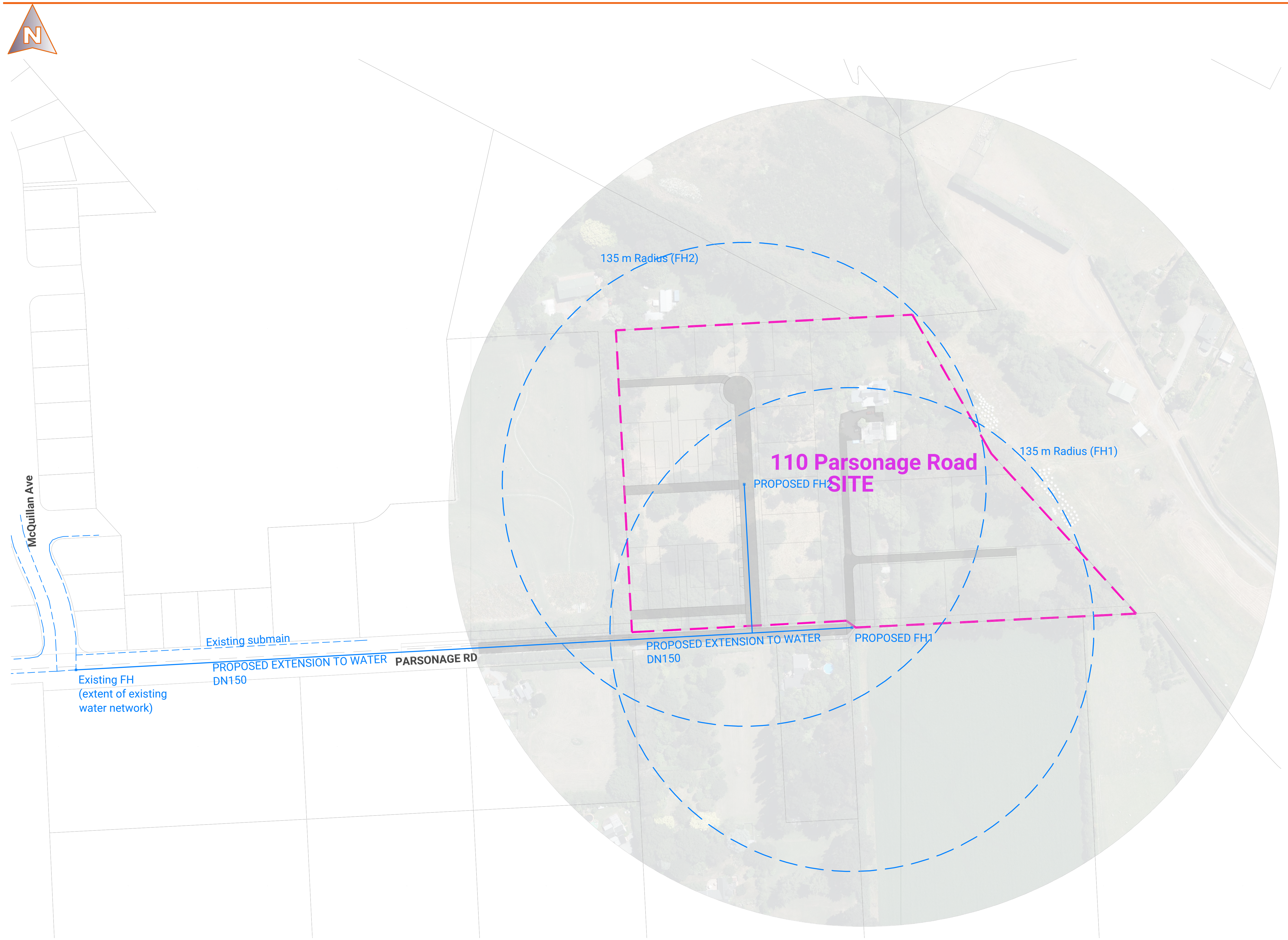
230821 - WW01-B

Plotted by: James Hopkins on 28-Feb-24 at 10:02. Filename:230821 - 110 Parsonage Road.dwg

APPENDIX G

Water supply / fire hydrant Concept Plan

PHELPS CIVIL
022 026 0504
james@phelpscivil.nz



NOTES:

1) This plan is for Plan Change Hearing Evidence Only
2) This plan indicates the means by which services can be extended to proposed development
3) Existing WDC Assets digitised from WDC GIS
4) Proposed water submains and private laterals connections not shown
5) Subject to detailed design and consent
6) Fire Fighting capacity in existing network to be confirmed at time of design

Potable Water And Firefighting

Scale (@ A3) 1:1000

230821 | 110 Parsonage Road, Woodend - Plan Change | 230821 - WP01-B

Plotted by: James Hopkins on 28-Feb-24 at 10:02. Filename:230821 - 110 Parsonage Road.dwg

APPENDIX H

Roading Concept Plan

PHELPS
CIVIL
022 026 0504
james@phelpscivil.nz

Roading Concept

Scale (@ A3) 1:1000

230821	110 Parsonage Road, Woodend - Plan Change	230821 - RD01-B

Plotted by: James Hopkins on 28-Feb-24 at 10:02. Filename:230821 - 110 Parsonage Road.dwg