Appendix F:

Infrastructure Report

Patterson Pitts Group

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NEW ZEALAND CHERRY CORP (LEYSER) L.P.

REQUEST FOR A CHANGE TO THE OPERATIVE CENTRAL OTAGO DISTRICT PLAN

INFRASTRUCTURE REPORT

"SHANNON FARM"

PROJECT:	New Zealand Cherry Corp, 144 Ripponvale Road, Cromwell, Request for a Change to the Operative Central Otago District Plan
PRINCIPAL:	New Zealand Cherry Corp (Leyser) L.P.
OUR REF:	C2528
DATE:	May 2019

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REVISION / APPROVAL PANEL

Rev:	Date:	Prepared By:	Reviewed By:	Comments:
0	13/12/18	PLD	MG	Issued for client review
1	25/2/19	PLD	BG	Client reviewed
2	20/5/19	PLD	CB	Rural Lifestyle references and areas changed

Job No: C2528
Date: 21 May 2019
Report Prepared For: New Zealand Cherry Corp

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CONTENTS

1. Scope

Paterson Pitts Limited Partnership (PPLP) has been engaged by New Zealand Cherry Corp (Leyser) L.P. (NZ Cherry Corp) to provide an infrastructure report to support a private plan change request for a Rural Residential development at 144 Ripponvale Road, Cromwell. The private plan change seeks to re-zone approximately 130 ha of a 243ha site for a Rural Residential development.

Up to 160 dwelling units are planned, ranging from 0.2ha to 3 ha minimum lot size.

This report covers the availability of the following infrastructure elements:

- Wastewater
- Water Supply – Potable and Firefighting
- Network Utility Services (electricity and telecommunications)
- Stormwater disposal from hardstand and roading. Please refer to the Geosolve Flood Hazard Report for the overall management of stormwater discharge off the site.

This report is to be read in conjunction with the Geotechnical and Flood Hazard Reports prepared by Geosolve Ltd in support of the plan change request and with the proposed structure plan for the development.

2. Executive Summary

2.1 Stormwater

Soakage tests and test pitting show the site is subject to highly variable sub soil conditions and permeabilities (soakage rates). However, the large size of the allotments planned means that the normal methods of stormwater disposal for a rural subdivision will be satisfactory. i.e.

- Road side drains (water tables) and grassed swales discharging to rock sump soak pits and/or natural drainage paths.
- Household roof and hardstand runoff will discharge to ground within each allotment by a variety of methods using Low Impact Urban Design and Development (LIUDD) principles.

2.2 Wastewater

It is proposed to connect the development to the Cromwell wastewater reticulation. Computer modelling of the Cromwell Wastewater reticulation by Mott MacDonald NZ Ltd shows that the Development will not have a detrimental effect on the existing network.

2.3 Water Supply

It is proposed to connect the development to the Cromwell water reticulation. Computer modelling of the Cromwell water reticulation by Mott MacDonald NZ Ltd shows that the development will not have a detrimental effect on the existing network.

2.4 Network Utility Services

Chorus New Zealand Ltd have confirmed that a suitable telecommunications (fibre) supply can be made available to the proposed development.

Aurora Energy Ltd have confirmed that it can make a power supply available to the development.

3. Stormwater

Test pits and soakage (permeability) tests have been undertaken over the site. See attached location plan, test pit logs and soakage test results.

Generally the site is overlain with good topsoil of depths varying between 0.20 and 0.30m.

Underneath the topsoil layer is usually a silty layer with traces of fine sands. This material generally has reasonable plasticity and can be quite compact when dry.

The foundation material is generally gravel based. This varies from alluvial outwash gravels, fairly permeable in nature, to very silty colluvial gravels with low to very low permeability. In all cases the gravels have low cohesion and would fret considerably when exposed to the elements.

In one case, (test pit 5), silts were found to underly the gravels at a depth of 1.4m.

Test pits generally had a terminal depth of between 2.0m and 2.7m.

Soakage:
Soakage results varied considerably, with infiltration rates beginning between 94mm/hr and 2000mm/hr. The average infiltration rate across the 9 tests was 584m/hr.

HIRDS gives a 1 in 20 year intensity of 56mm/hr (9.3mm depth in 10 minutes) and an 89mm/hr (14.8mm in 10 minutes) for 1 in 100 intensity (1%AEP).

The maximum hardstand (roof, paving, access roads) per lot will be approximately 1000m^2. The area required to dispose of run off for this amount of hardstand in a 1 in 100 year (1% AEP) rainfall event is, on average $89/(584 – 89) \times 1000m^2 = 180m^2$.

Given the highly variable nature of the sub soil permeability, traditional kerb and channelling of roads into mud tanks is not recommended and is also not in keeping with the rural nature of the proposed development.

Stormwater discharge from road carriageways can be disposed of by the usual methods for rural roads i.e. side drains (water tables) and grassed swales discharging into natural drainage paths and/or rural rock sumps.

Low impact urban design and development (LIUDD) principles are proposed for the management of stormwater run-off from servicing the development for access roading and for roof / hardstand / driveways within allotments. The proposed lots are large rural properties (0.2ha to 3.0ha), so there is ample area available for discharge of stormwater to ground entirely within each lot by a variety of methods, or combination thereof:

- Soak pits
- Attenuation using storage tanks with irrigation discharge to garden and lawns
- Discharge to natural drainage paths
- Direct to discharge to ground surface using dripper and soakage lines over the wider property (irrigation).

Total hardstand from roading, driveways, dwellings etc is expected to be approximately 5% of the total area of the site and will only have a marginal effect on overall peak flood flows off the site, which the Geosolve Flood Hazard Report addresses.

4. Wastewater

A Wastewater Assessment has been commissioned from Council's computer network modellers, Mott MacDonald. **See Appendix C.**

This concluded that the downstream pipework reticulation does have sufficient capacity to cope with the wastewater flows from the development.

There are three options outlined in the report to connect the Cromwell reticulation:

- The development discharges into a pump station which pumps into the Cromwell reticulation via the existing 50mm rising main in Ripponvale Road.
- As above, but a new 50mm rising main is constructed down Ripponvale Road.
- A new 150mm gravity only connection is constructed down Ripponvale Road

Within the development itself it is anticipated that there will be a combination of gravity reticulation for the smaller lots on the flatter part of the site within activity area Rural Lifestyle 1 & 2 and a "distributed" pumped supply for the larger lots on the higher parts of the site within activity areas Rural Lifestyle 3,4 and 5.

"Distributed" systems utilize individual household wet wells with macerating pumps, pumping into a small diameter common rising main. Such systems are now very common and enable reticulated sewage to difficult sites, no matter the terrain, slope, environmental sensitivities or complex topography. It is possible that the entire development will be serviced by a "distributed" scheme.

It may also be feasible for some of the larger, more remote lots in activity area Rural Lifestyle 5 to dispose of wastewater on-site, subject to meeting the requirements of AS/NZS1547:2012.

Final design decisions on all the above matters will be resolved at the subsequent subdivision stage.

5. Water Supply

A Water Impact Assessment has been commissioned from Mott MacDonald NZ Ltd, see **Appendix D**. Computer modelling shows that the development can be adequately serviced without adversely affecting the existing Cromwell Town Network reticulation.

The development will need to be connected to the Cromwell reticulation by a new 150mm main along Ripponvale Road. To fully service the development above reduced level 250m above sea level will require further on-pumping to a 90m^3 reservoir located on the upper part of the site. It is anticipated that the smaller lots within activity area Rural Lifestyle 1 & 2 will be serviced to Fire and Emergency New Zealand's (FENZ) SNZ PAS4509:2008 standard requirements. i.e. an "on demand" high pressure fully reticulated service. The larger lots within activity areas Rural Lifestyle 3-5 in the upper parts of the site can be serviced to a rural supply standard with firefighting to FENZ's requirements for a rural dwelling ie. individual 30m^3 reserve storage tanks with FENZ compatible couplings located within 90m of the dwelling, installed by the lot owner at the time of building a dwelling.

Final design decisions on the configuration of the water reticulation within the development will be made at the subsequent subdivision stage.

6. Network Utility Services

6.1 Telecommunications

Chorus New Zealand Ltd have confirmed that a suitable Air Blown Fibre (ABF) reticulation can be supplied to the proposed development. See **Appendix E.**

Individual home owners will also have the alternative option of the cellular network and several long-distance wi-fi providers for their telecommunications and computer media service.

6.2 Power

There are three possible options for a power supply to the development:

- An Aurora Energy Ltd supply with Aurora owning the subdivision infrastructure.
- "An embedded" supply from an alternative provider connected to a dedicated feeder off Aurora's Zone substation or a feed off Aurora's distribution network, with the alternative provider owning the subdivision infrastructure.
- An independent supply from an alternative provider from a Grid Exit Point (GXP) off the Transpower Cromwell substation, with the alternative provider owning the subdivision infrastructure.

Aurora Energy Ltd have confirmed that a supply can be made available from its distribution network with Aurora owning the subdivision infrastructure. Please refer to the attached supply availability letter. **Appendix F.**

7. Conclusion

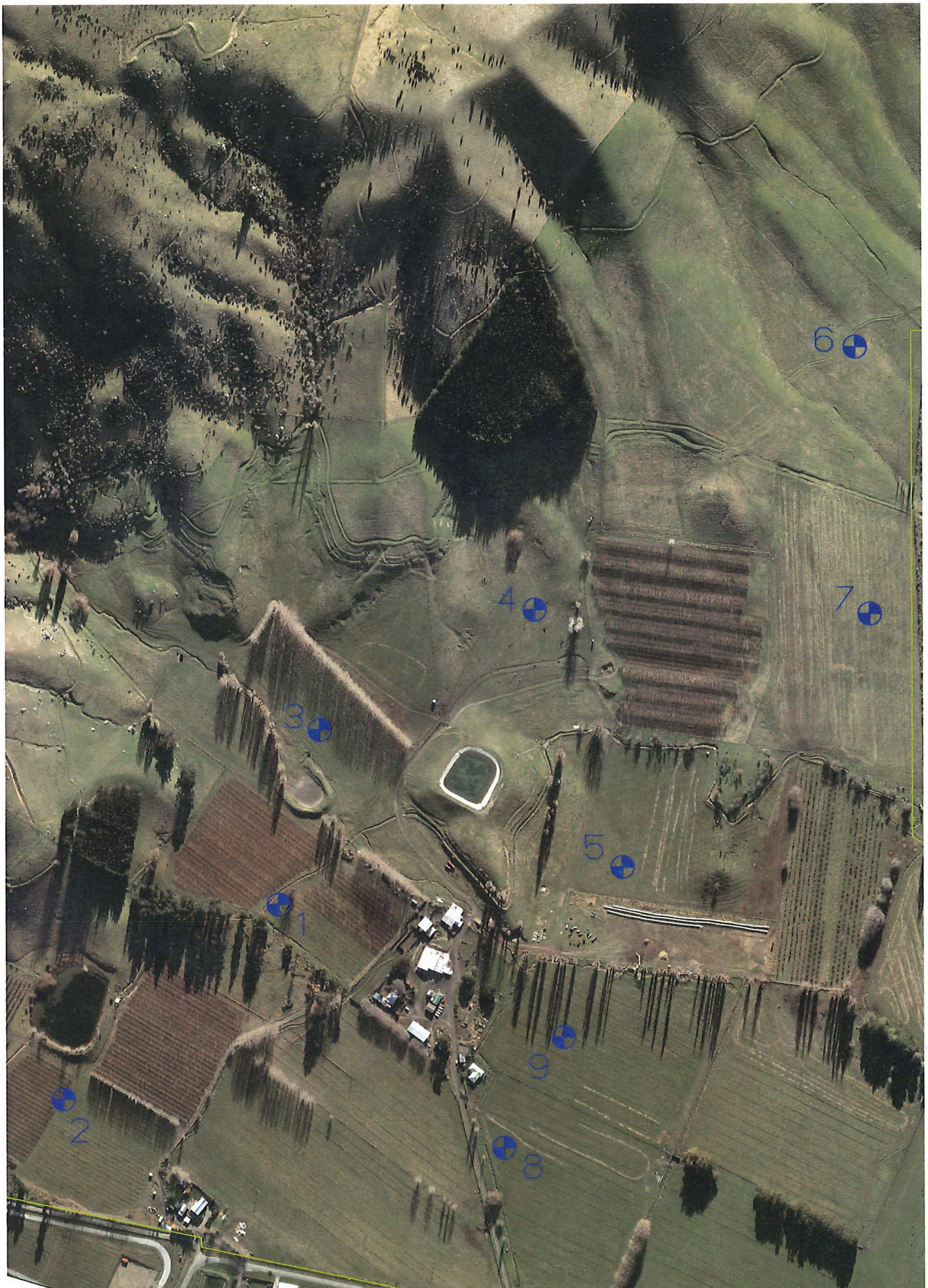
Suitable provision can be made for stormwater disposal from roading and hardstand / roof runoff within allotments and for wastewater, water supply and network utility services to the proposed development.

Trunk water main and wastewater connections to the Cromwell town reticulations will be required to service the development. These will not create any detrimental impact on the existing reticulations.

Peter L Dymock
Principal, B.Sc, Dip Mgt, R.P. Surv, MNZIS, CSNZ
Paterson Pitts Limited Partnership (Cromwell)

APPENDIX A

Location Plan of Test Pits & Test Pit Logs



6
4
7
3
5
1
9
2
8

TEST PIT 1

Ground	0.00	
	Topsoil compact	
	-0.30	
	Compact silts. Mid plasticity	
	-1.00	
	Colluvium silty broken <200mm	
	-2.00	

TEST PIT 2

Ground	0.00	
	Topsoil	
	-0.20	
	Sandy silts compact mid plasticity	
	-0.70	
	Otwash and colluvial gravels compact broken sandy silty <200mm	
	-2.10	

TEST PIT 3

Ground	0.00
Topsoil,	
	-0.30
Fine silts Low plasticity moist compact	
	-1.25
Outwash & colluvial gravels Very Silty <150mm	
	-2.20

TEST PIT 4

Ground 0.00

Topsoil

-0.25

Outwash &
colluvial gravels

Compact
Broken

Dry

free
running

Quite sandy

<200mm

-2.10

TEST PIT 5

Ground 0.00

Topsoil

-0.30

Silty colluvium

moist
compact

-1.00

Silty allivium
compact

-1.40

Silts
compact

moist

med plasticity

-2.10

TEST PIT 6

Ground		
Ground	0.00	
		Topsoil
	-0.30	
		Silts compact
	-0.60	
		silty alluvium
	-1.80	
		gravels. among silts Compact
	-2.10	
		silty gravels
	-2.40	

TEST PIT 7

Ground 0.00

Topsoil

-0.30

Very silty
gravels

compact

Poor binding

Sily lenses

-2.40

TEST PIT 8

Ground	0.00	
	Topsoil	
	-0.25	
	Sandy silts low plasticity	
	-0.60	
	Sandy colluvial gravels with some silt poor cohesion	
	-2.10	

TEST PIT 9

Ground 0.00

Topsoil

-0.25

sandy silts
mid
plasticity

-0.65

sandy
colluvium,
some silt.

poor
cohesion

-2.70

APPENDIX B
Soakage Tests, Infiltration Calculations & Rainfall Intensity Calculations

Pit Dimensions			Area		Test Pit	1			C2528							
Length	2		1.4													
Width	0.7															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.07	240		-0.3		-0.2									
240	0.05	-0.07	180		-0.4		-0.3									
420	0.1	-0.07	300		-0.2		-0.2									
720	0.15	-0.07	420		-0.2		-0.1									
1140	0.2			Average	-0.27		-0.19					Infiltration Rate		632		
		0.28	1140		0.2		0.2	For time period							mm/hr	
												Average across all tests:			5258	
														584	mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		164											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12.0	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		172.2364	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		8.611822	17.223644	two sided	
			3.445		Soakage Rate	l/s					(20m carriageway)					
			3.445		check											
													1 in 20			
											Soakage Capacity		277.8007	m2		
											metres of road		13.89004	27.780071	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	2			C2528							
Length	2		1.4													
Width	0.7															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.07	30		-2.3		-1.7									
30	0.05	-0.07	60		-1.2		-0.8									
90	0.1	-0.07	50		-1.4		-1.0									
140	0.15	-0.07	70		-1.0		-0.7									
210	0.2	-0.07	90		-0.8		-0.6									
300	0.25	-0.07	90		-0.8		-0.6									
390	0.3	-0.07	130		-0.5		-0.4									
520	0.35	-0.07	140		-0.5		-0.4									
660	0.4	-0.07	150		-0.5		-0.3									
810	0.45			Average	-1.00		-0.71					Infiltration Rate		2000		
		0.63	810		0.8		0.6	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		47											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		545.4154	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		27.27077	54.54154	two sided	
			10.908		Soakage Rate	l/s					(20m carriageway)					
			10.908		check											
													1 in 20			
											Soakage Capacity		879.7022	m2		
											metres of road		43.98511	87.97022	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	3			C2528							
Length	2		1.4													
Width	0.7															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.07	120		-0.6		-0.4									
120	0.05	-0.07	90		-0.8		-0.6									
210	0.1	-0.07	120		-0.6		-0.4									
330	0.15	-0.07	150		-0.5		-0.3									
480	0.2	-0.07	210		-0.3		-0.2									
690	0.25	-0.07	240		-0.3		-0.2									
930	0.3	-0.07	240		-0.3		-0.2									
1170	0.35			Average	-0.48		-0.34					Infiltration Rate		1077		
		0.49	1170		0.4		0.3	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		90											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		293.6852	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		14.68426	29.36852	two sided	
			5.874		Soakage Rate	l/s					(20m carriageway)					
			5.874		check											
													1 in 20			
											Soakage Capacity		473.6858	m2		
											metres of road		23.68429	47.36858	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	4			C2528							
Length	2		1.4													
Width	0.7				Sides collapsed in making further soakage analysis unreliable.											
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.07	120		-0.6		-0.4									
120	0.05	-0.07	210		-0.3		-0.2									
330	0.1			Average	-0.46		-0.33					Infiltration Rate		1091		
		0.14	330		0.4		0.3	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		89											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		297.4993	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		14.87497	29.74993	two sided	
			5.950		Soakage Rate	l/s					(20m carriageway)					
			5.950		check											
													1 in 20			
											Soakage Capacity		479.8376	m2		
											metres of road		23.99188	47.98376	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	5			C2528							
Length	2		1.4													
Width	0.7															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		$l/s/m^2$									
0	0	-0.028	120		-0.2		-0.2									
120	0.02	-0.07	810		-0.1		-0.1									
930	0.07	-0.028	600		0.0		0.0									
1530	0.09			Average	-0.12		-0.09					Infiltration Rate		212		
		0.126	1530		0.1		0.1	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		725											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		57.74986	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		2.887493	5.774986	two sided	
			1.155		Soakage Rate	l/s					(20m carriageway)					
			1.155		check											
													1 in 20			
											Soakage Capacity		93.14494	m2		
											metres of road		4.657247	9.314494	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	6			C2528							
Length	2.8		1.96													
Width	0.7															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.0588	660		-0.1		0.0									
660	0.03	-0.0392	390		-0.1		-0.1									
1050	0.05	-0.0392	600		-0.1		0.0									
1650	0.07			Average	-0.08		-0.04					Infiltration Rate		153		
		0.1372	1650		0.1		0.0	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		1397											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		41.6499	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		2.082495	4.16499	two sided	
			0.833		Soakage Rate	l/s					(20m carriageway)					
			0.833		check											
													1 in 20			
											Soakage Capacity		67.17726	m2		
											metres of road		3.358863	6.717726	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	7			C2528							
Length	2.4		1.68													
Width	0.7															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.0336	420		-0.1		0.0									
420	0.02	-0.0168	360		0.0		0.0									
780	0.03	-0.0168	420		0.0		0.0									
1200	0.04	0	330		0.0		0.0									
1530	0.04			Average	-0.06		-0.03					Infiltration Rate		94		
		0.0672	1530		0.0		0.0	For time period							mm/hr	
	Area to drain 1000m3 hardstand			*(1 in 100yr)*												
	TC = 10 min		Area required		17391											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		25.66661	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		1.28333	2.566661	two sided	
			0.513		Soakage Rate	l/s					(20m carriageway)					
			0.513		check											
													1 in 20			
											Soakage Capacity		41.39775	m2		
											metres of road		2.069888	4.139775	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	8			C2528							
Length	1.9		2.28													
Width	1.2															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.0912	150		-0.6		-0.3									
150	0.04	-0.114	300		-0.4		-0.2									
450	0.09	-0.114	270		-0.4		-0.2									
720	0.14	-0.114	330		-0.3		-0.2									
1050	0.19	-0.114	510		-0.2		-0.1									
1560	0.24	-0.114	660		-0.2		-0.1									
2220	0.29			Average	-0.36		-0.16					Infiltration Rate		470		
		0.6612	2220		0.3		0.1	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		233											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		128.2463	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		6.412316	12.82463	two sided	
			2.565		Soakage Rate	l/s					(20m carriageway)					
			2.565		check											
													1 in 20			
											Soakage Capacity		206.8489	m2		
											metres of road		10.34245	20.68489	two sided	
											(20m carriageway)					

Pit Dimensions			Area		Test Pit	9			C2528							
Length	2.1		2.52													
Width	1.2															
Time (s)	Depth	dVolume	dTime (s)		Soakage (l/s)		l/s/m²									
0	0	-0.0756	90		-0.8		-0.3									
90	0.03	-0.126	160		-0.8		-0.3									
250	0.08	-0.126	230		-0.5		-0.2									
480	0.13	-0.126	270		-0.5		-0.2									
750	0.18	-0.126	300		-0.4		-0.2									
1050	0.23	-0.504	1770		-0.3		-0.1									
1410	0.28	0.7056	-1410		-0.5		-0.2									
1920	0.33	0.8316	-1920		-0.4		-0.2									
2820	0.43			Average	-0.54		-0.21					Infiltration Rate		549		
		1.0836	2820		0.4		0.2	For time period							mm/hr	
	Area to drain 1000m3 hardstand			(1 in 100yr)												
	TC = 10 min		Area required		194											
			(takes into account incident rain)													
														total volume		
	1 in 100 (RCP6.0 2081 - 2100))										area m2	1000		12	m3	
	14.8	mm in 10 minutes		89mm/hr		Q=2.78CiA					runoff	0.8		rate per second		
						A = Q/2.78iC		0.060626	ha		depth	0.015		20	l/s	
	1 in 20 (RCP6.0 2081 - 2100))							606.2566			seconds	600				
	9.3	mm in 10 minutes		56mm/hr		0.096351	ha					1 in 100	runoff per m2 per s		0.02	l/s/m2
						919.3538						1 in 20	runoff per m2 per s		0.012	l/s/m2
													1 in 100			
Soakpit Base =			0.785398	m2							Soakage Capacity		149.6991	m2		
Effective soakage @ 2m deep			19.63495	m2	45 deg angle influence						metres of road		7.484956	14.96991	two sided	
			2.994		Soakage Rate	l/s					(20m carriageway)					
			2.994		check											
													1 in 20			
											Soakage Capacity		241.4502	m2		
											metres of road		12.07251	24.14502	two sided	
											(20m carriageway)					

APPENDIX C

Wastewater Impact Assessment

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NZ Cherry Ripponvale – Development Impact Assessment

10 September 2018

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This letter summarises the results of the assessment undertaken for a proposed development consisting of 160 residential units on 112 Ripponvale Rd, on the west side of the Cromwell water network.

1 Background

Mott MacDonald was commissioned by Central Otago District Council (CODC) to assess the system performance in terms of Level of Service (LOS) and firefighting capacity in the proposed development.

In this analysis, the latest Cromwell water supply model was used. The existing scenario was investigated with additional demand from the proposed development. The project location is shown in blue in Figure 1-1 below.

Figure 1-1: Proposed Development Location

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2 Assumptions

2.1 Demand Calculations

The demand for the development has been calculated based on the CODC addendum to NZS4404-2004, considering the following:

- 160 residentials units
- Daily consumption of 500 l/person/day
- Peak hour factor of 5
- Density: 3 persons per dwelling in residential areas.

Based on these assumptions, the following demand has been considered in the proposed development:

Table 2-1: Demand Calculation

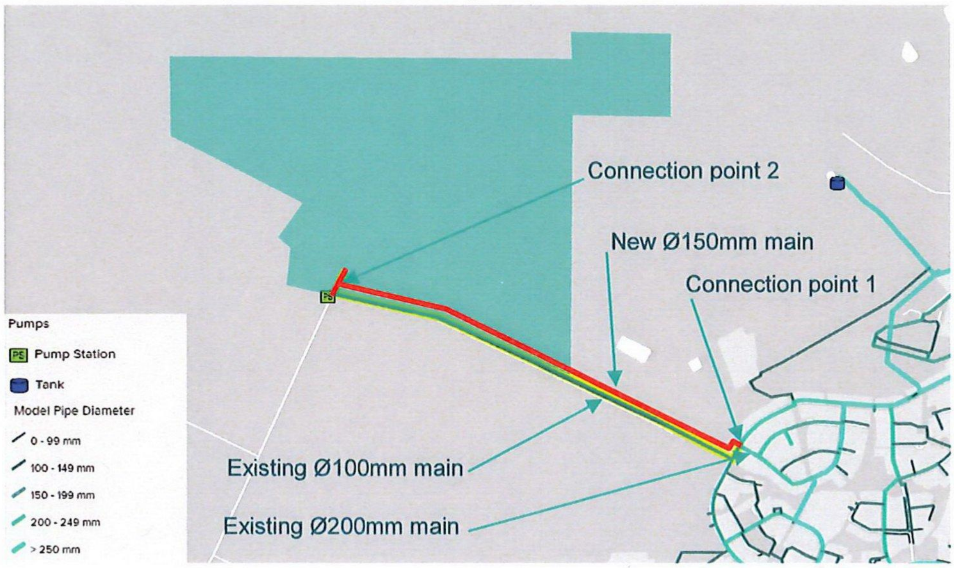
	Demand (l/s)
Average Daily Flow (l/s)	2.8
Instantaneous Peak Flow (l/s)	13.9

2.2 Proposed Connection Points

The development's elevation ranges between 225m and 440m. It was assumed that the development would be connected, via a new 150mm ID main, to the existing 200mm pipe along Waenga Drive, as the existing 100 mm main along Ripponvale Rd is too small to provide adequate fire supply (connection point 1). It was assumed that the new 150mm pipe would be connected at the end of the 100mm pipe (connection point 2) to improve conveyance and system performance.

Figure 2-1 below shows the proposed connection points and the new 1.8 km of 150mm watermain (in red) parallel to the existing 100mm pipe.

Figure 2-1: Development Location, Proposed Connection Points and Network



3 Scenarios Investigated

The scenarios investigated were based on the Cromwell base scenario (existing peak day - 13.1 MLD), including consented development in the area (namely McNulty Rd developments). The level of service achieved in the proposed residential development were assessed in terms of pressure, head loss and fire flow. The impact of the proposed development was verified in terms of pressure and head losses in the remaining of the network.

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Fire flow was based on the NZ Fire Service Code of Practice (SNZ PAS 4509:2008). FW2 classification (residential requirements) has been tested for the development zone based on 25 l/s at 2/3 of the peak demand.

4 Model Results

Results have been analysed to check that LOS (minimum pressure and maximum head losses) can be met in the proposed development. LOS were verified for the minimum and maximum elevation, and the maximum ground level that can be serviced from the existing network was identified.

4.1 System Performance in the Proposed Development

Table 4-1 below summarises the minimum and maximum pressure, the maximum head loss as well as the fire flow capacities forecasted at the minimum and maximum elevations in the development.

Table 4-1: Forecasted System Performance in the Development

	Pressure		Head Losses	Fire Flow
	Minimum	Maximum	Maximum	
Development min elevation (225m)	58.9	65.6	2.3	Can meet residential fire flow (FW2 –25 l/s with 10m residual pressure)
Development max elevation (440m)	0	0	--	Cannot meet residential fire flow (FW2 –25 l/s with 10m residual pressure)

The normal operating pressure and maximum head loss set by the NZS4404:2004 Standard (Development ad Subdivision Engineering Standards) are respectively 30 to 90m and 5m/km. As shown in the table above, minimum pressure and maximum head loss in the proposed development is within the recommended LOS for the development minimum elevation of 225m. For the maximum development elevation LOS are not met.

FW2 fire flow was tested at the end of the proposed 150mm line. The model predicts that the fire flow (FW2 – 25 l/s) can be provided with enough residual pressure (40m) at the development minimum elevation of 225m but not at the maximum elevation level of 440m.

4.2 Maximum Serviceable Elevation

The model predicted that the maximum ground level that can be serviced while providing sufficient LOS is 250m RL. Table 4-2 below summarises the minimum and maximum pressure and fire flow capacities forecasted at 250m RL in the development. To allow for additional local head losses and potential model inaccuracy, a residual pressure of 15m was considered for fire flow instead of the required 10m. It was assumed that the development internal network would include a 150mm loop to provide residential fire flow.

Table 4-2: Forecasted System Performance at 250m RL

	Pressure		Head Losses	Fire Flow
	Minimum	Maximum	Maximum	
Development at 250m elevation	33.8	40.6	2.3	Can meet residential fire flow (FW2 –25 l/s with 10m residual pressure)

4.3 System Performance Analysis in the Remaining Network

The section below describes the results of the system performance in the remaining of the Cromwell network. Results have been analysed to assess the effect of the proposed development. Figure 4-1 below shows the maximum head loss and minimum pressure conditions for the current peak demand, without the development, while Figure 4-2 shows the forecasted system performance with the development.

Figure 4-1: Current Peak Day System Performance – No Development

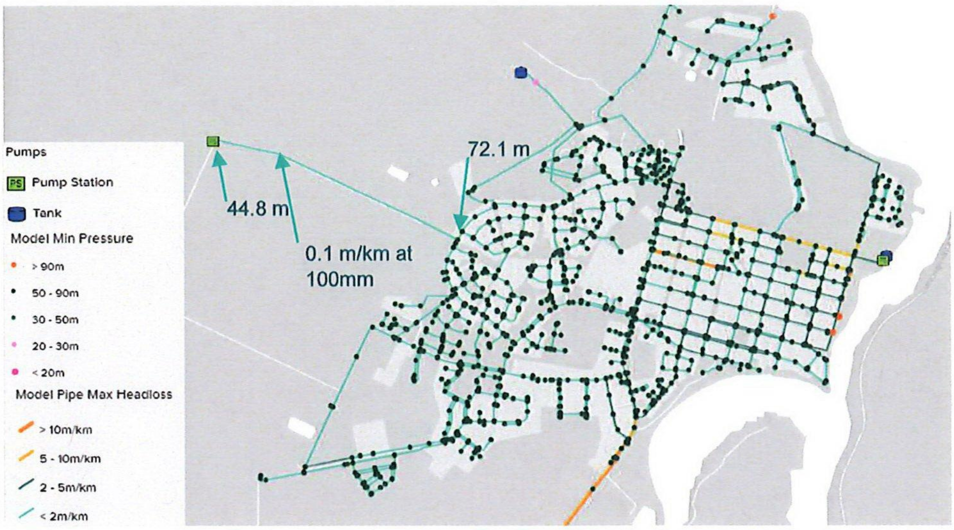
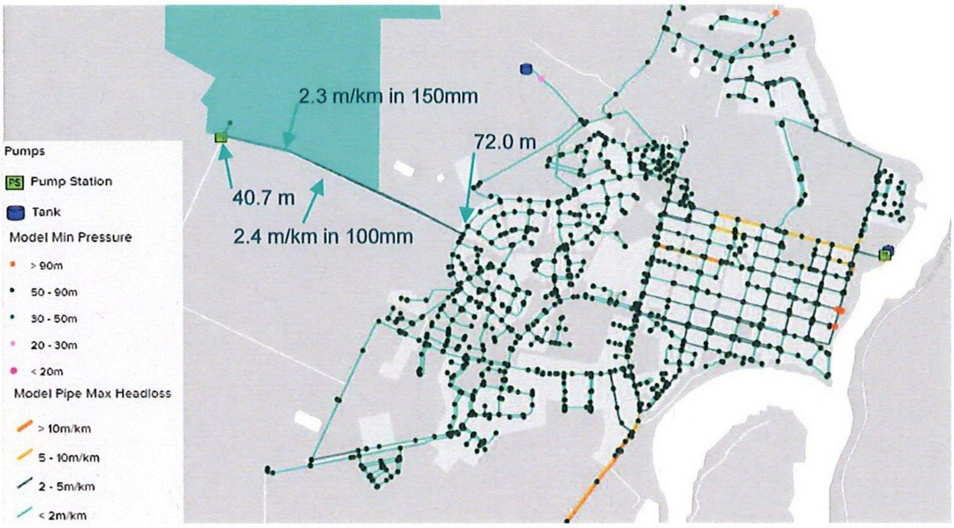


Figure 4-2: Current Peak Day System Performance – With Development



The Table 4-3 below summarises the maximum head losses along Ripponvale Rd and the minimum pressure forecasted at the connection points, before and after the proposed development:

Table 4-3: Forecasted System Performance at the Connection Points

	Minimum Pressure		Maximum Head Losses	
	Connection 1	Connection 2	100mm pipe	150mm pipe
Existing	72.1	44.8	0.1	-
Post Development	72.0	40.7	2.4	2.3
Drop/Increase	-0.1m	-4.1m	+2.3m/km	-

As shown in the Figure 4-1, Figure 4-2 and Table 4-3, the proposed development is predicted to have a negligible impact on most of the remaining of the water network with a maximum pressure drop of 0.1 m forecasted at the connection point 1

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(Waenga Dr). A larger pressure drop (4.1m) is predicted at the connection point 2 (Ripponvale Rd) however LOS remain satisfactory. Pressure is expected to remain above 40 m at the end of the 100 mm pipe along Ripponvale Rd and above 50 m in the remaining of the network. Head losses are predicted to increase to 2.4 m/km in the existing 100 mm pipe along Ripponvale Rd, below the recommended 5m/km. No other significant head loss increase is expected in the remaining of the network.

5 Conclusion and Recommendation

Additional residential demand for the proposed NZ Cherry Ripponvale development (160 lots at 112 Ripponvale Rd) was added to the network for the current peak day model to determine if suitable LOS could be obtained. It was assumed that the development would be connected, via a new 150mm main, to the existing 200mm pipe along Waenga drive.

The system performance at the proposed development site was first assessed. LOS are predicted to be met in terms of pressure, head loss and fire flow (FW2 – 25 l/s) for elevations up to 250m RL.

The system performance in the remaining of the network was also verified. The proposed development is predicted to have a negligible impact on most of the network (pressure drop of 0.1m) with the exception of the connection point 2 (112 Ripponvale Rd) where a pressure drop of 4m is expected. However, pressure is expected to remain above 40 m at the end of the 100 mm pipe along Ripponvale Rd and above 50 m in the remaining of the network. Head losses are not predicted to increase above the recommended LOS.

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Revision	Date	Originator	Checker	Approver	Description
A	10/09/18	Kori Ditmeyer	Julie Plessis	Chhan Chau	Draft for client review

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29th October 2018

1 Background

Mott MacDonald was commissioned by Central Otago District Council (CODC) to undertake a hydraulic modelling analysis to assess the impact of the Cherry Ripponvale development in Cromwell, located on the west side of the Cromwell wastewater network. This memo outlines the assumptions made and findings of this investigation.

The scope of work included the following:

- Update the existing Cromwell wastewater model to include the Cherry Ripponvale development.

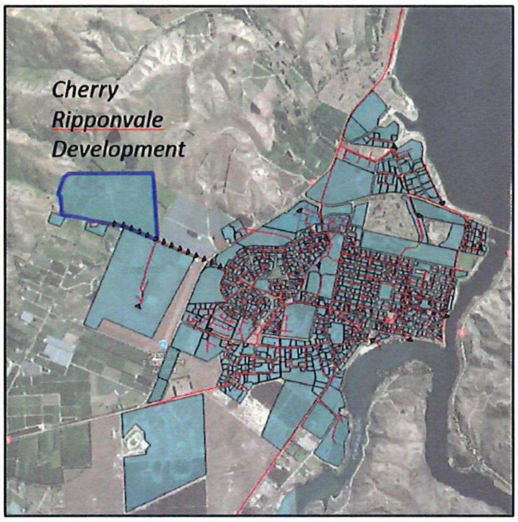

Figure 1: Development Location

- Model the new Pump Station which will service the development
- Estimate the additional wastewater discharge resulting from the development. There are approximately 160 residential lots.
- Simulate the current dry weather and wet weather (10-year ARI storm) scenarios with and without the new development.
- Perform system performance analysis in terms of capacity of the wastewater system to accommodate the proposed development.
- Assess the impact of the new development against the existing network to examine if there are any detrimental effects.
- Carry out option

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- *Report on investigation and results.*

2 Flow Calculation and Routing

Calculation of the wastewater loads were based on the New Zealand Standard for Land Development and Subdivision Infrastructure NZS 4404:2010:

- Daily consumption = 250 L/person/day
- Peaking factor (residential) = 2.5
- Density (residential) = 3 persons per dwelling in residential areas
- Infiltration & inflow scaling factor = 2

The flows are predominantly residential, and no commercial and/or industrial loads are expected from the development. A standard 24-hour diurnal profile having a peak factor of 2.5 was applied to the residential flow as shown in Figure 2. The resulting design peak dry and wet weather flows are summarised in Table 1.

Table 1: Flow Calculation

Parcels	Population	Manhole GIS Asset ID	Peak Wet Weather Flow (L/s)	Average Daily Flow (m^3/d)
160	480	CherryRipponvale _Dummy	6.9	120

The residential flows were applied the following diurnal pattern.

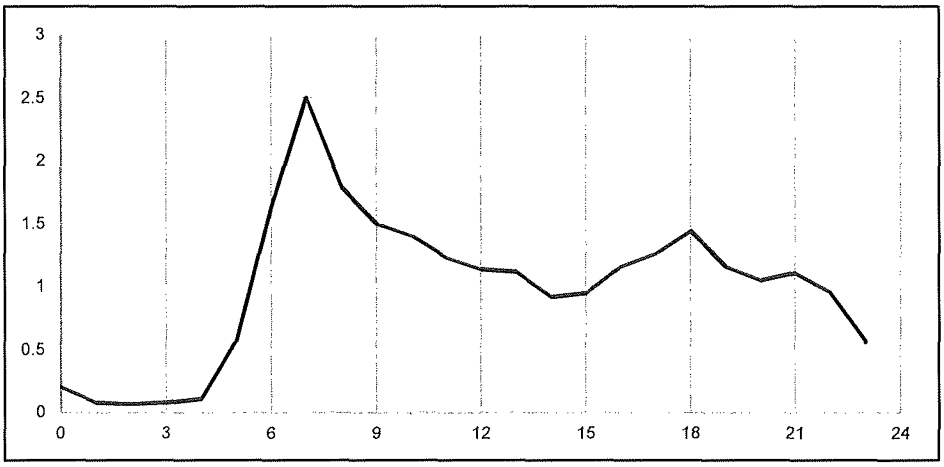

Figure 2: Residential diurnal profile

Three options were considered to connect the proposed development: two pressurised options (options 1a and 1b) and one gravity option (option 2).

Option 1a:

The development discharges into a pump station which pumps the flow to Waenga Street. No details were provided regarding the pump station, but Mott MacDonald believed it was relevant to model a dummy one with assumed characteristics providing the significant distance between the development and the connection point. The following assumptions were made:

- Manhole Diameter = 4m
- Ground Level is equal to closest manhole downstream (node ID 6989999)
- Constant flow = 6.5 l/s
- 500 mm difference between pump start and pump stop levels

Option 1b:

The residential development discharges straight into the existing 50 mm pressurised pipe along Ripponvale Road via a proposed rising main from a proposed pumping station.

The development connection is illustrated in Figure 3.

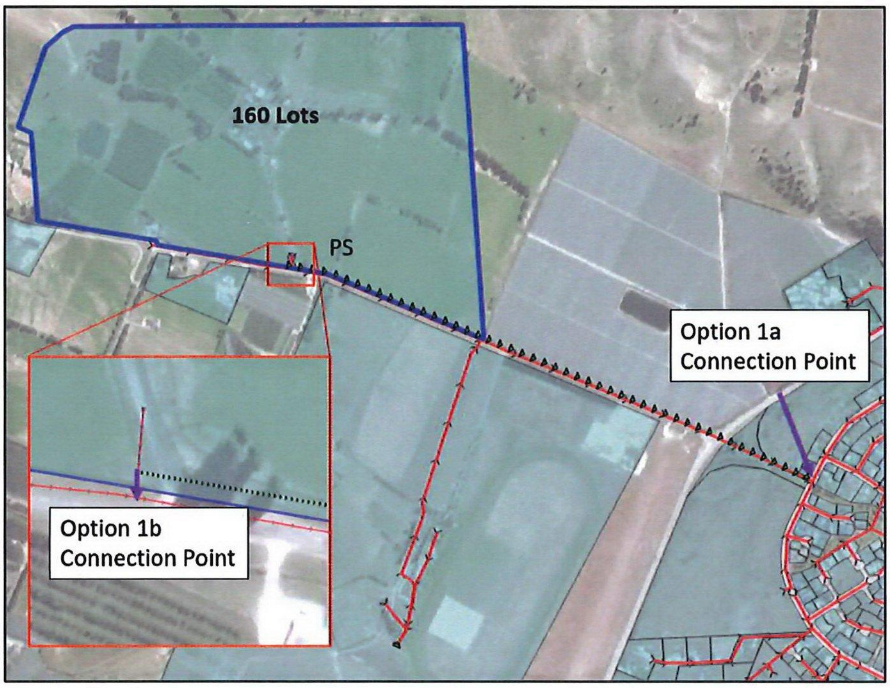

Figure 3: Network Connection – Options 1a and 1b

Option 2:

The 50mm pressurised pipe will be replaced with a new 150mm or a larger size (as the model recommends) gravity pipe. The new gravity sewer will serve all flows served by the existing 50mm pressurised pipe including existing connections and proposed development.

The layout for option 2 is shown in Figure 4 below.

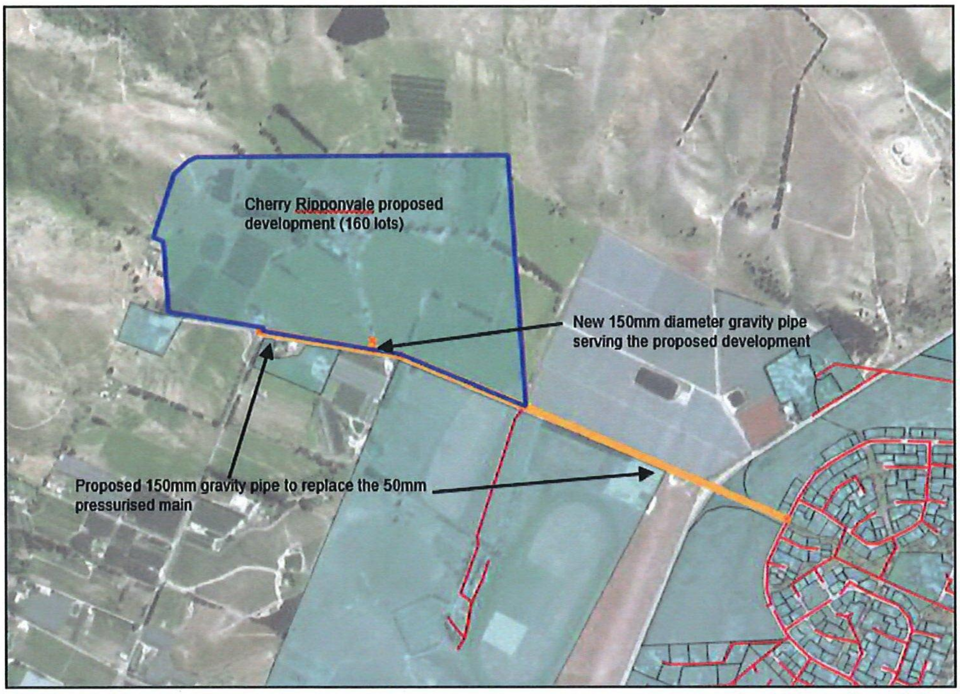

Figure 4: Option 2 layout

The following assumptions have been made for option 2:

1. Upgrade the existing 50mm uPVC pressurised main to a 150mm gravity pipe as shown in Figure 4.
2. New 150mm diameter pipe which will take flow from the proposed development to the upsized 150mm gravity pipe.

3 Scenarios Modelled

The primary objective of the system performance is to assess the wastewater network capacity and overflow occurrences under a few different scenarios as follows:

1. Existing model (Cromwell base scenario)
2. Option 1a (existing + Cherry Ripponvale)
3. Option 2 (existing + Cherry Ripponvale)

Table 2: System Performance Scenarios

Scenario ID	Network Load	Flow Scenarios
Existing	2017 network	DWF
		10-year storm
Option 1a	2017 network	DWF
		10-year storm
Option 2	2017 network	DWF
		10-year storm

4 Pipe Capacity in Dry and Wet Weather

Pipe capacities were evaluated in two ways. Firstly, by comparing the modelled peak flow with the theoretical pipe full capacity (Qmax/Qf) and secondly, by comparing the modelled peak depth with the pipe diameter (H_{max}/Diameter). Peak flows above the theoretical pipe capacity indicate that the pipe is undersized and cannot convey the peak flows that are required through the network.

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Option 1a:

An analysis of option 1a results indicated that the Cherry Ripponvale development caused very little detriment to the overall network capacity under dry and wet weather events. The number of pipes in the network that are surcharged in the model scenarios is presented in Table 3 and Figure 5 below. As illustrated in the graph, there is almost no difference between the pre-development and post-development scenarios with regards to both pipe flow capacity (Qmax/Qf) and pipe filling (Hmax/Diameter). The additional discharge causes a slight increase of levels downstream the connection point which fills 3 additional pipes for the DWF and 7 additional pipes for the WWF. However, it is not concerning in terms of pipe flow capacity since the only additional pipe under capacity for the WWF simulation corresponds to the dummy link located upstream of the Cherry PS.

Table 3: Option1a: Number of surcharged pipes in dry and wet weather

Scenario	No of Pipes Qmax/Qf > 1	%Total	No of Pipes Hmax/Dia > 1	%Total
Dry Weather Flow				
Existing	8	0.6	170	13.5
Existing + Cherry Ripponvale	8	0.6	173	13.8
Wet Weather Flow				
Existing	11	0.9	203	16.2
Existing + Cherry Ripponvale	12	1.0	210	16.6

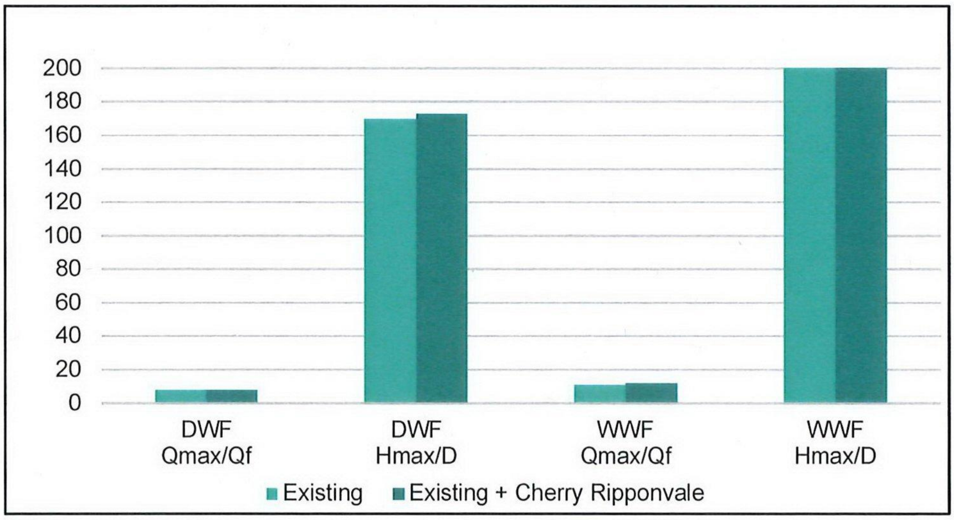

Figure 5: Option 1a Surcharged pipes in dry and wet weather

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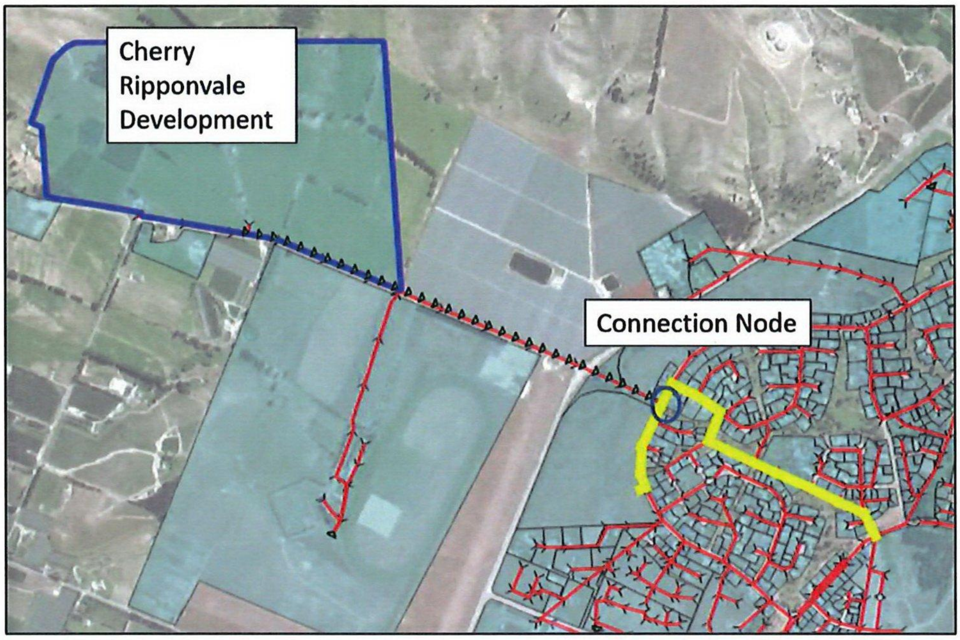

Figure 6: Option 1a Long Profile Location

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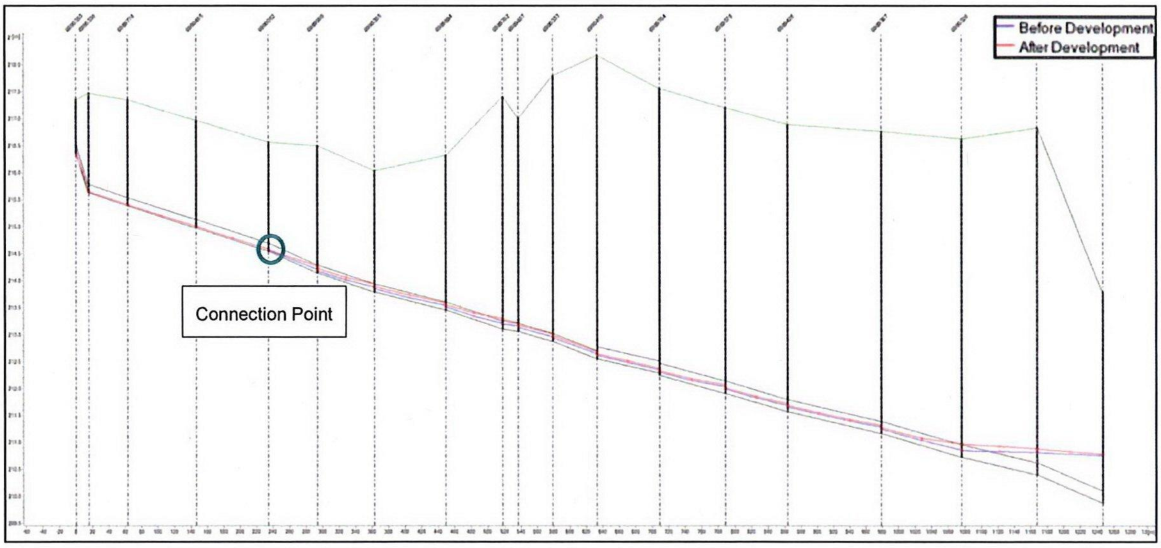

Figure 7: Option 1a - Impact of development along Waenga Road until Antimony Cres intersection (10-year ARI)

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The long section above shows the maximum water level in the pipe from upstream the connection at Waenga road until the Antimony Cres intersection (illustrated in Figure 6) before and after the development. It can be seen that the proposed development has minimal impact on the capacity of the local wastewater network.

Option 2:

- Dry Weather Flow:

The model predicts no issues with the proposed 150mm diameter gravity pipe (refer to Figure 9 and Figure 10).

- Wet Weather Flow:

The model predicts no issues with the proposed 150mm diameter gravity pipe: downstream flow has slightly increased, but the increase is not significant and has no noticeable impact on the downstream system (refer to Figure 12).

Table 4 Result of Option 2: Number of surcharged pipes in dry and wet weather

Scenario	No of Pipes Qmax/Qf > 1	%Total	No of Pipes Hmax/Dia > 1	%Total
Dry Weather Flow				
Existing	8	0.6	170	13.5
Option 1a	8	0.6	173	13.8
Option 2	8	0.6	170	13.5
Wet Weather Flow				
Existing	11	0.9	203	16.2
Option 1a	12	1.0	210	16.6
Option 2	12	1.0	212	16.9

Figure 8 Long Section Profiles

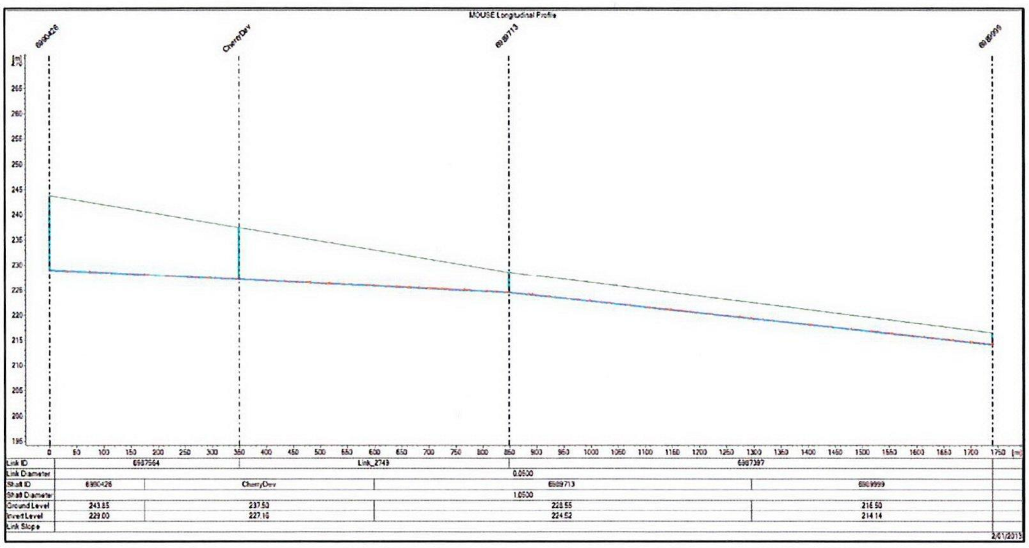

Figure 9 Option 2 - Long Section – DWF - A to B.

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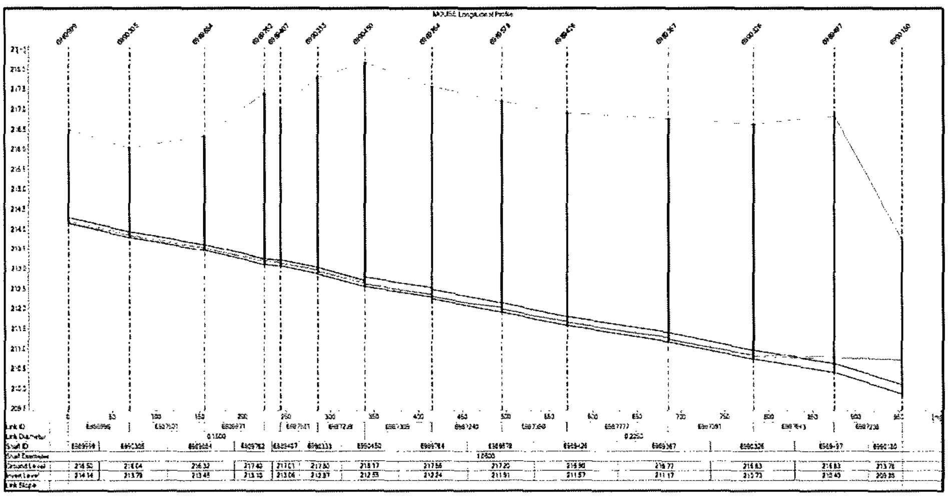

Figure 10 Option 2 - Long Section – DWF - B to C

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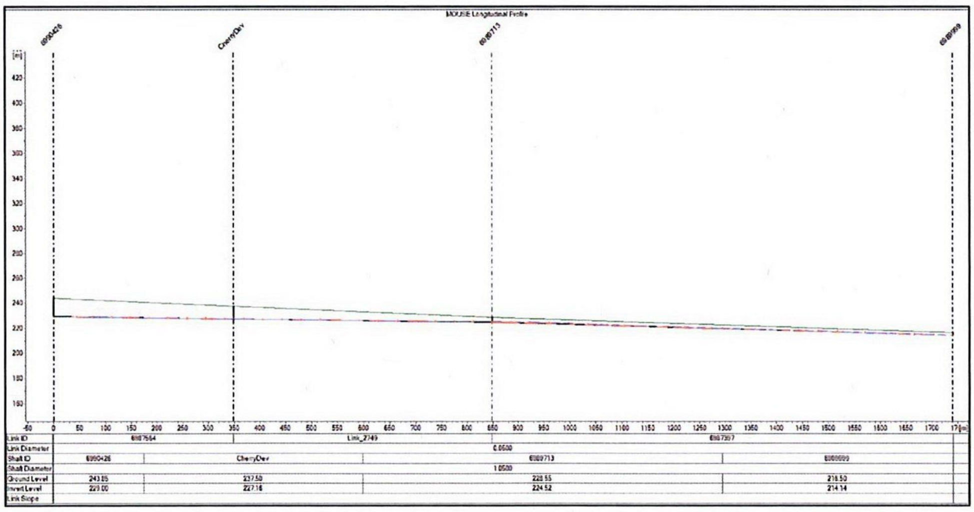

Figure 11 option 2 - Long Section - WWF – A to B

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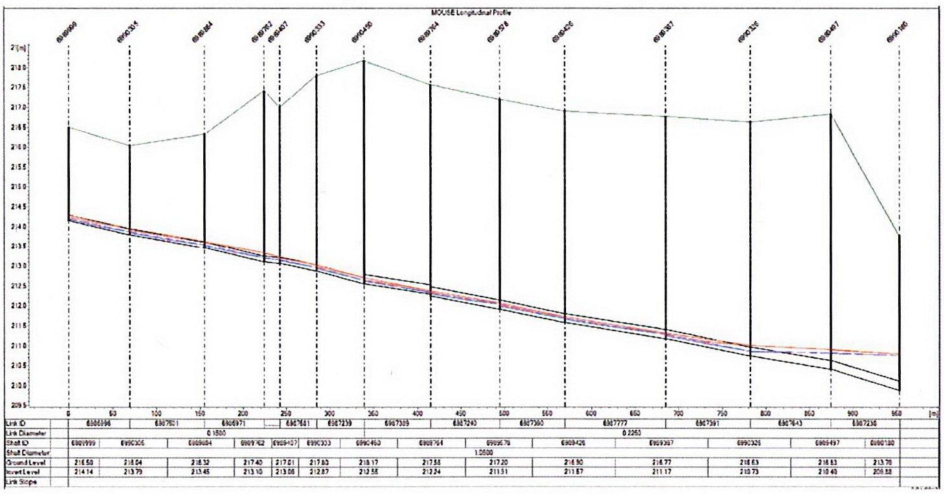

Figure 12 Option 2 - Long Section - WWF - B to C

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5 Overflows

The total number of spill locations for the dry weather and the 10-year ARI storm events are presented in Table 5. There is no additional uncontrolled dry weather and wet weather overflows occurring in the network.

Table 5 Option 1a Number of overflows (10-year ARI)

Scenario	Number of Overflows	Overflow Volume (m^3)
Dry Weather		
Existing	0	0
Existing + Cherry Ripponvale (Option 1a)	0	0
Existing + Cherry Ripponvale (Option 2)	0	0
Wet Weather		
Existing	2	220.4
Existing + Cherry Ripponvale (Option 1a)	2	220.4
Existing + Cherry Ripponvale (Option 2)	2	220.4

6 Conclusions and Recommendation

A high-level system performance assessment was undertaken to analyse the effect of the Cherry Ripponvale development on the network capacity for three different options.

Option 1a

The analysis yielded very similar results between the pre-development and post-development scenarios for both dry weather and wet weather events. Based on this high-level study, it is concluded that the Cherry Ripponvale development is unlikely to have a detrimental effect to the existing network with Option 1a.

Nevertheless, before proceeding with any further work, it is recommended to confirm the validity of the assumptions undertaken regarding the dummy pump station.

Option 1b

This option was briefly investigated and it was noted that the maximum top water level at the 50mm pressurised pipe at manhole 6990426, Ripponvale Road and at the proposed pumping station has increased by approximately 10 to 20m.

The connection from the proposed pumping station to the 50mm pressured main is assumed; proposed local system serving the 160 lots have not been included in the model and proposed pump operational regime has not been considered either.

The current connections from local pumping stations connecting to the existing 50mm pressurised main have not been modelled in detail.

No information was available regarding the proposed pump station and the pump operational regime; therefore it was not possible to undertake a detailed assessment of the pressurised main capacity. There is a possibility that the additional flow may exceed the pipe pressure rating and causes pipe burst, or surge in the existing 50mm pressurised main.

The model predicts no issues in the downstream system if the proposed pumping station is to be discharged to the current 50mm pressurised pipe. However, it is noted that the 50mm pressurised pipe limits the flow discharge to the Waenga Road manhole.

It is recommended that if the proposed 160 lots are to be discharged directly to the current 50mm pressurised main, further investigation should be undertaken regarding the effect of the additional flow on the existing 50mm pressured main. Detailed information would be required such as rising main characteristics, proposed pumping station information including wet well dimensions and pump operational regime. The current local connections to the existing 50mm pressurised main should also be reviewed and updated in the model.

Option 2

The model predicts no issues with option 2, the proposed 150mm gravity sewer replacing the 50mm pressurised main has capacity to convey the dry weather and wet weather flow without causing any detriment to the downstream condition.

Based on this high-level study, it is concluded that option 2 is unlikely to have a detrimental effect to the existing network.

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Revision	Date	Originator	Checker	Approver	Description
A	11/09/2018	Tom Lecomte	Chhan Chau	Julie Plessis	Draft for client review
B	16/10/2018	Chhan Chau	Tom Lecomte	Julie Plessis	Draft for client review
C	19/10/2018	Chhan Chau	Tom Lecomte	Julie Plessis	Draft Final submission
D	29/10/2018	Chhan Chau	Tom Lecomte	Julie Plessis	Final submission

APPENDIX D

Water Impact Assessment

MOTT MACDONALD

Quentin Adams,
Central Otago District Council
1 Dunorling Street
PO Box 122, Alexandra 9340
New Zealand

NZ Cherry Ripponvale – Development Impact Assessment

10 September 2018

Mason Bros. Building
Level 2, 139 Pakenham Street West
Wynyard Quarter
Auckland 1010
PO Box 37525, Parnell, 1151
New Zealand

T +64 (0)9 375 2400
mottmac.com

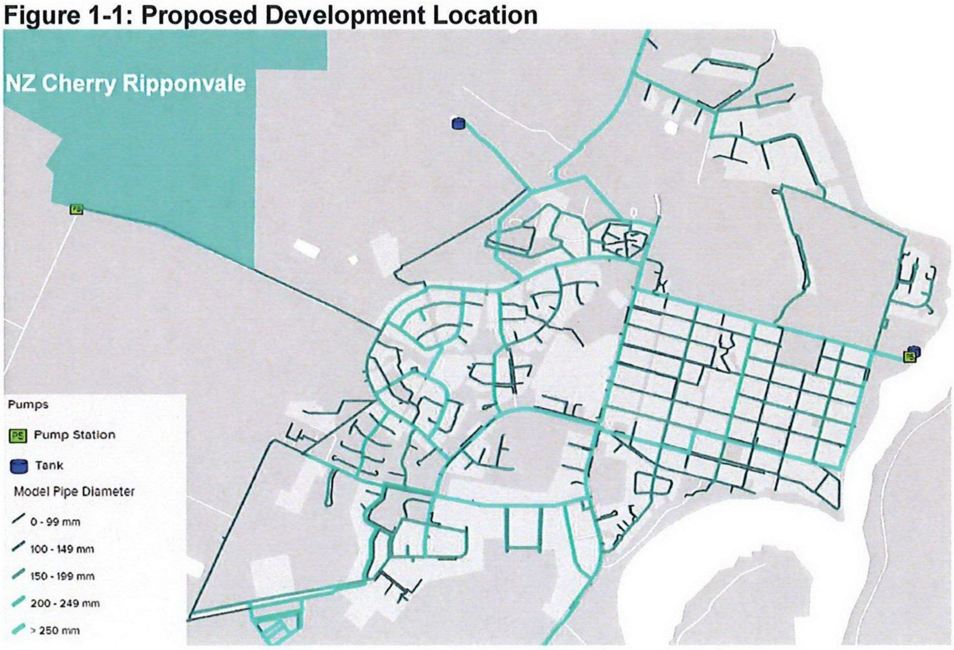
This letter summarises the results of the assessment undertaken for a proposed development consisting of 160 residential units on 112 Ripponvale Rd, on the west side of the Cromwell water network.

1 Background

Mott MacDonald was commissioned by Central Otago District Council (CODC) to assess the system performance in terms of Level of Service (LOS) and firefighting capacity in the proposed development.

In this analysis, the latest Cromwell water supply model was used. The existing scenario was investigated with additional demand from the proposed development. The project location is shown in blue in Figure 1-1 below.

Figure 1-1: Proposed Development Location



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2 Assumptions

2.1 Demand Calculations

The demand for the development has been calculated based on the CODC addendum to NZS4404-2004, considering the following:

- 160 residentials units
- Daily consumption of 500 l/person/day
- Peak hour factor of 5
- Density: 3 persons per dwelling in residential areas.

Based on these assumptions, the following demand has been considered in the proposed development:

Table 2-1: Demand Calculation

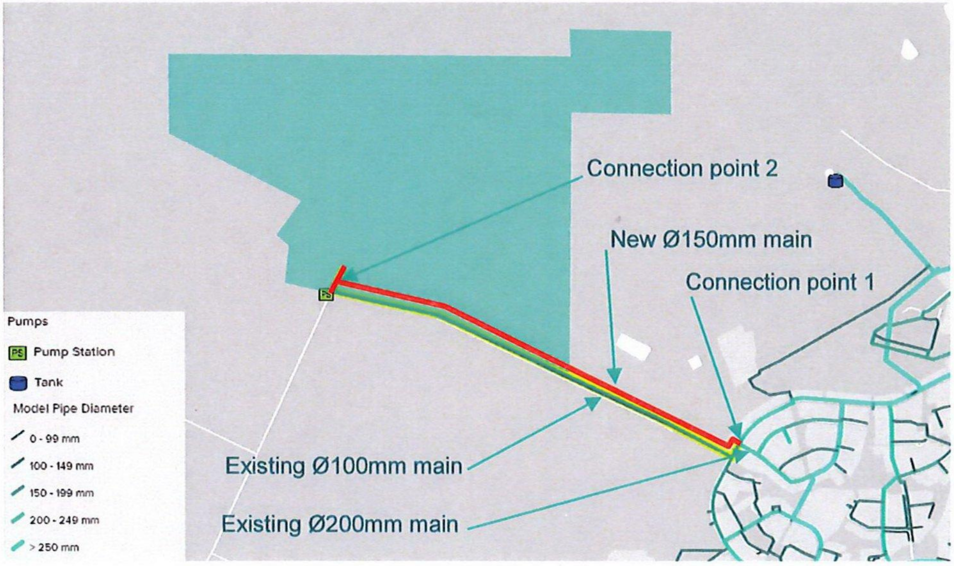
	Demand (l/s)
Average Daily Flow (l/s)	2.8
Instantaneous Peak Flow (l/s)	13.9

2.2 Proposed Connection Points

The development's elevation ranges between 225m and 440m. It was assumed that the development would be connected, via a new 150mm ID main, to the existing 200mm pipe along Waenga Drive, as the existing 100 mm main along Ripponvale Rd is too small to provide adequate fire supply (connection point 1). It was assumed that the new 150mm pipe would be connected at the end of the 100mm pipe (connection point 2) to improve conveyance and system performance.

Figure 2-1 below shows the proposed connection points and the new 1.8 km of 150mm watermain (in red) parallel to the existing 100mm pipe.

Figure 2-1: Development Location, Proposed Connection Points and Network



3 Scenarios Investigated

The scenarios investigated were based on the Cromwell base scenario (existing peak day - 13.1 MLD), including consented development in the area (namely McNulty Rd developments). The level of service achieved in the proposed residential development were assessed in terms of pressure, head loss and fire flow. The impact of the proposed development was verified in terms of pressure and head losses in the remaining of the network.

Fire flow was based on the NZ Fire Service Code of Practice (SNZ PAS 4509:2008). FW2 classification (residential requirements) has been tested for the development zone based on 25 l/s at 2/3 of the peak demand.

4 Model Results

Results have been analysed to check that LOS (minimum pressure and maximum head losses) can be met in the proposed development. LOS were verified for the minimum and maximum elevation, and the maximum ground level that can be serviced from the existing network was identified.

4.1 System Performance in the Proposed Development

Table 4-1 below summarises the minimum and maximum pressure, the maximum head loss as well as the fire flow capacities forecasted at the minimum and maximum elevations in the development.

Table 4-1: Forecasted System Performance in the Development

	Pressure		Head Losses	Fire Flow
	Minimum	Maximum	Maximum	
Development min elevation (225m)	58.9	65.6	2.3	Can meet residential fire flow (FW2 –25 l/s with 10m residual pressure)
Development max elevation (440m)	0	0	--	Cannot meet residential fire flow (FW2 –25 l/s with 10m residual pressure)

The normal operating pressure and maximum head loss set by the NZS4404:2004 Standard (Development ad Subdivision Engineering Standards) are respectively 30 to 90m and 5m/km. As shown in the table above, minimum pressure and maximum head loss in the proposed development is within the recommended LOS for the development minimum elevation of 225m. For the maximum development elevation LOS are not met.

FW2 fire flow was tested at the end of the proposed 150mm line. The model predicts that the fire flow (FW2 – 25 l/s) can be provided with enough residual pressure (40m) at the development minimum elevation of 225m but not at the maximum elevation level of 440m.

4.2 Maximum Serviceable Elevation

The model predicted that the maximum ground level that can be serviced while providing sufficient LOS is 250m RL. Table 4-2 below summarises the minimum and maximum pressure and fire flow capacities forecasted at 250m RL in the development. To allow for additional local head losses and potential model inaccuracy, a residual pressure of 15m was considered for fire flow instead of the required 10m. It was assumed that the development internal network would include a 150mm loop to provide residential fire flow.

Table 4-2: Forecasted System Performance at 250m RL

	Pressure		Head Losses	Fire Flow
	Minimum	*Maximum*	*Maximum*	
Development at 250m elevation	33.8	40.6	2.3	Can meet residential fire flow (FW2 –25 l/s with 10m residual pressure)

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4.3 System Performance Analysis in the Remaining Network

The section below describes the results of the system performance in the remaining of the Cromwell network. Results have been analysed to assess the effect of the proposed development. Figure 4-1 below shows the maximum head loss and minimum pressure conditions for the current peak demand, without the development, while Figure 4-2 shows the forecasted system performance with the development.

Figure 4-1: Current Peak Day System Performance – No Development

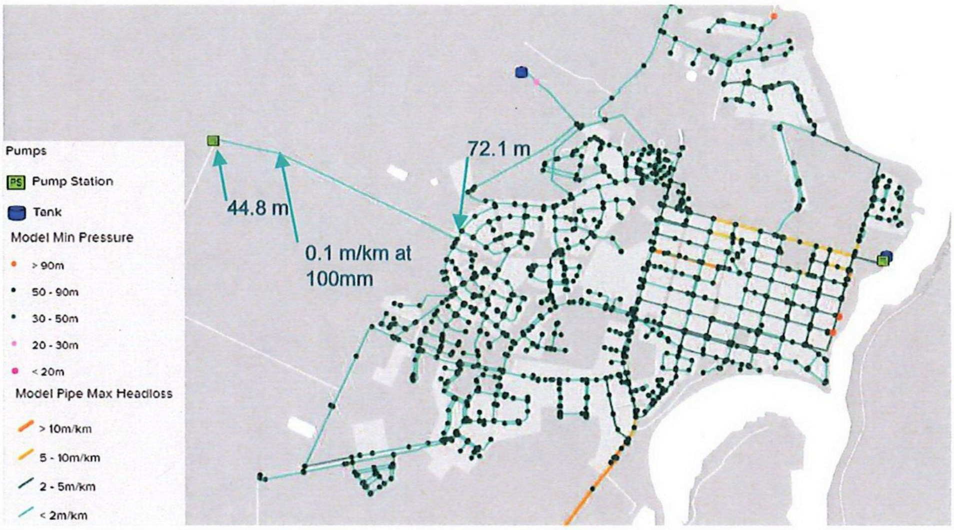
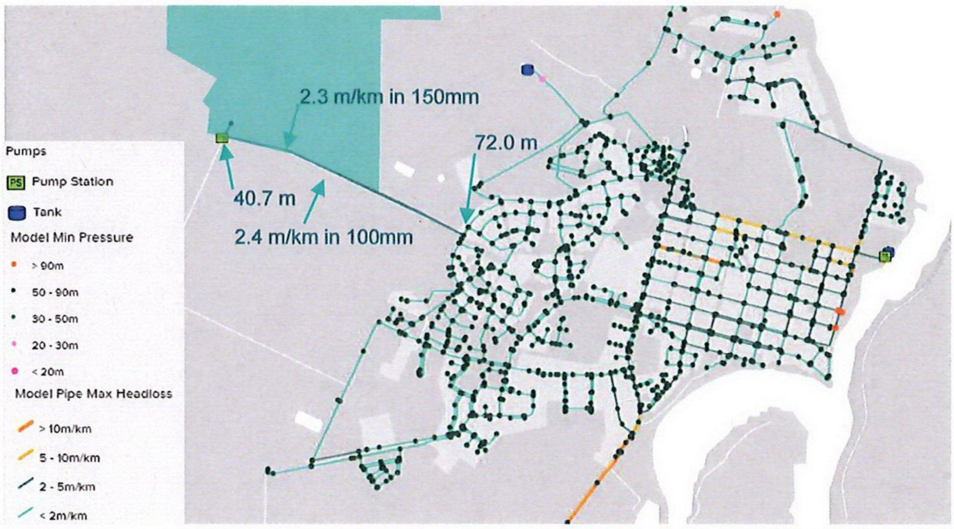


Figure 4-2: Current Peak Day System Performance – With Development



The Table 4-3 below summarises the maximum head losses along Ripponvale Rd and the minimum pressure forecasted at the connection points, before and after the proposed development:

Table 4-3: Forecasted System Performance at the Connection Points

	Minimum Pressure		Maximum Head Losses	
	Connection 1	Connection 2	100mm pipe	150mm pipe
Existing	72.1	44.8	0.1	-
Post Development	72.0	40.7	2.4	2.3
Drop/Increase	-0.1m	-4.1m	+2.3m/km	-

As shown in the Figure 4-1, Figure 4-2 and Table 4-3, the proposed development is predicted to have a negligible impact on most of the remaining of the water network with a maximum pressure drop of 0.1 m forecasted at the connection point 1

(Waenga Dr). A larger pressure drop (4.1m) is predicted at the connection point 2 (Ripponvale Rd) however LOS remain satisfactory. Pressure is expected to remain above 40 m at the end of the 100 mm pipe along Ripponvale Rd and above 50 m in the remaining of the network. Head losses are predicted to increase to 2.4 m/km in the existing 100 mm pipe along Ripponvale Rd, below the recommended 5m/km. No other significant head loss increase is expected in the remaining of the network.

5 Conclusion and Recommendation

Additional residential demand for the proposed NZ Cherry Ripponvale development (160 lots at 112 Ripponvale Rd) was added to the network for the current peak day model to determine if suitable LOS could be obtained. It was assumed that the development would be connected, via a new 150mm main, to the existing 200mm pipe along Waenga drive.

The system performance at the proposed development site was first assessed. LOS are predicted to be met in terms of pressure, head loss and fire flow (FW2 – 25 l/s) for elevations up to 250m RL.

The system performance in the remaining of the network was also verified. The proposed development is predicted to have a negligible impact on most of the network (pressure drop of 0.1m) with the exception of the connection point 2 (112 Ripponvale Rd) where a pressure drop of 4m is expected. However, pressure is expected to remain above 40 m at the end of the 100 mm pipe along Ripponvale Rd and above 50 m in the remaining of the network. Head losses are not predicted to increase above the recommended LOS.

Kori Ditmeyer
Hydraulic Engineer
Kori.Ditmeyer@mottmac.com

Revision	Date	Originator	Checker	Approver	Description
A	10/09/18	Kori Ditmeyer	Julie Plessis	Chhan Chau	Draft for client review

APPENDIX E
Confirmation of Telecom Supply

Chorus Network Services
PO Box 9405
Waikato Mail Centre
Hamilton 3200
Telephone: 0800 782 386
Email: tsg@chorus.co.nz

27 November 2018

Chorus Ref #: CMW49464
Your Ref #:

NZ Cherry Corp (Leyser) LP
C/- Paterson Pitt Group

Attention: **Peter Dymock**
Dear Sir / Madam

SUBDIVISION RETICULATION – CMW: 112 Ripponvale Road, Cromwell. Shannon Farm. 160 Lots (Simple Estimate)

Thank you for your enquiry regarding the above subdivision.

Chorus is pleased to advise that, as at the date of this letter, we would be able to provide ABF telephone reticulation for this subdivision. In order to complete this reticulation, we require a contribution from you to Chorus' total costs of reticulating the subdivision. Chorus' costs include the cost of network design, supply of telecommunications specific materials and supervising installation. At the date of this letter, our estimate of the contribution we would require from you is $294,400.00 (including GST).

We note that (i) the contribution required from you towards reticulation of the subdivision, and (ii) our ability to connect the subdivision to the Chorus network, may (in each case) change over time depending on the availability of Chorus network in the relevant area and other matters.

If you decide that you wish to undertake reticulation of this subdivision, you will need to contact Chorus (see the contact details for Chorus Network Services above). We would recommend that you contact us at least 3 months prior to the commencement of construction at the subdivision. At that stage, we will provide you with the following:

- confirmation of the amount of the contribution required from you, which may change from the estimate as set out above;

- a copy of the Contract for the Supply and Installation of Telecommunications Infrastructure, which will govern our relationship with you in relation to reticulation of this subdivision; and

- a number of other documents which have important information regarding reticulation of the subdivision, including - for example - Chorus' standard subdivision lay specification.

Yours faithfully

Hollie Jackson

Hollie Jackson
Property Development Coordinator

PATERSONPITTSGROUP

APPENDIX F

Confirmation of Power Supply

AURORA ENERGY LIMITED
PO Box 5140, Dunedin 9058
PH 0800 22 00 05
WEB www.auroraenergy.co.nz

Aurora ENERGY

22 February 2019

Peter Dymock
Paterson Pitts Group
PO Box 84
Cromwell 9342

By email only: peter.dymock@ppgroup.co.nz

Dear Peter

ELECTRICITY SUPPLY AVAILABILITY FOR 160 LOT SUBDIVISION
112 RIPPONVALE ROAD, CROMWELL – LOT 2 DP 330709 & SEC 4, 11, 98, 101, 103 BLK III CROMWELL SD & PT SEC 5, 25 BLK III CROMWELL SD & PT RUN 1201R

Thank you for your inquiry outlining the above proposed development.

Subject to technical, legal and commercial requirements, Aurora Energy can make a Point of Supply[1] (PoS) available for this development.

Disclaimer

This letter confirms that a PoS **can** be made available. This letter **does not** imply that a PoS is available now, or that Aurora Energy will make a PoS available at its cost.

Next Steps

To arrange an electricity connection to the Aurora Energy network, a connection application will be required. General and technical requirements for electricity connections are contained in Aurora Energy's Network Connection Standard. Connection application forms and the Network Connection Standard are available from www.auroraenergy.co.nz.

Yours sincerely

Richard Starkey
COMMERCIAL MANAGER

[1] Point of Supply is defined in section 2(3) of the Electricity Act 1993.