Appendix J

Infrastructure Report

Infrastructure Assessment Report

Project

Applicant	Mount Iron Junction Limited
Patersons Ref	P240103
Date	20/02/2026

Version	Date	Prepared by	Reviewed by	Comments
0	20/02/2026	PHJ	AH	For fast track application

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1. Project Details

Site Location	237 Wānaka – Luggate Highway, 1 Junction Road, 10 and 21 Mountain Road and 37 Lake Hāwea – Albert Town Road, Wanaka
Legal Description	Lots 2 and 6 DP605028 and Lot 3 DP359869
Record of Title	1186396 (Lots 2 and 6 DP 605028) 243580 (Lot 3 DP 359869)
Area	5.9768 Ha

2. Scope

This report details the preliminary roading and servicing design for the substantive application for the Mount Iron Junction housing scheme under the Fast-track Approvals Act 2024. This application seeks fast track approval to construct 250 residential units and construct and operate a retail food market, café and childcare centre, along with associated infrastructure including roading, stormwater reticulation and disposal, services, reserves and cycle/walkway connections.

This report describes the following infrastructure elements for the development proposal

- Earthworks & Geotechnical
- Roading
- Stormwater
- Wastewater
- Water Supply
- Network Utility Services (electricity and telecommunications)
- Subdivision Staging

The earthworks and infrastructure design for the project is shown on the plans in **Appendix F of the fast track substantive application.**

A separate and subsequent Engineering Review and Approval process with Queenstown Lakes District Council (QLDC or Council) is anticipated to review and approve the detailed engineering design associated with each stage of the development. This approach is standard for land development projects in the QLDC area. Conditions requiring this Engineering Review and

Approval process are included with the draft set of conditions appended to the fast-track application. These conditions are the standard Engineering Review and Approval conditions tailored to the proposed development.

3. Proposed Infrastructure

3.1 Earthworks

3.1.1 Background Information

The development site is located on the intersection of State Highway 6 – Lake Hawea – Albert Town Road, State Highway 84 – Wanaka – Luggate Highway and Riverbank Road at the entrance to Wanaka and the base of Mount Iron. In recent years the site has been modified by the removal of the mature pine shelter belts across the site to enable the construction of the 450mm PE water supply main and the 375mm UPVC wastewater main that runs within the site (Lot 3 DP 359869) and parallel with the northwestern boundary. The southeastern part of the site has also been modified by the construction of the five-leg roundabout at the intersection of SH6, SH84 and Riverbank Road. The site has been further modified with the construction of Junction Road (the leg of the roundabout that provides access to the site) and Mountain Road within the site. An L-shaped bund around 7 Junction Road has been created from the surplus topsoil from these projects as that site has resource consent to be developed as a service station.

There is an existing 400m^2 workshop building onsite along with a number of outbuildings that will be removed as part of the construction works for the overall development.

The site slopes to the northeast at a grade of roughly 0.5%.

3.1.2 Proposed Development

The proposal is to develop the site into 250 residential units including a mixture of terrace houses and 2x three storey apartment buildings, and construct and operate a retail food market, café and childcare centre. It is anticipated that the works will be undertaken in multiple stages.

Earthworks to enable the development will extend over the entire site and will consist of cut to stockpile, cut to waste and cut to fill earthworks along with associated construction of the civil infrastructure.

The following table shows the overall anticipated earthwork volumes for entire development.

Earthwork Activity	Site Area	Volume m^3 insitu
Strip and Cut to Waste (Topsoil) offsite	5.9768Ha	$5000m^3$ (0.2m average depth)
Cut to waste existing topsoil stockpiles		$3250m^3$
Cut to Stockpile (Topsoil)	5.9768Ha	$7000m^3$
Dump to waste topsoil screening waste.		$1400m^3$
Topsoil respread	1.8700 Ha	$5600m^3$ (0.3m deep respread)
Cut to fill compact and certify	n/a	$5770m^3$
Import clean fill compact and certify	n/a	$4750m^3$
Import roading metal (300mm deep)	1.295Ha roading	$3885m^3$

3.1.2.1 Environmental Management Plan

An environmental management plan (EMP) has been developed by Enviroscope Limited outlining the various environmental mitigation measures available to facilitate the construction of the development. A copy of this report is included as an appendix to the Fast -track application.

The geotechnical reporting (contained in **Appendix B**) indicates that the site is underlaid by outwash gravels. These outwash gravels are highly permeable with measured rates in excess of 540mm per hour to the extent that the soakage rates in a number of soakage test pits were unable to be tested as the test pits drained to quickly to be measured.

This allowed the construction of the initial stage of the development and the adjacent roundabout without requiring any additional environmental measures beyond exposing of underlying gravels to contain stormwater onsite.

The overall site gradients pre and post developments are generally under 1% in all directions. The stripping of topsoil and other non-engineering soils to undertake the works will expose outwash gravels resulting in less potential runoff for the majority of the construction timeframes.

The measurements proposed by Enviroscope are contained within their report. These measures include

- Silt Fences
- Dirty and Clean water diversions
- Sediment ponds
- Stabilized entrances

A stage specific environmental management plan will be developed and submitted as part of the Engineering Review and Approval application for each stage of the development to address the design of specific environmental management details for the construction of each stage of the development.

3.2 Road Design Statement

3.2.1 Scope of Road Design Statement

The intention of this roading design statement is to provide design context for the proposed roading network. This roading design statement covers the following design aspects:

- Road dimensions and layout
- Link and place functions
- Connectivity
- How target operating speeds will be achieved
- How LID principles have been considered for stormwater run-off from the roads
- How Cyclists will be catered for
- Carparking
- Surrounding network

3.2.2 Road Dimensions & Layout

The land use and area type matrix contained in table 3.1 of the QLDC Land Development and Subdivision Code of Practice 2025 (LDSCoP) defines the land use and area type for the road design standards contained in Table 3.3 of the LDSCoP. The land use as defined by these tables is “Live and Play”.

The area type assessments are based on population density and public transport availability. There is no public transport in the Upper Clutha area which suggests a suburban area type, whilst the population density is in excess of 50 people per hectare at around 100 for the proposed development.

The development densities equate to either a E12 suburban or a E22 urban road layout, the requirements for these roads are identical apart from the design speed being either 30km/hr for E22 or 40km/hr for E12.

We have placed more weight on the public transport requirements than the population densities of table 3.1 of the LDSCoP and have therefore assessed the E12 road configuration for design.

The following tables describe the road design parameters for each proposed road and the reasons (if any) for a departure from the QLDC LDSCoP.

Table 1: **Mountain Road (West)**

Design Element	NZS4404 Table 3.3 Standard	Proposed Design	Comments
NZS4404 Figure Number	E12	E12 within 15m legal road reserve.	
Typical cross-section reference	-	Road Type A	Refer engineering drawings
Area Type	Urban	Urban	
Land Use	Live and Play	Live and Play	
Local Use	Primary access to Housing	Primary access to Housing	
Locality served	1 – 200 Dwelling Units	Serves 63 dwelling units.	Number of dwelling units served by this road decrease as vehicles progress to the cul-de-sac head.
Target operating speed	40km/ hr	40km/hr	Combination of recessed parking, narrow lane width, raised and flush crossings ensure 40km target operating speed is achieved.
Minimum road width (legal width)	15m	15m	
Maximum vertical grade	12.5%	0.745%	
Pedestrians	1.5m one side or 1.5m each side where more than 20 du or more than 100m in length.	1.5m each side	More than 20du and 100m length.

Passing, parking, loading & shoulder	Shared in movement lane	Shared in movement lane	Combination of recessed and shared in movement lane proposed.
Cyclists	Shared in movement lane	Shared in movement lane	
Classification	Local Road (< 2000vpd)		
Turning head		Cul-de-sac	In accordance with QLDC standard drawing B5-1
Road to be vested (YES/NO)	Yes		

Table 2: **Mountain Road (East)**

Design Element	NZS4404 Table 3.3 Standard	Proposed Design	Comments
NZS4404 Figure Number	E12	E12 within 15m legal road reserve.	
Typical cross-section reference	-	Road Type A	Refer engineering drawings
Area Type	Urban	Suburban	
Land Use	Live and Play	Live and Play	
Local Use	Primary access to Housing	Primary access to Housing	
Local attributes	Primary access to Housing	Primary access to Housing	
Locality served	1 – 200 Dwelling Units	Less than 187du	
Target operating speed	40km/ hr	40km/hr	Combination of ' narrow lane width recessed parking, raised and flush crossings to limit speed environment

Minimum road width (legal width)	5.5m- 5.7m	5.7m movement lane	
Maximum vertical grade	12.5%	0.65%	
Pedestrians	1.5m one side or 1.5m each side where more than 20 du or more than 100m in length.	1.5m each side	More than 20du and 100m length.
Passing, parking, loading & shoulder	Shared parking in the movement lane up to 100 du, separate parking required over 100du	Recessed carparking	Total dwelling units served by in excess of 100du
Cyclists	Shared in movement lane	Shared in movement lane	
Classification	Local Road		
Turning head	Travel around block		No turning head required
Road to be vested (YES/NO)	Yes		

Table 3: **Junction Road**

Design Element	NZS4404 Table 3.3 Standard	Proposed Design	Comments
NZS4404 Figure Number	E12	E12 within 15m legal road reserve.	
Typical cross-section reference	-	Road Type B	Refer engineering drawings
Area Type	Urban	Urban	
Land Use	Live and Play	Live and Play	
Local Use	Primary access to Housing	Primary access to Housing	
Local attributes	Primary access to Housing	Primary access to Housing	

Locality served	1 – 200 Dwelling Units	Less than 105 du equivalents	
Target operating speed	40km/ hr	40km/hr	Recessed parking, narrow lane width, raised and flush crossings to limit speed environment
Minimum road width (legal width)	5.5m- 5.7m 15m legal	5.7m movement lane 15m legal	
Maximum vertical grade	12.5%	0.65%	
Pedestrians	1.5m one side or 1.5m each side where more than 20 du or more than 100m in length.	1.5m on northwestern and southeastern side and shared with cyclists 3.0m on southeastern/western side	More than 20du and 100m length
Passing, parking, loading & shoulder	Shared parking in the movement lane up to 100 du, separate parking required over 100du	Recessed carparking where less than 100 du served, combination of recessed and in movement lane for balance.	Theoretical maximum of 105 dwelling unit equivalent served. Maximum flow entering extension of Junction Road from State Highway direction of 85 units served.
Cyclists	Shared in movement lane	shared with pedestrians 3.0m on southeastern/western side	Pedestrian/cycle connection through site to Mount Iron
Classification	Local Road		
Turning head		Travel around block or turn at cul-de-sac head	
Road to be vested (YES/NO)	Yes		

Table 4: **Road 1**

Design Element	NZS4404 Table 3.3 Standard	Proposed Design	Comments

NZS4404 Figure Number	E12	E12 within 15m legal road reserve.	
Typical cross-section reference	-	Road Type C	Refer engineering drawings
Area Type	Suburban	Suburban	
Land Use	Live and Play	Live and Play	
Local Use	Primary access to Housing	Primary access to Housing	
Local attributes	Primary access to Housing	Primary access to Housing	
Locality served	1 – 200 Dwelling Units	44 dwelling units	
Target operating speed	40km/ hr	40km/hr	Combination of recessed parking, narrow lane width, and short road length limit speed environment
Minimum road width (legal width)	5.5m- 5.7m	6.0m movement lane to accommodate adjacent angled parking.	No parking lines both sides of road to ensure single lane both ways and no parking within lane.
Maximum vertical grade	12.5%	0.98%	
Pedestrians	1.5m one side or 1.5m each side where more than 20 du or more than 100m in length.	1.5m each side	Second footpath added for majority of length to service carparking.
Passing, parking, loading & shoulder	Shared parking in the movement lane up to 100 du, separate parking required over 100du	Recessed carparking	45° and 90° carparking within road
Cyclists	Shared in movement lane	Shared in movement lane	
Classification	Local Road	Local Road	

Turning head	Cul-de-sac	Cul-de-sac	
Road to be vested (YES/NO)	Yes		

Table 5: **Roads 2 and 3** (identical design)

Design Element	NZS4404 Table 3.3 Standard	Proposed Design	Comments
NZS4404 Figure Number	E10	E10 within 12m legal road reserve.	No design standard for one way road. Assessed against E10 and best fit.
Typical cross-section reference		Road Type E	Refer engineering drawings
Area Type	Suburban	Suburban	
Land Use	Live and Play	Live and Play	
Local Use	Side or rear service access		Rear access to carparking for 24 units and primary access to 7 lots. Carparking associated with 18 public carparks
Local attributes	Side or rear service access	Carpark access for 24 lots and primary access for 7 lots.	
Locality served	Up to 100m in length between streets 1 to 20 lots. Passing every 50m.	95m between streets, serving 31 lots in single direction.	
Target operating speed	20km/ hr	20km/hr	The combination of a narrow carriageway width with angle parking restricts desirable speeds.

Minimum road width (legal width)	2.5m – 3.0m	4.35m from rear of carparks, no passing line length of road.	Minimum of 4m required for fire service, sufficient width for broken down car to be passed.
Maximum vertical grade	12.5%	0.65%	
Pedestrians	Shared in movement lane	1.5m each side.	More than 20du and 100m length.
Passing, parking, loading & shoulder	Allow for passing every 50m	Recessed carparking, One way road.	
Cyclists	Shared in movement lane	Shared in movement lane	
Classification	Local Road		
Turning head	N/A	Travel around block	
Road to be vested (YES/NO)	Yes		

Table 6: **Lane 4**

Design Element	NZS4404 Table 3.3 Standard	Proposed Design	Comments
NZS4404 Figure Number	E12	E12	
Typical cross-section reference	-	Road Type D	Refer engineering drawings
Area Type	Suburban	Suburban	
Land Use	Live and Play	Live and Play	
Local Use	Primary access to Housing	Primary access to Housing	
Local attributes	Primary access to Housing	Primary access to Housing	
Locality served	1 – 200 Dwelling Units	27du	
Target operating speed	40km/ hr	30km/hr	

Minimum road width (legal width)	5.5m	5.5m	
Maximum vertical grade	12.5%	1% max	
Pedestrians	One * 1.5m footpath	One * 1.5m footpath	
Passing, parking, loading & shoulder	Shared in movement lane	Shared in movement lane	
Cyclists	Shared in movement lane	Shared in movement lane	
Movement lane width	5.5m	5.5m	
Classification	Local road	Local road	
Turning head	N/A	Travel around block	
Road to be vested (YES/NO)	No		

3.2.3 Departures from LDSCoP

The transport assessment undertaken by Carrageway Consulting indicates roads are generally in compliance with the requirements of the QLDC LDSCoP and that the non-compliances identified can all be mitigated.

3.2.4 Place and Link Function

Section 3.2.4 QLDC LDSC0P states that "*the two fundamental roles of a road are to provide a space for interaction between people for a range of purposes and access to land so that movement between places can occur*".

The following two sections discuss the proposed design regarding both 'place context' and 'link context.'

Place Context
Place context is defined for the specific land use served and the broader area type in which the development is located.

The land use characteristic is defined according to the description of predominant activities in individual areas. QLDC LDSCOP uses "live, play, shop, work and learn, in addition to activities associated with growing, manufacturing and transporting goods and products".

Using Table 3.1 from QLDC LDSCoP, we have categorised the development area as follows:

(a) Land use: **live and play**
(b) Area type: **suburban**

The live and play land use is defined as "homes, home-based businesses, and mixed-use developments with residential uses, as well as parks and low-impact recreation". The proposed use of the development is for residential homes, local purpose and recreation reserves, walkway linkages and stormwater reserves and is consistent with the live and play land use.

Table 3.1 explains the transport context for the suburban area type as private vehicles are the predominant form of transport, with public transport providing peak flow on arterial and connector/collector roads. It further explains that non-motorised trips are primarily recreational and occur on local roads. Whilst the public transport component of this explanation is not currently applicable in the Wanaka context (as there is no publicly funded public transport system for Wanaka) and private vehicles will be the predominant form of transport for the next few years along with cycling and walking, it is anticipated that public transport will be in place at some time in the future.

3.2.5 Connectivity

Section 3.2.5 of QLDC LDSCoP states that a well-connected networks (roads and other links) are achieved with smaller block sizes and regular connections. Network connectivity shall be designed to achieve:

(a) Shorter travel distances;
(b) An increased number of alternative routes for all types of users;
(c) Increased opportunity for interaction;
(d) Improved access to public transport, cycling and walking networks, and access to destinations.

The site is limited to one road connection to the wider transport network via one leg of the adjacent roundabout. The internal roading layout is predominantly defined by the land parcel shape with an internal loop being created to join the existing roading, with no exit roads served by Cul-de-sac heads being provided from this to service the extremities of the site.

The site is adjacent to the Mount Iron Scenic Reserve which includes the active transport link between Albert Town and Wanaka. A new path is proposed to join onto this link to provide for both pedestrian and cycling access to the development.

3.2.6 Target Operating Speeds

Section 3.3.5 of QLDC LDSCOP states that traffic management shall be included in the road design to ensure that the target operating speeds (TOS) are achieved. Target operating speed can be managed by physical and psychological devices such as narrow movement lanes, reduced forward visibility, slow points, build-outs, lengths, chicanes, planting and landscaping, street furniture, and artwork. The two critical geometric factors contributing to achieving the target operating speed are carriageway width and forward visibility.

The proposed carriageway widths are consistent with QLDC LDSCoP's requirements to provide a suitable number of through lanes and make provision for car parking and passing manoeuvres.

An assessment of vehicle operating speeds has been completed by Carriageway Consulting and is appended to the fast-track application. The assessment indicates that the target operating speeds can be achieved.

3.2.7 LID Principles for Stormwater Runoff from Roads

It is proposed that all stormwater runoff from roads be directed to the roadside kerb and channel, which will, in turn, discharge into mud tanks and an underground piped drainage network. Ultimately all stormwater runoff from the roads will be piped to various stormwater reserves located across the site where the runoff will be detained to balance pre and post flows.

The design of the stormwater system is discussed in the stormwater report, which is included in **Appendix C**. The design is considered 'low impact' since all stormwater will be attenuated to pre-development flow rates. The proposed solution responds to the soil types on this site, which are dominated by outwash gravels which provide soakage for onsite disposal.

3.2.8 How Cyclists will be Provided For

The construction of Junction Road and Mountain Roads in the previous stage of the development provided a cycling connection to both the NZTA state highway network along with a connection to the adjacent Mount Iron Scenic Reserve.

The proposed layout provides a 3.0m shared pedestrian and cycle path along Junction Road from the northern tip adjacent to the highway to the Mount Iron Reserve. From here existing informal pedestrian/cycle paths are to be upgraded to connect into the existing commuter and recreational cyclist track around the base of Mount Iron which then connects within the active transport networks around Wanaka, Three Parks and Albert Town.

3.2.9 Carparking

Carparking is provided via a combination of public and private carparks to meet the requirements of the development. The proposed density of the develop leaves insufficient space between driveways to accommodate the required number of on-street carparks, this has been addressed by a combination of indented parks where space allows, on street parking where allowed and a central public carpark to address the shortfall in available street frontage. The central public carpark is located adjacent to the existing walkway access to the Mount Iron Reserve so will also serve a catchment wider than just the residents within the development. The specific provision of public and private carparks and required numbers are further discussed within the traffic report.

3.2.10 Refuse Collection

Refuse collection will be provided by a combination of both private and public collection. The units on the west side of Mountain Road West and Junction Road will utilise Council's roadside refuse collection services and sufficient space has been provided for the placement of wheelie bins on the berms in front of each of the units on rubbish collection day and for screened wheelie bin storage.

Units/lots 1 -62 are proposed to have rubbish collection via Council's weekly roadside refuse collection service.

Common collection facilities have been shown adjacent to apartments and central carparks for all other sies within the development. There are six private waste collection points proposed adjacent to Units 300, 327, 412, 518, the two apartment blocks will each have a private refuse area as will the childcare centre and the café market building. The private roading around these areas has been designed to enable rubbish truck access and manoeuvring as well as space for manoeuvring area for accessing the communal bins.

The building act requires that these common collection areas have facilities to allow washdown and that the drainage from these facilities is collected by the wastewater network. A foul sewer, stormwater and water connection has been provided to each of these proposed facilities.

An automatic gate between the café / market and the adjacent apartment building has been placed to facilitate one way access for trucks to service both for deliveries and rubbish collection.

3.2.11 Street Lighting

A full street lighting design will be provided to meet the needs of each stage of the development as part of the associated Engineering Review and Acceptance process. Indicative road lighting has been shown on design plans based on the spacing of the existing luminaires in the first stage of the development.

There is sufficient space within the roadside berms to accommodate more or less streetlights to satisfy the requirements of Council's Southern lighting Strategy and minimum illumination calculations required to satisfy minimum lux.

3.2.12 Vesting of Assets

The following roads will be vested in Council as part of the development.

Mountain Road (both east and west sections)
Junction Road
Roads 1 - 3
Carpark 2

Other accesses and parking areas will remain private.

3.3 Stormwater

3.3.1 Proposal

Stormwater is proposed to be disposed to ground via engineered soakage upto the 1% AEP event. Emergency or super design flows will track via roads to existing offsite flow paths.

The stormwater report for the proposed development is attached as **Appendix C** of this report. The engineering plans in **Appendix A** show the proposed location of the stormwater network for the development, subject to detailed design.

3.3.2 Feasibility Design

The design has been undertaken on the based of the stormwater report and there is sufficient space to accommodate the required soakpits in the locations shown. It is envisaged that each of these soakpits will incorporate a pre-treatment device prior to discharge to ground which will be detailed as part of the engineering acceptance process.

We would envisage that each soakpit location will be re-tested at the proposed discharge level prior to the engineering approval application to validate the stormwater report soakage rate assumptions.

3.3.3 Apartment Blocks

The land associated with the two apartment blocks and shopping hub located on lots 101-103 is not serviced by the proposed reticulation, the associated land for these two blocks slopes away from the proposed roads within the development and will require specific stormwater design to be undertaken when the buildings/carparks are constructed.

3.3.4 Private Stormwater

The soakpits and treatment devices located outside of the Council roads and reserves servicing the common carpark areas will be owned and maintained by the associated residents association / body corporate. This includes the following soakpits A,B, F and G.

3.3.5 Vesting of Assets

It is proposed that all new stormwater reticulation located within the roads and reserves will vest in Council on completion of the subdivision, except for the laterals and soakpits within private lots.

3.4 Wastewater

3.4.1 Overview

The development is located adjacent to the trunk wastewater reticulation and the primary sewage pump station serving the Upper Clutha area. A 375mm ID gravity main was constructed through the site along the northwestern boundary and into the reserve land adjacent to the site as part of Council's upgrade works in 2024.

As part of the installation of this sewer main two 150mm ID branch stub laterals were provided to service the development land. These pipes are however of insufficient depth to service the majority of the site by gravity, with only the sites adjacent to Mountain Road West being serviceable.

The use of a low pressure sewer network has been investigated but was ruled out due to the density of the development and a common pump station being more cost effective both in construction costs and ongoing maintenance costs.

A common vested foulsewer pump station is proposed to service the majority of the development. This pump station is proposed to be located in Lot 804 at the northern tip of the site, adjacent to the existing Council sewage pump station.

3.4.2 Proposed Wastewater Infrastructure

The proposal is to reticulate the majority of the site to a new pump station adjacent to the existing SPPP2- Albert Town #2 pump station located at the northeastern end of the site. The detail associated with this pump station will be submitted as part of the detailed design and the associated engineering acceptance process.

The remainder of the site, being the lots to the north of Mountain Road West will be reticulated via 150 dia gravity reticulation connecting into a new manhole on the existing 150dia Council sewer line ID 402350 at the southern end of the existing Mountain Road.

3.4.3 Design Constraints/Configuration

The location of the existing 450mm ID trunk water supply main and the 375mm uPVC wastewater line along the northwestern boundary is affected by the location of the northern units and means that access to the pipe for future maintenance would be via the rear yards of 37 residential units. It is therefore proposed to construct replacements for these trunk mains within Junction Road.

There were two options for the location of the new 150dia gravity main across the site, being via Junction Road or across the site as shown. The depth of this service along Junction Road precluded this option as the separation required between services to accommodate the vertical separations could not be accommodated within the road corridor without compromising the ability to access/ maintain these in the future.

3.4.4 Modelling

The proposed rising main from the new pumpstation is shown connecting into the trunk main adjacent to the site but it can connect into the most suitable location to meet Councils operating requirements.

All reticulation has been designed to meet the minimum grade requirements of Tables 5.4 and 5.5 of the QLDC LDSCoP. The majority of the grades are 0.55% with increased grades of 1% for the terminal ends.

In practical terms a 150 dia sewer at these grades will service up to 250 dwellings The total number of units / lots within the development is 250. There is only one section of the sewer that accommodates the full 250 lots being the last section to the pump station (line 1-1 – 1-2), this line has been steepened to 0.8% to ensure capacity.

Council's Property and Infrastructure Team via email from Richard Powell dated 12/09/2025 have confirmed that modelling is not required for this development as the infrastructure we are connecting to has been sized for growth within the wider network including this site.

3.4.5 Vesting of Assets

It is envisaged that all foul sewer infrastructure excluding 100mm lot laterals will be vested in Council. This includes the proposed gravity reticulation pump station and associated rising main.

This will require service easement corridors for sewers located within private land.

3.5 Water Supply

3.5.1 Overview

The subject site is currently served via a 450mm ID water main located along the northwestern boundary of the site. It is proposed to relocate the portion of this main that is currently through the site into the Junction Road corridor. The previous stage of the development constructed a new DN 250mm main beneath Mountain Road from this main.

The proposed network consists of a network of 180mm and 125mm mains to provide firefighting along with 63mm ridermains to provide domestic supply.

Fire hydrants have been located to meet spacing requirements of PAS SNZ 4509:2008 along with sufficient hydrants being over 6m from adjacent buildings to satisfy FW2 firefighting requirements.

3.5.2 Water modelling

Council have confirmed via email from Richard Powell dated 12/09/2025 that modelling for this development is not required for the new density as the infrastructure this development is connecting to has been sized for growth within the wider area including this site

3.5.3 Existing Water Reticulation

The site is currently served by a single ended 250mmID watermain, at the completion of the development this will be linked back onto the 450mm trunk main to create a looped supply off this main.

3.5.4 Vesting of Assets

The majority of water infrastructure is located within proposed road reserves and will vest in Council at the completion of each stage of the development.

There are two rider mains located on private land serving Lots 1-5 and 310 -313 that will also be vested in Council. These will include suitable easements to protect Councils interests.

3.6 Network Utility Services

3.6.1 Electricity

Aurora Energy have confirmed that supply can be made available to the site (see confirmation in **Appendix D**). There is sufficient space within the roading corridor to accommodate the associated cabling. The transformer locations will be dictated by future staging. The density of the development means that these will be located outside of the roading corridors.

3.6.2 Telecommunications

Chorus have confirmed supply can be made available to the site. A copy of this confirmation email is contained in **Appendix E**. There is sufficient space within the roading corridor to accommodate the associated cabling, service boxes, etc.

3.7 Development Staging

3.7.1 Staging Proposal

It is envisaged that the development will be undertaken in various stages to accommodate site constraints and market demands. A preliminary staging plan is contained in the scheme plan set included within the fast-track application. It is anticipated that Stage One will be the lots adjacent to Mountain Road West as these lots have all services at the start of the road extension.

The other stages are less certain as require the completion of the foul sewer pump station and relocation of the existing trunk services.

A separate Engineering Approval will be applied for each stage of the development to address the required infrastructure requirements.

3.8 VESTING OF INFRASTRUCTURE

3.8.1 Vesting of Assets

It is envisaged that Roads 1-3 , Mountain Roads (east and west) and Junction Road will vest in Council. The associated services within these roads will also vest in Council.

All trunk services located on private land will be covered by easements and the associated services vested in Council , this is envisaged to be the 150mm diameter sewer (Line 1) across the site along with the initial sections of the 450mm trunk sewer and 375mm watermain.

A full vesting summary will be provided as part of the Engineering Acceptance for each individual stage of the development.

4. Conclusion

All necessary roads, drainage and reticulated services can be provided to the proposed residential lots. The detailed design of these services and any associated staging will be further developed under the detailed design phase of the development.

Development contributions payable for this development will mitigate any effect on the Council's existing infrastructure.

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APPENDIX A ENGINEERING PLANS

See Fast Track substantive application Appendix F

APPENDIX B GEO-TECHNICAL REPORT

Geotechnical Report

237 Wanaka-Luggate Highway,

Wanaka

Report prepared for:

Mt Iron Junction Limited

Report prepared by:

GeoSolve Limited

Distribution:

Mt Iron Junction Limited

Paterson Pitts Group

GeoSolve Limited (File)

March 2018

GeoSolve Ref: 170839

Table of Contents

1 Introduction

1.1 General

This report presents the results of a geotechnical investigation carried out by GeoSolve Ltd in order to determine subsoil conditions, stormwater soakage capability and earthworks recommendations at 237 Wanaka-Luggate Highway, Wanaka. Geotechnical design parameters and foundation bearing parameters are provided. Rockfall and seismic hazard has been assessed. The proposed development plan area has been provided by McCoy Wixon Architects via Paterson Pitts Group.

Photo 1. View of the site looking northeast from TP1.

The investigation was carried out for Mount Iron Junction Limited in accordance with GeoSolve Ltd.'s proposal dated 27 October 2017, which outlines the scope of work and conditions of engagement. This report will supplement a resource and earthworks consent application.

1.2 Proposed Development

We understand it is proposed to develop the above property into a commercial area and this requires geotechnical assessment of the site to assess suitability for development and to identify any geotechnical issues.

Figure 1, Appendix A shows a concept plan for the proposed development.

2 Site Description

2.1 General

The subject property, legally described as Lot 5 DP 15016, is located approximately 2.5 km east of central Wanaka, as shown in Figure 1 below.

Figure 1: Site location (blue symbol) in relation to Wanaka township (Source: http://maps.qldc.govt.nz/qldcviewer/)

The property is accessed off Wanaka-Luggate Highway and is situated to the southeast of Mt Iron.

Two dwellings, a large garage and a sleepout currently occupy the site. The remaining area of the site has been divided into small paddocks which are separated by fencing. Unsealed roads have been created to access the dwellings with some asphalt poured within the driveway of the northeast dwelling. Ground cover comprises grass, shrubs and pine trees.

The site is bounded by the Wanaka-Luggate Highway to the south, Albert Town-Lake Hawea Road to the east and Crown Land and 37 Albert Town-Lake Hawea Road to the northwest.

2.2 Topography and Surface Drainage

The site topography is generally sub-horizontal across the property. The site was observed to be naturally free-draining. No earthworks plans have been provided to GeoSolve at this stage although these are anticipated to be relatively minor.

No spring flows or seepages were observed during site investigations.

3 Geotechnical Investigations

GeoSolve Ltd visited the subject property on the 18th and 19th of December 2017 and the 17th of January 2018 undertaking an engineering geological site inspection with confirmatory subsurface investigations.

The investigations carried out for the purposes of this report are as follows:

- A site inspection and field mapping by an engineering geologist to assess rockfall risk for the proposed development;
- 17 Test pits (TP), extending to a maximum depth of 4.5 m below ground level (bgl) to produce geological logs of the subsoils;
- 10 Scala penetrometer tests extending to a maximum depth of 1.4 m bgl to assess relative density of the subsoils;
- 2 Heavy Dynamic Probe (DPH) tests, extending to a maximum depth of 15 m bgl to assess relative density of subsoils at depth and confirm the ground water model below the site;
- Piezometer installation;
- 4 Soakage pits and 2 HT21 standpipe permeability tests to assess permeability in the two proposed soakage areas at the development.

Test pit and Scala penetrometer locations and logs are presented in Appendices A and B respectively.

DPH locations and logs are presented in Appendices A and C.

Permeability test locations and logs are presented in Appendices A and D.

4 Subsurface Conditions

4.1 Geological Setting

The site is located in the Wanaka Basin, a feature formed predominantly by glacial advances. The schist bedrock within the basin has been extensively scoured by ice and lies at considerable depth below this site. Overburden material above the schist in this region includes glacial till, alluvial outwash sediments, lake sediment and beach deposits.

During the Mt Iron and Hawea Glacial Advances 18-23,000 years before present, the glaciers terminated upstream from Albert Town forming moraine loops and outwash terraces. Well-consolidated glacial till gravels were laid down on the flanks and beds of the glaciers. With the final retreat of the ice, about 16,000 years ago, Lake Wanaka formed and the Clutha River became entrenched in the glacial deposits.

Mt Iron lies directly to the north of the property where bedrock is exposed along the face of the southeastern bluff.

The Cardrona fault is mapped near the southeast corner of the property, this fault is considered capable of earthquakes of Magnitude 7.3 but has an average return period of

5,000-10,000 years. The Alpine Fault, located approximately 70 km away, runs along the western foothills of the Southern Alps, and is likely to present a more significant seismic risk to the site in the short term. There is a high probability that an earthquake of Magnitude 8 or more will occur along the Alpine Fault within the next 50 years and such a rupture is likely to result in strong ground shaking in the vicinity of Wanaka.

4.2 Stratigraphy

Results from the test pitting indicate the sub-surface stratigraphy comprises:

- 0.1 to 0.2 m of topsoil, overlying;
- Isolated uncontrolled fill (1.2-1.7 m in TP6, SP1 and 2 only), overlying;
- Isolated buried topsoil (0.1-0.4 m in TP6, SP1 and 2 only), overlying;
- 0.1 to 0.3 m of loess, overlying;
- 0.3 to 4.2 m+ of outwash gravel, interbedded with;
- Lenses of outwash sand, 0.3-0.9 m thick were observed within the outwash gravel.
- Lake sediment is inferred to underlie the outwash gravel at approximately 11-12 m bgl in the area of DPH 1.

Topsoil was observed at the surface of all test pits except SP1 and 2 and predominately comprises brown, organic SILT with roots and rootlets.

Uncontrolled fill was observed to underlie the topsoil in TP6, SP 1 and 2 and extends to 1.2 to 1.7 m bgl. The fill comprises light grey/grey, loose to medium dense, gravelly SAND with minor organic silt and trace sticks, rootlets and wire, SAND with some gravel and silt, and sandy GRAVEL with trace cobbles, boulders and organic silt.

Buried topsoil was observed to underlie the uncontrolled fill in TP6, SP1 and 2 and extends to 1.6 to 1.8 m bgl. Buried topsoil comprises, brown sandy organic SILT with minor rubbish and gravel.

Loess was observed to underlie the topsoil in 15 of the 17 test pits and extends to a depth of 0.2 to 0.5 m bgl. The loess predominately comprises light brown, firm silty SAND with minor rootlets.

Outwash gravel was observed to underlie the loess, topsoil or buried topsoil in all test pits. Outwash gravel typically comprises brown, grey and dark grey medium dense, sandy GRAVEL with some to trace cobbles and trace boulders. Boulders up to 0.7 m diameter were observed. Outwash gravel was observed to the termination depth of all test pits.

0.3 to 0.9 m thick outwash sand lenses were observed in TPs 3, 6 and 11 and typically comprise grey/brown, medium dense SAND with minor to some gravel and gravelly SAND.

Lake sediment is inferred to underlie the outwash gravel at 11-12 m bgl in the DPH 1 area from the relative density observed in the DPH test and knowledge of relative levels and relative density of lake sediment in the Albert Town area.

Full details of the observed subsurface stratigraphy can be found within the test pit and soakage pit logs contained in Appendix B and D respectively.

4.3 Groundwater

Groundwater was not observed during test pit investigations which extend to 4.5 m bgl.

A piezometer was installed to 6.7 m depth in close proximity to DPH2 and was dipped dry to full depth. Piezometers could not be installed any deeper to reach the water table due to coarse cobbles and boulders within the outwash gravel unit.

4.4 Slope Stability

No instability features were observed on the site during investigations.

The bluffs of Mt Iron outcrop to the north of the site and a rockroll hazard is shown on the QLDC hazard database within 70 m of the north-western boundary. Rockfall from the bluffs to the north has been assessed as part of site investigations, this is detailed in section 6.7 of this report.

5 Liquefaction Analysis

5.1 Introduction

A preliminary liquefaction assessment has been undertaken using test pit and heavy dynamic probe (DPH) data. Two Heavy Dynamic Probe (DPH) tests were undertaken within the site to assess liquefaction risk.

5.2 Earthquake Scenarios

In accordance with NZS1170 – Structural Design Actions[1], the following two earthquake scenarios were considered based on a building with Importance Level 2 with a 50 year design life.

These scenarios represent the following design performance requirements:

- Serviceability Limit State (SLS) – to avoid damage that would prevent the structure from being used as originally intended, without repair, and;
- Ultimate Limit State (ULS) – to avoid collapse of the structural system.

In terms of NZS 1170, Class D subsoil conditions (deep soils) were assumed to underlie the site.

The methods presented within the NZTA Bridge Manual (2014)[2] have been adopted for deriving the site peak ground accelerations (PGA) as they use unweighted seismic hazard factors and corresponding (effective) earthquake magnitudes that are better suited to be used in the assessment of liquefaction.

[1] NZS1170-5 (2004) Structural Design Actions, Part 5: Earthquake Actions – New Zealand.
[2] NZTA Bridge Manual, Third Addition, Amendment 2, Effective from May 2016 (Manual Number SP/M/022).

Table 1 below provides a summary of the annual exceedance probability, effective magnitude and PGA adopted for each seismic case analysed in the liquefaction assessment.

Table 1: Annual exceedance probability, effective earthquake magnitude and peak horizontal ground accelerations for each seismic case

Seismic Case	Annual Exceedance Probability (AEP)	Effective Magnitude	Peak Horizontal Ground Acceleration (g)
Serviceability Limit State (SLS) design earthquake	1/25	6.1	0.08
Ultimate Limit State (ULS) design earthquake	1/500	6.2	0.32

5.3 Liquefaction Assessment

5.3.1 General

Liquefaction occurs when susceptible, saturated soils attempt to move to a denser state under cyclic shearing. In this report, liquefaction is defined as when pore pressures rise to reach the overburden stress. When this occurs, the following effects can happen at flat sites:

- Loss of strength;
- Ejection of material under pressure to the ground surface (i.e. surface disruptions), and;
- Post-liquefaction volumetric densification as the soils reconsolidate.

In addition, sloping sites or sites with a ‘free face’ may experience lateral spreading or movement.

The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and depth to the groundwater table.

5.3.2 DPH Analysis

Analyses were performed to evaluate the liquefaction potential of the lake sediment unit underlying the outwash gravel and the discrete sand lenses within the outwash gravel unit utilising the methods recommended by Idriss & Boulanger (2014)[3]. These methods use information obtained from soil logging and in situ testing, such as soil type, fines content, layer thicknesses, and blow count.

[3] Boulanger R.W. and Idriss, I.M. (2014). ‘CPT and SPT Based Liquefaction Triggering Procedures,’ Report No. UCD/CMG–14/01, Dept. of Civil & Environmental Engineering, University of California at Davis.

A piezometer was installed to 6.7 m bgl in close proximity to DPH 1 which was dipped dry to full depth. This has been assumed as the water table depth for analysis purposes even though this is likely a conservative assumption.

The liquefaction analysis results are summarised below in Table 2.

Table 2: Summary of liquefaction results from DPH testing

Factor	Assessment		Implications
Crust thickness	The crust thickness is determined to be at least 8.7 m for the ULS design earthquake. Data from the Canterbury earthquake sequence plus other historic earthquakes[4] has been collated and observed surface damage compared with crust thickness. This data indicates that surface damage is likely for crusts of less than about 3.5 m thickness.		The crust is significantly thicker than 3.5 m and therefore should be sufficiently thick to limit surface damage in a ULS seismic event. Particularly given the minor (0-10 mm) predicted liquefaction induced settlement within the upper 10 m of the soil profile.
LSN	1/500 AEP (ULS)	LSN range = 0-7	Surface expression of liquefaction unlikely.
Free field settlement	1/500 AEP (ULS)	0-10 (80) mm	0-10 mm estimated in the upper 10 m in the areas tested. 80 mm of total settlement is predicted within testing completed to 15 m depth. Lake sediment is inferred at 11.5-14.5 m bgl in DPH 1 which is predicted to liquefy under ULS seismic loading.
Lateral spread	Lateral spreading under seismic loading is not expected to occur as the site is relatively flat without any nearby free face.		None.

[4] Bowen, H.J. and Jacka, M.E. (2013). Liquefaction induced ground damage in the Canterbury Earthquake: Predictions versus reality. Proceedings of the 19th NZGS Geotechnical Symposium. Editor CY Chin. Queenstown, New Zealand.

6 Engineering Considerations

6.1 General

The recommendations and opinions contained in this report are based upon ground investigation data obtained at discrete locations on site and historical information held on the GeoSolve database. The nature and continuity of subsoil conditions away from the investigation locations is inferred and cannot be guaranteed.

6.2 Geotechnical Parameters

Table 3 provides a summary of the recommended geotechnical design parameters for the soils expected to be encountered during construction of any future buildings and retaining walls.

Table 3: Recommended Geotechnical Design Parameters

Unit	Thickness (m)	Bulk Density γ (kN/m^3)	Effective Cohesion c' (kPa)	Effective Friction ϕ' (deg)	Elastic Modulus E (kPa)	Poissons Ratio ν
Topsoil/Buried Topsoil (organic SILT with roots and rootlets and sandy organic SILT with minor rubbish and gravel)	0.1-0.4	16	To be removed from building and engineered fill footprints			
Uncontrolled Fill (loose to medium dense, gravelly SAND with minor organic silt, SAND with some gravel an silt and sandy GRAVEL with trace organic silt, cobbles and boulders)	1.2-1.7	18	To be removed from building and engineered fill footprints			
Loess (firm, silty SAND)	0.1-0.3	18	0	31	5,000	0.3
Outwash Gravel with Outwash Sand lenses (medium dense, sandy GRAVEL with trace to some cobbles and trace boulders. Lenses of gravelly SAND to SAND with minor gravel)	0.3-4.2	18	0	36 (32 in Sand)	10,000-20,000	0.3

6.3 Site Preparation/Earthworks

During the earthworks operations all topsoil, uncontrolled fill, organic matter and other unsuitable materials should be removed from the construction areas in accordance with the recommendations of NZS 4431:1989. These soil types (and loess SILT) will also need to be removed from areas where engineered fill is proposed.

Robust, shallow graded sediment control measures should be instigated during construction where rainwater and drainage run-off across exposed soils is anticipated. If slope gradients in excess of 4% are proposed in fine-grained soils then the construction and lining of drainage channels is recommended, e.g. with geotextile and suitably graded rock, or similarly effective armouring.

Water should not be allowed to pond or collect near or under a foundation slab. Positive grading of the subgrade should be undertaken to prevent water ingress or ponding.

All fill that is utilised as bearing for foundations should be placed and compacted in accordance with the recommendations of NZS 4431:1989 and certification provided to that effect. The outwash gravel soils can be used as engineered fill on site (during good weather and in accordance with an earthfill specification). The topsoil, loess and uncontrolled fill is not suitable as a fill source. Maximum density and optimum moisture content will vary in the outwash gravel. Boulders and cobbles over 100 mm in size will need to be screened from engineered fill sources.

6.4 Excavations

At this stage no earthworks plans have been provided, although it is expected cuts will be made within topsoil, uncontrolled fill, loess, and outwash soils. It is expected that only minor earthworks will take place across the site due to the generally sub-horizontal topography and the shallow depth to suitable bearing soils across the majority of the site. Earthworks plans have yet to be developed for the development.

Recommendations for temporary and permanent batter slope angles are described below in Table 4. Slopes that are required to be steeper than those described below should be structurally retained or subject to specific geotechnical design.

All slopes should be periodically monitored during construction for signs of instability and excessive erosion, and, where necessary, corrective measures should be implemented to the satisfaction of a Geotechnical Engineer or Engineering Geologist.

No seepage was encountered during test pitting and hence groundwater is unlikely to be encountered during excavations. However, a geotechnical practitioner should inspect any seepage, spring flow or under-runners that may be encountered during construction.

The soils are anticipated to be excavated by conventional methods, however boulders are likely to be encountered within the outwash gravel.

6.4.1 Cut Slopes in Soil Materials

Table 4 summarises the recommended batter angles for temporary and permanent slopes up to 3 m high, which are formed in the soil materials identified at the site.

Table 4: Recommended maximum batter angles for cut slopes up to 3 m high in site soils

Material Type	Recommended Maximum Batter Angles for Temporary Cut Slopes Formed in Soil (horizontal to vertical)		Recommended Maximum Batter Angles for Permanent Cut Slopes Formed in Soil – dry ground only (horizontal to vertical)
	Dry Ground	Wet Ground	
Topsoil/Loess/Uncontrolled Fill	2H: 1V	3H: 1V	3H: 1V
Outwash gravel	1H: 1V	2H: 1V	2H: 1V

6.5 Engineered Fill Slopes

All fill should be placed and compacted in accordance with the recommendations of NZS4431: 1989 and Queenstown Lakes District Council Standards. All cut and fill earthworks should be inspected and tested as appropriate during construction and certified by a Chartered Professional Engineer.

All un-retained fill slopes which are less than 3.0 m high should be constructed with a batter slope angle of 2.0H:1.0V (horizontal to vertical) or flatter and be benched into sloping ground.

Reinforced earth slopes can be considered if batters need to be steeper than 2H:1V.

6.6 Ground Retention

All retaining walls should be designed by a Chartered Professional Engineer using the geotechnical parameters recommended in Table 3 of this report. Due allowance should be made during the detailed design of all retaining walls for forces such as surcharge due to the sloping ground surface behind the retaining walls, groundwater, seismic and traffic loads.

All temporary slopes for retaining wall construction should be battered in accordance with the recommendations outlined in Table 4 of this report. Where these batter slopes cannot be achieved temporary retaining will be required.

Groundwater was not observed within a piezometer installed to 6.7 m beneath the site or within any of the test pit excavations. To ensure any groundwater seeps and flows are properly controlled behind the retaining walls, the following recommendations are provided:

- A minimum 0.3 m width of durable free draining granular material should be placed behind all retaining structures;
- A heavy duty non-woven geotextile cloth, such as Bidim A14, should be installed between the natural ground surface and the free draining granular material to prevent siltation and blockage of the drainage media;
- A heavy-duty (TNZ F/2 Class 500) perforated pipe should be installed within the drainage material at the base of all retaining structures to minimise the risk of

excessive groundwater pressures developing. This drainage pipe should be connected to the permanent piped storm water system, and;

- Comprehensive waterproofing measures should be provided to the back face of all retaining walls forming changes in floor level within the dwelling to minimise groundwater seepage into the finished buildings.

It is recommended that the retaining wall excavation batters are inspected by a suitably qualified and experienced Geotechnical Engineer or Engineering Geologist.

6.7 Rockfall Hazard

6.7.1 General

An engineering geologist has undertaken site mapping to assess the risk of rockfall hazard to the proposed development. The assessment reviews the risk of boulders originating as rockfall from the steep bluffs below Mount Iron rolling out into the proposed development and damaging proposed structures. Rockfall events require a trigger such as strong seismic shaking or long-term weathering and failure of the rock mass.

Numerous boulders of varying diameters and shapes have been observed on the sub-horizontal alluvial outwash surface at the base of Mount Iron. To assess the risk to the proposed development boulders observed on the ground surface were mapped and differentiated between those originating as rockfall and those originating as alluvial outwash boulders (Appendix A, Figure 2). Roll out distance between the base of Mount Iron and the north-western property boundary was also considered including any natural barriers against rockroll.

There is no evidence of historic rockroll boulders on the ground surface within the boundaries of the proposed development nor was there any evidence of historic rockroll boulders observed in test pits. All boulders observed in test pits are interpreted to be alluvial outwash boulders. The mapped maximum roll out distance of historic rockroll boulders from the base of Mount Iron onto the outwash surface ranges between 40-70 m. The minimum distance between the base of Mount Iron and the northwest property boundary is approximately 115 m at the southwest corner of the proposed development. The roll out distance between the base of Mount Iron and the proposed development gradually increases towards the northeast to a maximum distance of approximately 180 m. It is also noted on the proposed development plans provided by McCoy Wixon Architects that there is a designated “no build zone” on the southwest corner of the proposed development.

There are several existing natural barriers against rockroll present between the base of Mount Iron and the proposed development. Existing rockfall debris at the base of Mount Iron and the dense patches of native kanuka scrub on the outwash surface provide a natural barrier against rockroll. The wing of a lateral moraine ridge extends towards the northeast and acts as a natural rockroll bund for the southwest corner of the proposed development (see Appendix A, Figure 2). Numerous boulders resulting from rockroll have already been observed to be piled up behind this moraine ridge north of the Wanaka-

Luggate Highway. The sub-horizontal (0-5°) alluvial outwash surface from the base of Mount Iron to the proposed development provides a suitable setback for rockroll fallout.

Rockfall hazard mapping is shown on Appendix A, Figure 2.

6.7.2 Rockfall Hazard Considerations and Recommendations

Based on the mapping of historic rockroll boulders and the roll out distance from the base of Mount Iron the resulting hazard envelope does not reach the proposed development. Therefore, the rockfall hazard poses no risk to the proposed development and further detailed analysis of the rockfall hazard is not considered necessary.

As a precaution the existing row of pine trees along the northwest property boundary could be left in place to provide a further natural barrier against rockroll. Alternatively, the pine trees could be replaced with another tree species if this is desired.

6.8 Seismic Hazard

The Cardrona Fault is mapped near the southeast corner and eastern boundary of the property and its location is recorded as concealed on published geological mapping beneath the Albert Town area. The Cardrona Fault is indicated as 'active'. The risk of ground rupture on the site from known faulting is considered unlikely. Movement on the Cardona Fault would however result in ground shaking of the site, and the wider Wanaka area.

Geosolve have completed an assessment of the risk posed by the Cardona Fault using guidelines provided by the Ministry of Environment for developing land close to active faults. For the assessment, the Cardrona Fault has been categorised with a return period of 5,000 to 10,000 years (GNS Science website, Active Faults Database), and the location is assessed as uncertain, as indicated on published geological mapping.

Following the Ministry of the Environment guidelines provided in Section 11 "Taking a Risk-Based Approach to Resource Consents", building importance category structures 1, 2a and 2b, are a permitted activity and category 3 structures are a discretionary activity. NZS 3604 dwelling structures fall under category 2a, and are therefore considered to be a permitted activity in close proximity to the Cardrona Fault system.

In conclusion, given the relatively long return period for the Cardrona Fault (5,000 - 10,000 Years), the Alpine Fault, with a return period for major earthquakes of 300-350 years, and predicted ground accelerations an order of magnitude higher than the Nevis Cardona, is considered to provide the governing seismic risk to the area.

6.9 Groundwater Issues

The regional water table is expected to lie at depth below any future foundation levels and is not expected to be encountered during construction on this site. Dewatering or other groundwater-related construction issues are therefore unlikely to be required.

It is important that GeoSolve be contacted should there be any seepage, spring flow or under-runners encountered during construction.

6.10 Foundation Considerations

Topsoil, uncontrolled fill and loess should be stripped from the building platform areas. Foundation loads will be transferred to the outwash gravel or engineered fill overlying outwash gravel in all cases.

All unsuitable materials identified in foundation excavations, particularly those softened by exposure to water, should be undercut and replaced with engineered fill during construction. Any fill that is utilised as bearing for foundations should be placed and compacted in accordance with NZS 4431:1989 and certification provided to that effect.

To minimise the effects of freeze-thaw cycles in footings founded on soil, all shallow foundations should be founded a minimum of 0.4 m below the adjacent finished ground surface.

Figure 2 summarises the recommended working stresses for shallow footings, which bear upon outwash gravel and engineered fill overlying the same. It should be noted the foundation working stresses presented on Figure 2 are governed by bearing capacity in the case of narrow footings and settlement in the case of wide footings.

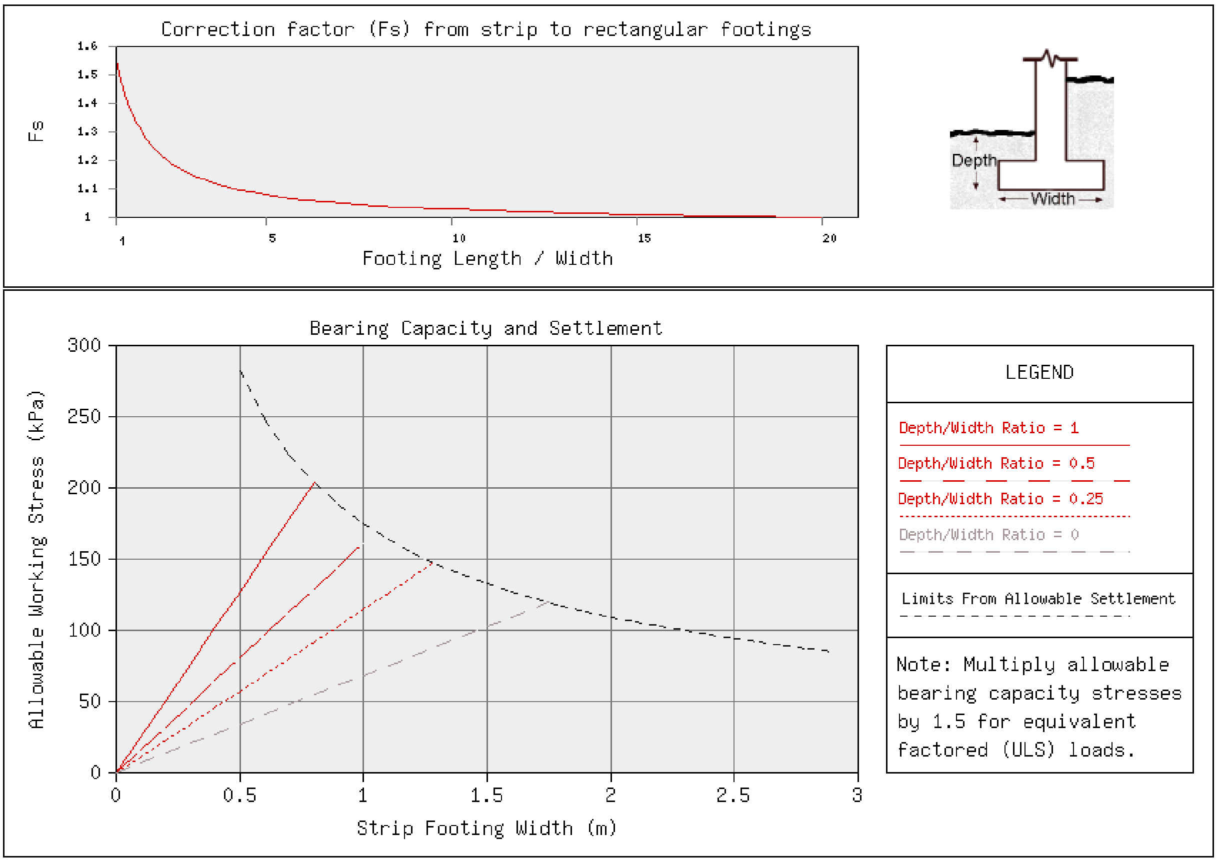


Figure 2. Recommended Bearing for Shallow Footings on Outwash Gravel and Engineered Fill overlying Outwash Gravel.

From Figure 2 it can be seen an allowable working stress of approximately 100 kPa is recommended for a 400 mm wide by 400 mm deep strip footing founded within outwash gravel and engineered fill overlying outwash gravel. This corresponds to a factored (ULS) bearing capacity of approximately 150 kPa and an ultimate geotechnical bearing capacity of 300 kPa.

Inspection and testing (dynamic probe/Scala penetrometers) should be completed along footing alignments during construction to confirm the above values are applicable and that the soil has not been softened by weather or excavation. Plate compaction or rolling is recommended following building platform and footing excavation.

6.10.1 Outwash Sand Bearing

Thin lenses of outwash sand have been observed in test pits. If substantial outwash sand is observed under a building platform bearing capacity should be assessed on a case by case basis.

6.11 Settlement

Settlement and differential settlement of shallow foundations are expected to be within structurally acceptable limits provided the recommendations of Section 6.10 are followed and all unsuitable materials, particularly those softened by water, are undercut and replaced with engineered fill during construction.

6.12 Site Subsoil Category

For detailed design purposes it is recommended the magnitude of seismic acceleration be estimated in accordance with the recommendations provided in NZS 1170.5:2004.

The site is “Class D” (Deep soil site) in accordance with NZS 1170.5:2004 seismic provisions.

6.13 QLDC Land Development and Subdivision Code of Practice

Section 2.2.4 of the QLDC Land Development and Subdivision Code of Practice (QLDC CoP) requires the developer of any subdivision to appoint a geo-professional to carry out the following functions from the planning to construction phases of the subdivision:

a) Check regional and district plans, records, and requirements prior to commencement of geotechnical assessment;
b) Prior to the detailed planning of any development, to undertake a site inspection and such investigations of subsurface conditions as may be required, and to identify geotechnical hazards affecting the land, including any special conditions that may affect the design of any pipelines, underground structures, or other utility services;
c) Before construction commences, to review the drawings and specifications defining any earthworks or other construction and to submit a written report to the Territorial Authority (TA) on the foundation and stability aspects of the project (if required);

d) Before and during construction, to determine the extent of further geo-professional services required (including geological investigation);
e) Any work necessary to manage the risk of geotechnical instability during the construction process;
f) Before and during construction, to determine the methods, location, and frequency of construction control tests to be carried out, determine the reliability of the testing, and to evaluate the significance of test results and field inspection reports in assessing the quality of the finished work;
g) During construction, to undertake regular inspection consistent with the extent and geotechnical issues associated with the project;
h) On completion, to submit a written report (i.e. Geotechnical Completion Report) to the Territorial Authority (TA) attesting to the compliance of the earthworks with the specifications and to the suitability of the development for its proposed use including natural ground within the development area. Where NZS 4431 is applicable, the reporting requirements of that Standard shall be used as a minimum requirement.

This resource consent level report can be considered to have completed items a) and b) from the above list. Once resource consent for the subdivision has been granted a geo-professional will need to be appointed by the developer to review the earthworks drawings and specifications prior to finalising the documentation for tendering and/or construction, and to oversee the construction phase of the project including certification of fill and provide a Geotechnical Completion Report (GCR) and Schedule 2A in accordance with the QLDC CoP.

The GCR and Schedule 2A should detail the results of site observations, testing and monitoring during earthworks construction, confirm the stability of the finished earthworks, and identify any specific geotechnical design requirements that must be addressed in order to construct a building on site. Any identified specific design requirements will then be registered on the subject lots' 'certificate of title' and will need to be addressed during the building consent process.

The geo-professional completing the GCR and Schedule 2A which includes the certification of fill should in all cases be engaged by the developer not the contractor. It is also advisable that the geo-professional review the earthworks contract to assist in managing the developers risk and ensuring that the contract is clear with respect to geotechnical risks and responsibilities during construction.

The use of this report and any of its findings or recommendations as part of the GCR and Schedule 2A may only be used with our prior review and written agreement.

7 Stormwater Disposal

7.1 Suitability of soil types

We understand that an on-site soakage system, in keeping with other developments nearby, will be adopted to manage stormwater at the site.

The geotechnical investigations identified that stormwater infiltration areas are located on a glacial outwash terrace that runs adjacent to Albert Town-Lake Hawea Road with the exception of soakage area one (SP1) where moderate depths of fill were observed, presumably associated with the historic construction of the adjacent highway.

Table 5. Suitability of soakage disposal based on soil type

Location	Stratigraphy	Suitability for Stormwater Disposal
SP1	1.8 m of fill and buried topsoil (SAND with some gravel and silt and organic SILT with minor rubbish) overlying outwash gravel and sand to base of pit.	Confirmed favourable from 1.8-2.6 m and 2.9-4.2 m (TP6 shows a 0.3 m thick sand lens at 2.6 m underlying outwash gravel). Soakage rate = 0.07 L/s/m^2
SP2	1.6 m of fill and buried topsoil (sandy GRAVEL and sandy organic SILT) overlying outwash gravel and sand to base of pit.	Confirmed favourable from 1.6-2.4 m and 2.9-4.2 m (TP6 shows 0.3 m thick sand lens at 2.6 m underlying outwash gravel). Soakage rate = 0.18 L/s/m^2
TP6	1.6 m of fill and buried topsoil (gravelly SAND and organic SILT) overlying outwash gravel with a 0.3 m thick gravelly SAND and SAND lens observed at 2.6 m.	Favourable from 1.6-2.6 m and 2.9-4.2 m depth. Will need to consider thin sand lens. No test completed in this test pit.
HT21 (1)	2.0 m of fill and buried topsoil (sandy GRAVEL and sandy organic SILT) overlying outwash gravel.	Favourable from 2 m (TP6 shows 0.3 m thick sand lens at 2.6-2.9 m depth in the outwash gravel). K (m/s) = 5 $x10^{-5}$
SP3	0.3 m of topsoil and loess overlying outwash gravel.	Favourable from 0.3 m. Soakage rate undetermined, water draining away faster than could put into test pit. Free draining.
SP4	0.3 m of topsoil and loess overlying outwash gravel.	Favourable from 0.3 m. Soakage rate undetermined, water draining away faster than could put into test pit. Free draining.
HT 21 (2)	0.3 m of topsoil and loess overlying outwash gravel.	Favourable from 0.3 m. K (m/s) = 2 $x10^{-4}$

7.2 Site testing

Standpipe field permeability testing and soakage testing of the outwash gravel was carried out at six field locations (see Site Plan, Appendix A and D for test locations and results respectively).

Four soakage pit tests and two standpipe field permeability tests were completed, all within the predominant sandy GRAVEL soils. It is important to note that the subordinate sand lenses will have significantly lower permeability than the gravels, possibly of the order of 1 x 10^{-5} m/sec which has likely influenced the testing in soakage area 1 and will affect long-term soakage rates.

Soakage testing was undertaken at the base of the soak pits in SP1-4. This was performed by introducing water from an 8,000L watercart until the water level of the pit reached the designated testing level. The inflow was then ceased and the time it took for the water level to drop recorded. The results were then analysed to determine indicative soakage rates, which are presented in Appendix D.

The standpipe field permeability test was undertaken using the HT21 methodology. Hydraulic conductivity was then obtained using published correlations (Van Hoorn, Glover, Phillip, HT21).

Table 6: Hydraulic Conductivity and Soakage Rate Values

Location	Test method	Output	Results
SP1	Open pit soakage test	Soakage Rate	0.07 L/s/m^2
SP2	Open pit soakage test	Soakage Rate	0.18 L/s/m^2
HT21 (1)	Standpipe field permeability test	Hydraulic Conductivity (K)	5x10^{-5} m/s
SP3	Open pit soakage test	Soakage Rate	Free draining*
SP4	Open pit soakage test	Soakage Rate	Free draining*
HT21 (2)	Standpipe field permeability test	Hydraulic Conductivity (K)	2 x10^{-4} m/s
*Insufficient water was able to be introduced to establish a pool of water at the base of the pit due to high soakage			

7.3 Infiltration design

Extensive permeability testing of outwash gravels was carried out for hydroelectric investigations in Upper Clutha Valley in the 1980s. This found typical bulk hydraulic conductivities (K) in outwash gravels, similar to those in soakage area 2 (SP3 and 4) of the proposed development at Mt Iron Junction, to be of the order of 4 x 10^{-4} m/s.

Standpipe field permeability HT21 (2) in outwash gravels within soakage area 2 found K=2 x 10^{-4} m/s which agrees well with the historic Upper Clutha Valley testing. HT21 (1), however, indicates lower than anticipated hydraulic conductivity (5×10^{-5} m/s) which is interpreted to be influenced by the underlying sand lens observed in TP6.

Estimation of a representative average hydraulic conductivity of the outwash soils is difficult, due to the limited number of tests and importance that geological variations (i.e. discrete minor sand lenses) can have on these values. The presence of lenses and layers of sand in the sequence will tend to lower the overall [bulk] hydraulic conductivity compared with that of the gravel component. The test pit logs indicate the sand lenses constitute only a small minority of the soil materials across the site.

However, a provisional estimate of the order of $K=2\times 10^{-4}$ m/s is considered reasonable for this unit, based on the site data and comparison with the known hydraulic conductivities of similar local outwash gravels. It is considered a value of 1×10^{-5} to 5×10^{-5} is suitable within the outwash gravel with sand lenses in soakage area 1. It is recommended that the infiltration zone is excavated to at least 3 m bgl in soakage area 1 to pass through the observed sand lens (TP6) and uncontrolled fill. This is anticipated to increase soakage potential, however confirmation that no extensive sand lenses are present is required during construction excavation inspections.

SP1 and SP2 also appear to have been influenced by the underlying outwash sand lens observed in TP6. SP1 and 2 returned an estimated soakage rate of 0.07 and 0.18 L/s/m^2 respectively.

Soakage pit testing in SP3 and SP4 was unable to establish a full test due to high soakage rates, in both cases, draining away all introduced after the hole was pre-soaked.

Table 7 presents the recommended soakage rate and hydraulic conductivity values[5] to be used for design. We recommend a reduction factor of at least 0.5 be applied to these values to allow for any loss of soakage performance over time.

Table 7: Summary of results in soakage areas 1 and 2

Location	Soakage Rate	Hydraulic Conductivity (K)
Soakage Area 1*	0.07 – 0.18 L/s/m^2	5×10^{-5} m/s
Soakage Area 2	Free Draining**	2×10^{-4} m/s
*Soakage Area 1 results likely influenced by sand lens observed from 2.6-2.9 m in TP6 **Water did not fill up bottom of soakage pit, draining away too fast		

Due to the uncertainties associated with soakage/permeability estimation and the importance that the value can have on design of soakage systems, we recommend that additional field tests (such as permeameter tests in 44 gallon drums) be conducted during construction to allow any necessary adjustments to be made to the design.

[5] It should be appreciated that estimation of soakage rates and hydraulic conductivity values utilize separate methods and hence cannot be balanced by unit conversion. We are happy to review our test results and provide alternative units (i.e. infiltration rates in mm/hr) if needed.

8 Neighbouring Structures/Hazards

Natural Hazards: Known seismic hazards affecting the development are detailed in Section 4.1 and appropriate allowance should be made for seismic loading during detailed design of the proposed building, foundations, and retaining walls. The development is not located within any mapped slope instability features, liquefaction susceptibility areas or any other hazard features on the QLDC or GeoSolve databases.

Liquefaction has been assessed using DPH testing, detailed in Section 5. Liquefaction risk is considered to be low due to the depth to groundwater and observed relative density of the site subsoils within the upper 11.5 m.

A rockfall hazard has been mapped within 70 m of the northwest boundary of the property on the QLDC hazard register. An assessment of the rockfall risk to this property has been completed and is detailed in section 6.7 of this report.

Seismic risk associated with the Cardrona Fault is detailed in Section 6.8.

Flooding has not been assessed as part of this assessment, the site is naturally free draining and the development is significantly higher than the closest body of flowing water that runs to the south of the site.

Distances to adjoining structures: It is assumed the existing buildings on site will be removed prior to construction and therefore no adverse geotechnical implications are expected to apply for neighbouring properties during construction provided appropriate vibration and dust mitigation measures are taken during construction. If existing buildings should remain onsite then the vibration effects should be considered if fill is to be compacted within 10 m of an existing structure.

Aquifers: No aquifer resource will be adversely affected by the development.

Erosion and Sediment Control: The site presents low potential to generate silt runoff during heavy rainfall events due to the predominately sub-horizontal topography and site geology. However if required effective systems for erosion control are runoff diversion drains and contour drains, while for sediment control, options are earth bunds, silt fences, hay bales, vegetation buffer strips and sediment ponds. Only the least amount of subsoil should be exposed at any stage and surfacing established as soon as practical. Details for implementation are given in Appendix B within the following link: http://esccanterbury.co.nz/.

Noise: It is expected that conventional earthmoving equipment, such as excavators, trucks and rollers will be required during construction. The earthworks contractor should take appropriate measures to control the construction noise, and ensure QLDC requirements are met in regard to this issue.

Dust: Regular dampening of soil materials with sprinklers to QLDC standards should be effective if required.

Vibration: No vibration induced settlement is expected in these soil types. The effects of vibrations from rollers and plate compactors on adjacent structures will need to be considered if fill is compacted within 10 m of an existing structure.

9 Conclusions and Recommendations

- The site is underlain by surficial topsoil and loess, which overlies outwash gravel with rare thin outwash sand lenses, which extends to at least 4.5 m beneath the surface of the proposed development. Isolated areas of uncontrolled fill were observed.
- Groundwater seepage was not observed during test pit investigations on the site completed to a maximum depth of 4.5 m. A piezometer was installed in close proximity DPH 1 to 6.7 m bgl and was dipped dry to full depth.
- No to minor liquefaction induced settlement (0-10 mm) is predicted across the site within the upper 10 m of the soil profile.
- No evidence of existing slope instability has been identified on site. Rockfall hazard is assessed as low risk and is detailed in section 6.7 of this report.
- Bearing on the site will be governed by the outwash gravel or engineered fill overlying outwash gravel. The outwash gravel and engineered fill will provide good bearing (100 kPa allowable), for 400 mm wide by 400 mm deep shallow footings.
- Recommendations for temporary and permanent batter slope angles are described in Table 4. Slopes that are required to be steeper than those described should be structurally retained or subject to specific geotechnical design.
- All retaining walls should be designed by a Chartered Professional Engineer using the geotechnical parameters recommended in Table 3 of this report.
- The outwash gravel soils are considered suitable for use as engineered fill (in accordance with an earthfill specification).
- All unsuitable soils identified in foundation excavations, particularly those softened by exposure to water, should be undercut and replaced with engineered fill during construction.
- Any fill that is utilised as bearing for foundations should be placed and compacted in accordance with NZS 4431:1989 and certification provided to that effect.
- For detailed design purposes it is recommended that the site is classified “Class D – Deep subsoil” in accordance with NZS 1170.5:2004 seismic provisions.
- Based on the geological conditions observed, testing data and experience with similar outwash gravel, the bulk permeability of the deposit is estimated to be in the order of 2 $\times 10^{-4}$ m/s in Soakage area 2. A lesser value of 1-5$\times 10^{-5}$ is recommended where sand lenses are present such as observed in Soakage area 1. Soakage rates are also provided in Table 7. To allow for any loss of soakage performance over time we recommend a reduction factor of at least 0.5 be applied to the value adopted in each of the two soakage areas for design purposes.
- A geotechnical practitioner should inspect all foundation excavations, batter slopes, soak pit excavations and additionally any seepage, spring flow or under-runners that may be encountered during construction.

10 Applicability

This report has been prepared for the benefit of Mt Iron Junction Limited with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

It is important that we be contacted if there is any variation in subsoil conditions from those described in this report.

Please don't hesitate to contact the undersigned if we can provide any further assistance with this project.

Report prepared by:

....	....
Mike Plunket	James Stewart
Geotechnical Engineer	Engineering Geologist

Reviewed for GeoSolve Ltd by:

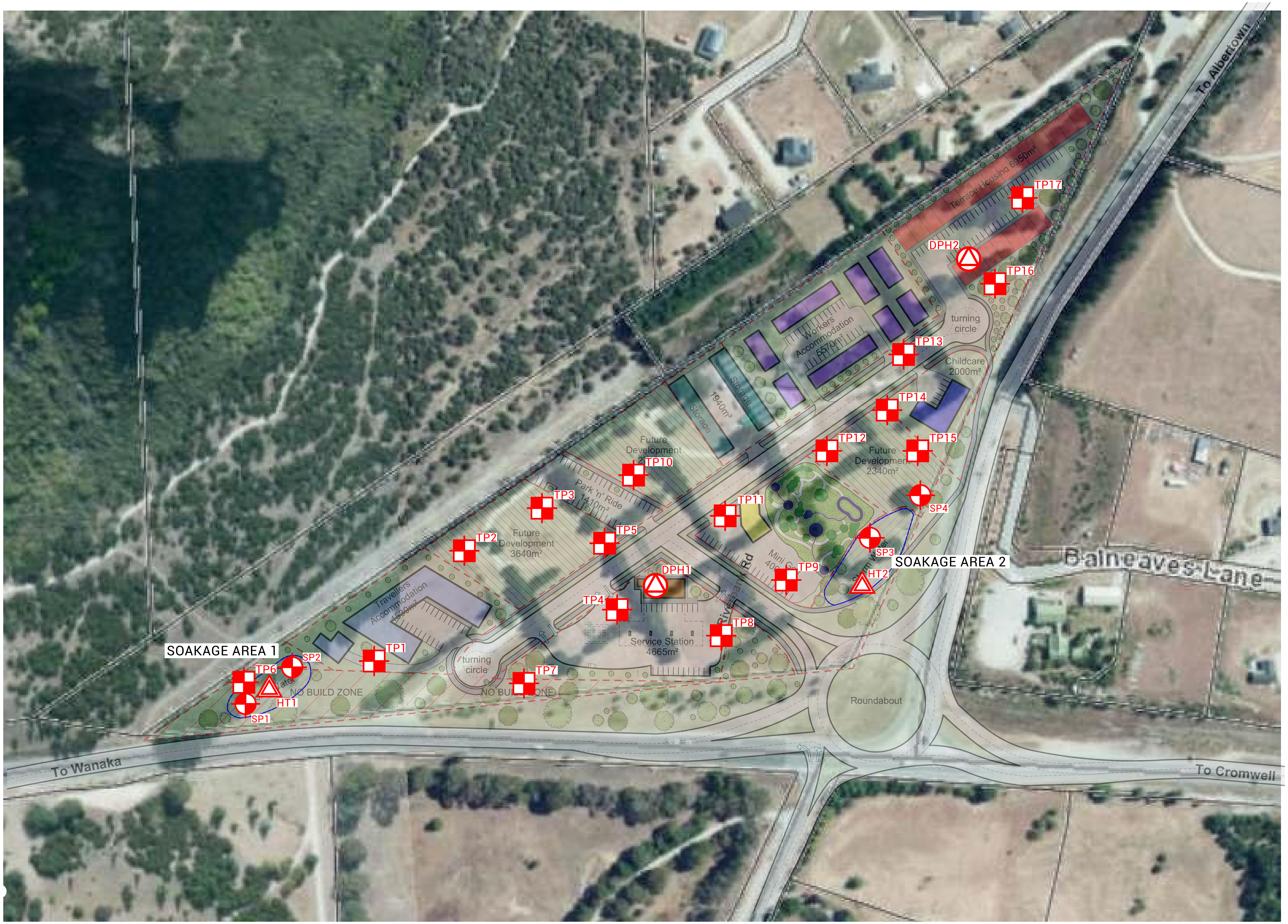
....

Fraser Wilson

Senior Engineering Geologist

GeoSolve Ltd

Appendix A: Site Investigation Plan



Key

TP1 = Test Pit

DPH1 = Heavy Dynamic Probe (DPH) test location

SP1 = Soakage Pit Test

HT1 = HT21 Permeability Test

Scale 1:2000

0 20 40 60 80 (m)

GEOSOLVE

70 Macandrew Road, PO Box 2427, South Dunedin 9044. ph 03 466 4024

DRAWN	MDP	2/18
DRAFTING CHECKED	FAW	2/18
APPROVED	FAW	2/18

FILE : PDF

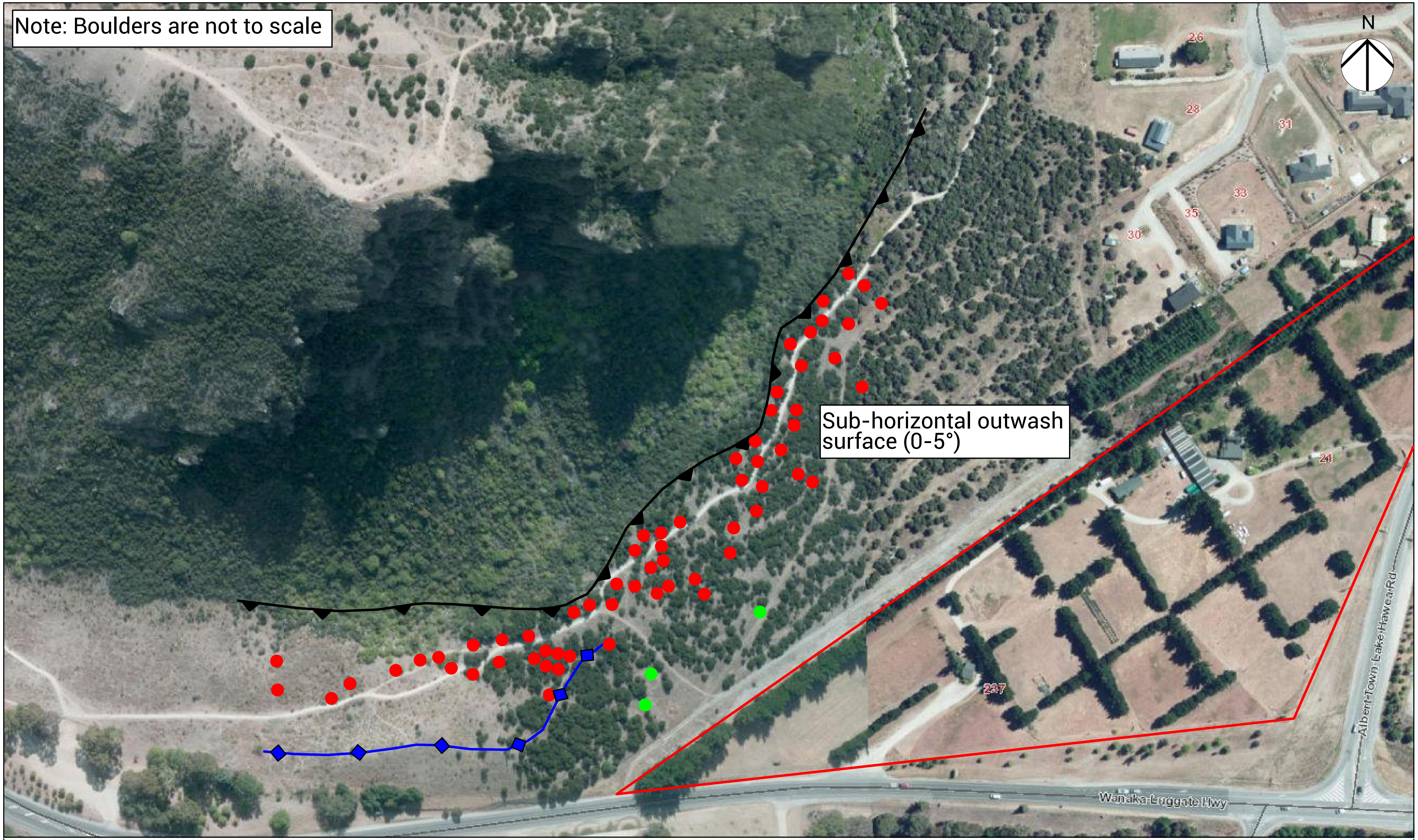
SCALE: (AT A3 SIZE) As Shown

PROJECT No. 170839

MT IRON JUNCTION LIMITED
Geotechnical Investigation
237 Wanaka-Luggate Highway
Site Plan

FIG No. Appendix A - Figure 1

REV. 0



Key
- = Observed rockroll boulder
- = Observed outwash boulder
- = Base of Mount Iron slopes
- = Crest of lateral moraine ridge
- = Property boundary

Scale 1:2000
0 20 40 60 80 (m)

GEOSOLVE

70 Macandrew Road, PO Box 2427, South Dunedin 9044. ph 03 466 4024

DRAWN	MDP	2/18
DRAFTING CHECKED	JAS	2/18
APPROVED	FAW	2/18

FILE : PDF
SCALE: (AT A3 SIZE) As Shown
PROJECT No. 170839

MOUNT IRON JUNCTION LTD
Geotechnical Investigation
237 Wanaka-Luggate Highway
Rockroll Hazard Map

FIG No. Appendix A - Figure 2
REV. 0

Appendix B: Investigation Data

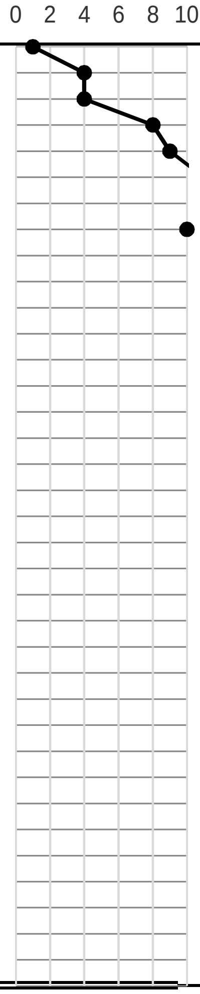
EXCAVATION LOG

EXCAVATION NUMBER:
TP 1

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm
0.15	TOPSOIL		Light brown/brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.2	OUTWASH GRAVEL		Brown/grey, sandy GRAVEL with trace cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 80 mm. Medium dense. Bedded. Dry.			
3.6	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 3.6 m

COMMENT:	Logged By: MDP
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	Sheet: 1 of 1

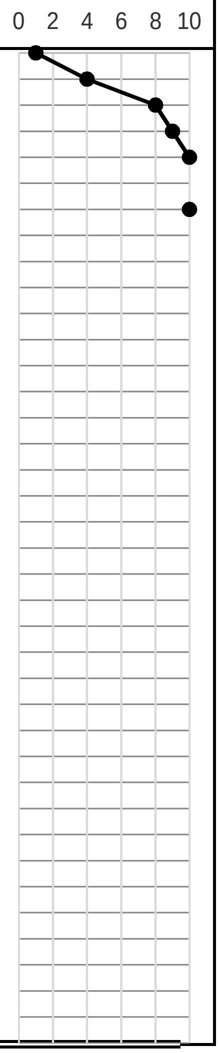
EXCAVATION LOG

EXCAVATION NUMBER:
TP 2

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm
0.15	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.3	OUTWASH GRAVEL		Grey, sandy GRAVEL with minor rootlets. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.			
3.8	OUTWASH GRAVEL		Grey, sandy GRAVEL with trace cobbles and boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 300 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 3.8 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

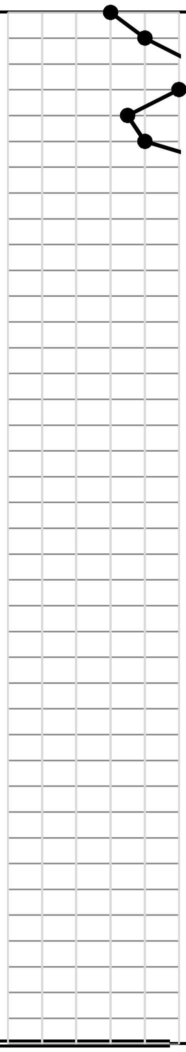
EXCAVATION LOG

EXCAVATION NUMBER: **TP 3**

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm 0 2 4 6 8 10
0.15	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.35	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
0.6	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense.			
1.5	OUTWASH SAND		Grey/brown, SAND with some to minor gravel. Sand is fine to coarse. Medium dense. Bedded. Dry.			
4.0	OUTWASH GRAVEL		Grey, sandy GRAVEL with minor cobbles and boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 300 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 4 m

COMMENT:	Logged By: MDP
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	Sheet: 1 of 1

GEOSOLVE

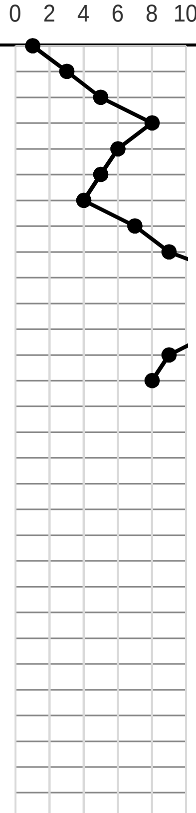
EXCAVATION LOG

EXCAVATION NUMBER:
TP 4

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm
0.2	TOPSOIL		Brown, organic SILT with roots and rootlets. Silt is non-plastic. Dry.			
0.5	LOESS		Light brown, silty SAND with minor gravel. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
3.0	OUTWASH GRAVEL		Grey, sandy GRAVEL with minor rootlets. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 3 m

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	Logged By: MDP
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	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **TP 5**

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.2	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
1.8	OUTWASH GRAVEL		Grey, sandy GRAVEL with minor rootlets. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.			
3.5	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	

Total Depth = 3.5 m

COMMENT:	Logged By: MDP
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	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER:

TP 6

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
1.3	UNCONTROLLED FILL		Grey, gravelly SAND with minor topsoil and trace sticks, rootlets and wire. Sand is fine to coarse. Gravel is fine to medium. Gravel is sub-angular to sub-rounded. Loose. Dry.			
1.6	BURIED TOPSOIL		Brown, organic SILT with rootlets and roots. Silt is non-plastic. Dry.			
2.6	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.			
2.9	OUTWASH SAND		Grey/brown, gravelly SAND to SAND with some gravel. Sand is fine to coarse. Medium dense. Bedded. Dry.			
4.2	OUTWASH GRAVEL		Grey, sandy GRAVEL with some cobbles and minor boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 300 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	

Total Depth = 4.2 m

COMMENT:	Logged By: MDP
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	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER:
TP 7

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.15	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
2.3	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.			
4.0	OUTWASH GRAVEL		Dark grey, sandy GRAVEL with minor cobbles and boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 250 mm. Medium dense. Bedded. Dry to moist.		NO SEEPAGE	

Total Depth = 4 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **TP 8**

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
2.6	OUTWASH GRAVEL		Grey, sandy GRAVEL with minor rootlets, cobbles and boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 200 mm. Medium dense. Bedded. Dry.			
3.8	OUTWASH GRAVEL		Dark grey, sandy GRAVEL with minor cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 90 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	

Total Depth = 3.8 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

GEOSOLVE

EXCAVATION LOG

EXCAVATION NUMBER:
TP 9

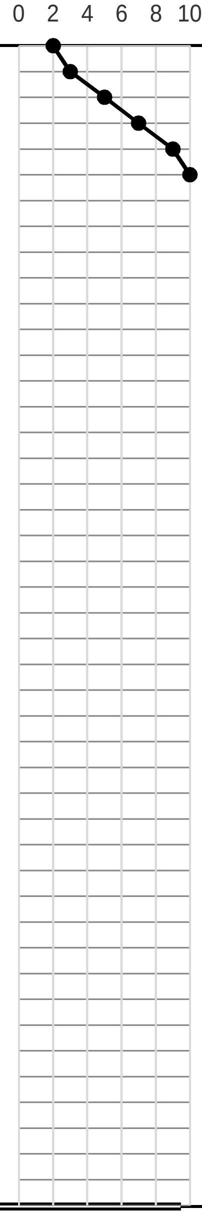
PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.		
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.		
1.3	OUTWASH GRAVEL		Grey, sandy GRAVEL with some cobbles and minor boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 200 mm. Medium dense. Bedded. Dry.		
4.5	OUTWASH GRAVEL		Dark grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE

SCALA PENETROMETER

Blows per 100mm



Total Depth = 4.5 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **TP 10**

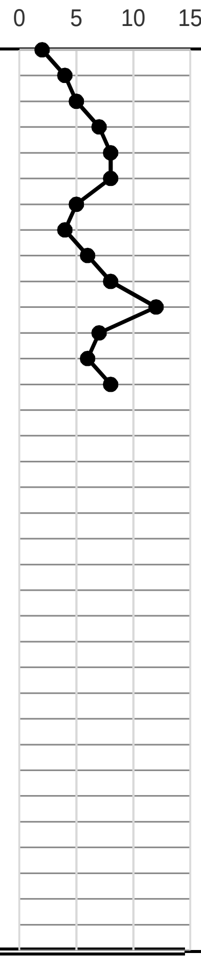
PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.		
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.		
2.4	OUTWASH GRAVEL		Brown/grey, sandy GRAVEL with minor rootlets and trace cobbles and roots. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 80 mm. Medium dense. Bedded. Dry.		
3.5	OUTWASH GRAVEL		Brown/grey, sandy GRAVEL with some to minor cobbles and minor boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 300 mm. Medium dense. Bedded. Dry.		NO SEEPAGE

SCALA PENETROMETER

Blows per 100mm



Total Depth = 3.5 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER:
TP 11

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.1	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.			
2.5	OUTWASH GRAVEL		Grey, sandy GRAVEL with some to minor cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 100 mm. Medium dense. Bedded. Dry.			
2.9	OUTWASH SAND		Grey/brown, gravelly SAND. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-rounded to sub-angular. Medium dense. Bedded. Dry.			
3.4	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	

Total Depth = 3.4 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

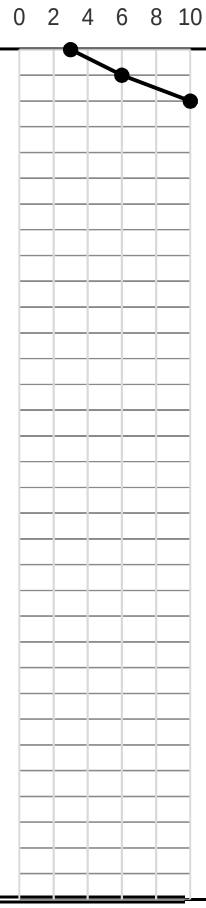
EXCAVATION LOG

EXCAVATION NUMBER: **TP 12**

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm
0.1	TOPSOIL		Brown, organic SILT with rootlets and roots. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets and trace roots. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.9	OUTWASH GRAVEL		Grey, sandy GRAVEL with some cobbles, minor rootlets and trace boulders. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Boulders to 300 mm. Medium dense. Bedded. Dry.			
3.3	OUTWASH GRAVEL		Dark grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 3.3 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

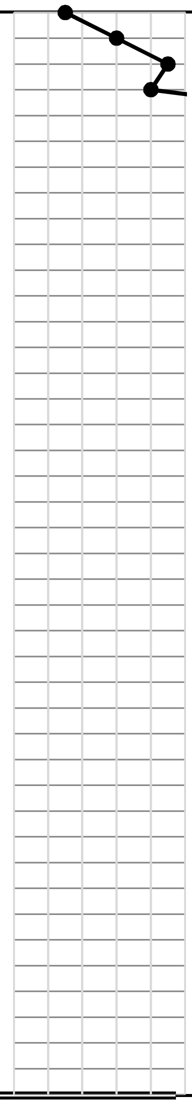
GEOSOLVE

EXCAVATION NUMBER:
TP 13

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm (0 2 4 6 8 10)
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
2.5	OUTWASH GRAVEL		Grey/brown, sandy GRAVEL with some cobbles and trace rootlets. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 100 mm. Medium dense. Bedded. Dry.			
4.2	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 4.2 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **TP 14**

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.2	OUTWASH GRAVEL		Grey, sandy GRAVEL with some cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 100 mm. Medium dense. Bedded. Dry.			
3.1	OUTWASH GRAVEL		Grey, sandy GRAVEL with minor to trace cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 100 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	

Total Depth = 3.1 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

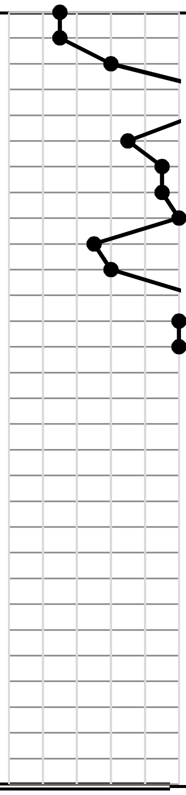
EXCAVATION LOG

EXCAVATION NUMBER:
TP 15

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm 0 2 4 6 8 10
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.3	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.6	OUTWASH GRAVEL		Grey/brown, sandy GRAVEL with minor cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 100 mm. Medium dense. Bedded. Dry.			
3.0	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 3 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

GEOSOLVE

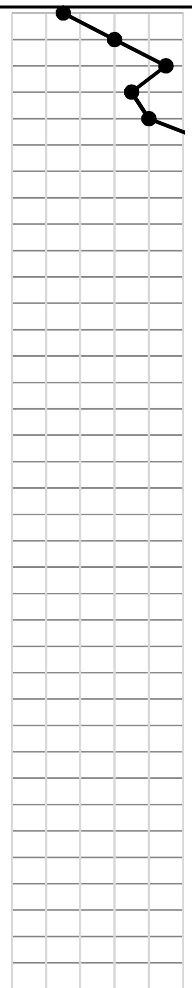
EXCAVATION LOG

EXCAVATION NUMBER:
TP 16

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER Blows per 100mm 0 2 4 6 8 10
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.2	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
3.7	OUTWASH GRAVEL		Grey, sandy GRAVEL with trace rootlets and cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 200 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	



Total Depth = 3.7 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER:
TP 17

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	19-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	19-Dec-17

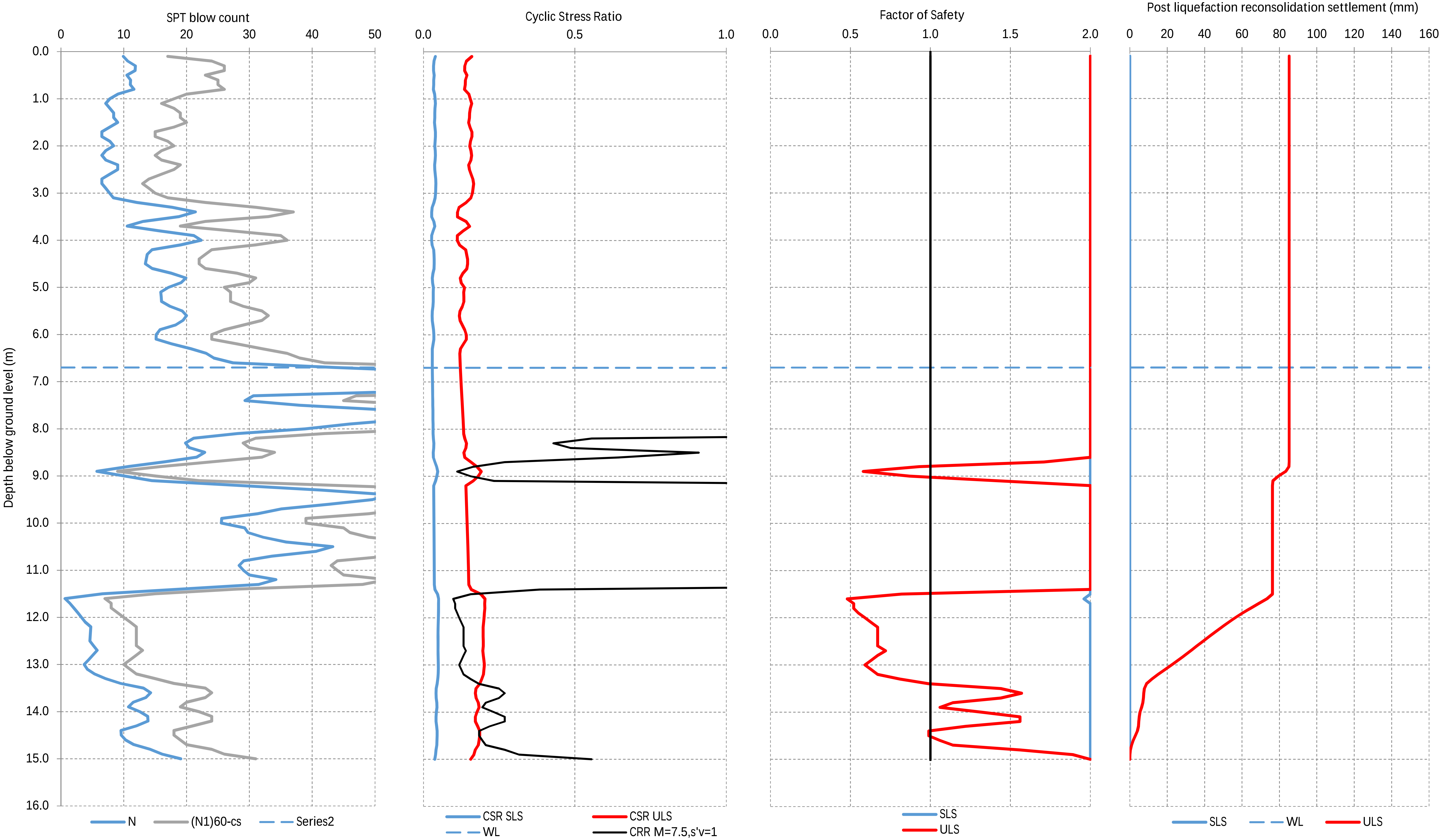
DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.1	TOPSOIL		Brown, organic SILT with rootlets. Silt is non-plastic. Dry.			
0.2	LOESS		Light brown, silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Dry.			
1.9	OUTWASH GRAVEL		Grey, sandy GRAVEL with trace rootlets and cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 200 mm. Medium dense. Bedded. Dry.			
3.5	OUTWASH GRAVEL		Grey/brown, sandy GRAVEL with minor cobbles. Sand is fine to coarse. Gravel is fine to coarse. Gravel is sub-angular to sub-rounded. Cobbles to 200 mm. Medium dense. Bedded. Dry.		NO SEEPAGE	

Total Depth = 3.5 m

COMMENT:	Logged By: MDP
	Checked Date:
	Sheet: 1 of 1

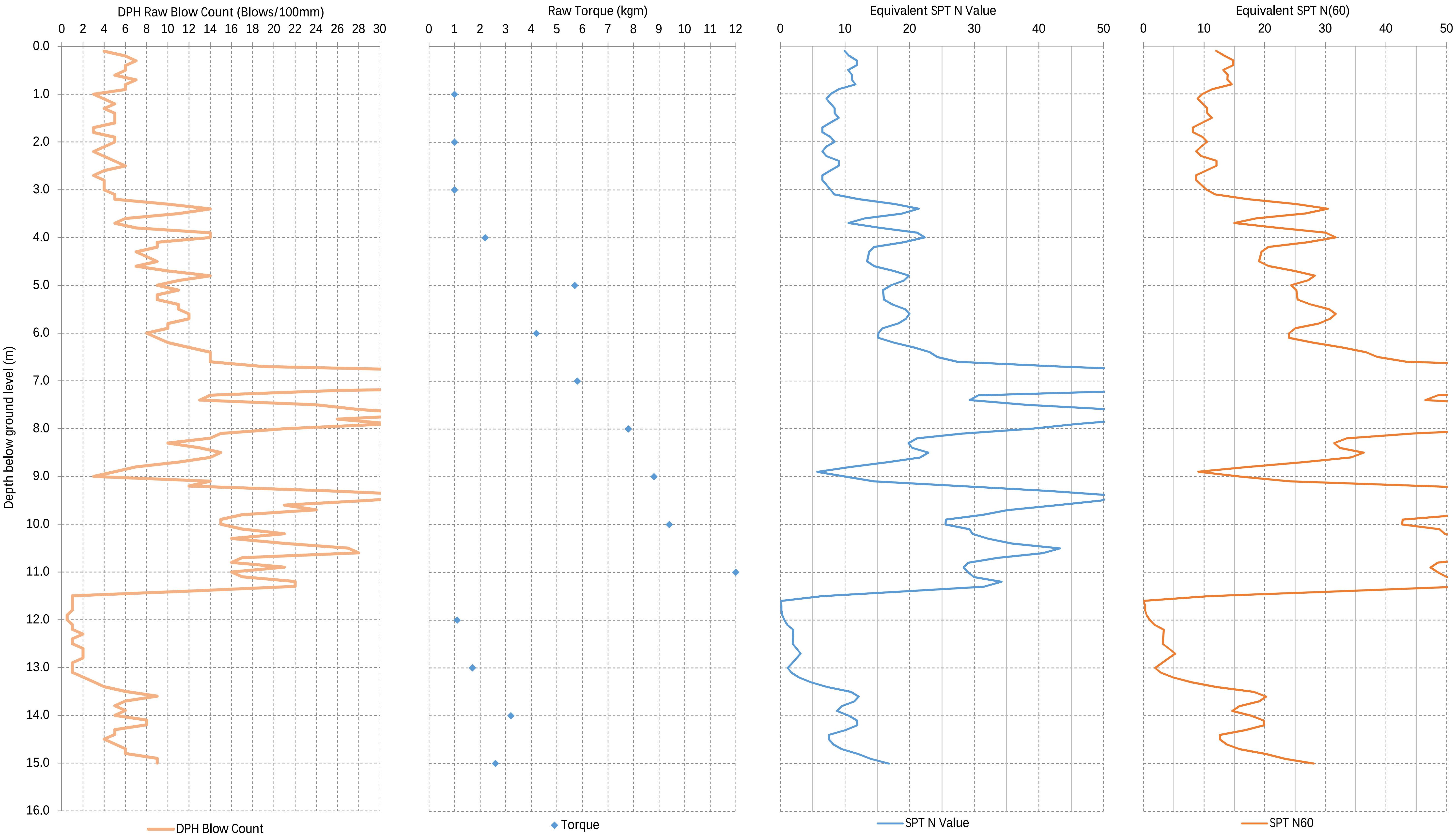
Appendix C: Liquefaction Analysis

LSN (SLS)	LSN (ULS)	Crust Thickness (SLS)	Crust Thickness (ULS)
0.0	7.1	0.0	8.7

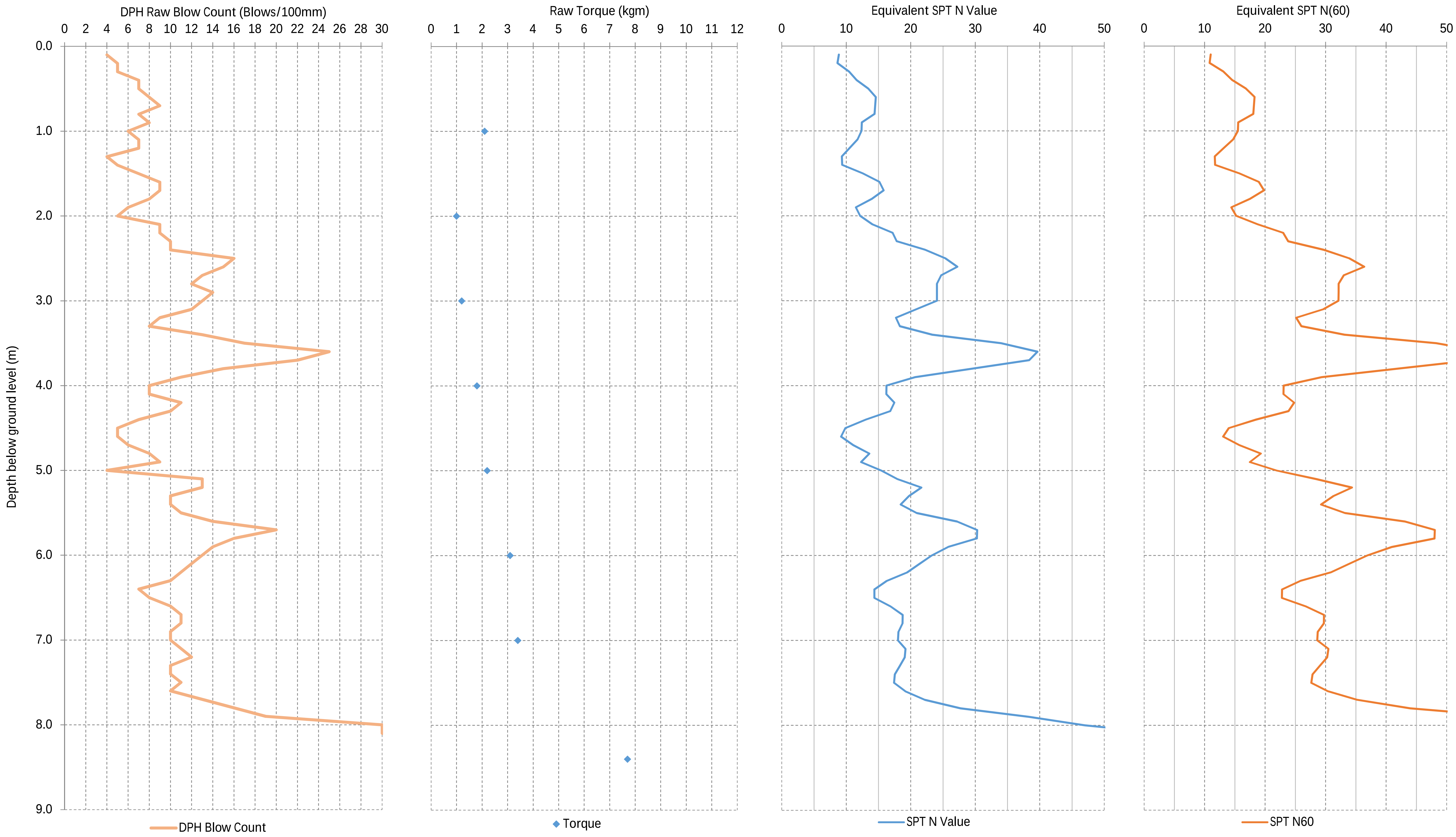


Note: Settlements as per Idriss and Boulanger (2014)

GEOSOLVE	PROJECT	170839 - Mt Iron Junction	CLIENT	Mt Iron Junction Limited	LOGGED BY	RC
	DESCRIPTION	Liquefaction Analysis	TEST NUMBER	DPH1	ANALYSED BY	MDP
	LOCATION	Wanaka	DATE	17/01/2018	CHECKED BY	FAW



GEOSOLVE						
	PROJECT	170839 - Mt Iron Junction	CLIENT	Mt Iron Junction Limited	LOGGED BY	RC
	DESCRIPTION	Liquefaction Analysis	TEST NUMBER	DPH1	ANALYSED BY	MDP
	LOCATION	Wanaka	DATE	17/01/2018	CHECKED BY	FAW



GEOSOLVE	PROJECT	170839 - Mt Iron Junction	CLIENT	Mt Iron Junction Limited	LOGGED BY RC
	DESCRIPTION	Liquefaction Analysis	TEST NUMBER	DPH2	ANALYSED BY MDP
	LOCATION	Wanaka	DATE	17/01/2018	CHECKED BY FAW

Appendix D: Permeability Testing Results

EXCAVATION LOG

EXCAVATION NUMBER:
SP 1

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
1.7	UNCONTROLLED FILL		Light grey, SAND with some gravel and silt, trace cobbles and boulders. Sand is fine to coarse. Gravel is fine to coarse and sub-rounded. Boulders up to 700 mm diameter. Loose to medium dense. Massive. Dry.			
1.8	BURIED TOPSOIL		Brown, organic SILT with minor rubbish. Rubbish includes wire fence. Firm. Massive. Dry.			
2.6	OUTWASH GRAVEL		Grey, sandy GRAVEL with lenses of gravelly SAND and silty SAND. Sand is fine to coarse. Gravel is fine to coarse and sub-rounded. Medium dense. Bedded. Moist.		NO SEEPAGE	

Total Depth = 2.6 m

COMMENT:	Logged By: JAS/MDP
	Checked Date:
	Sheet: 1 of 1

GEOSOLVE

EXCAVATION LOG

EXCAVATION NUMBER: **SP 2**

PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

EASTING:	mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:	mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:	m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:		EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
1.2	UNCONTROLLED FILL		Light grey, sandy GRAVEL with trace cobbles and organic SILT. Sand is fine to coarse. Gravel is fine to coarse and sub-rounded. Loose to medium dense. Massive. Dry.			
1.6	BURIED TOPSOIL		Dark brown, sandy organic SILT with minor gravel. Sand is fine. Gravel is fine to coarse. Firm. Massive. Moist.			
2.4	OUTWASH GRAVEL		Grey, sandy GRAVEL with trace cobbles. Sand is fine to coarse. Gravel is fine to coarse and sub-rounded. Medium dense. Bedded. Moist.		NO SEEPAGE	

Total Depth = 2.4 m

COMMENT:	Logged By: JAS/MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **SP 3**

PROJECT:	Mt Iron Junction		JOB NUMBER:	170839

EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.3	TOPSOIL/LOESS		Brown, organic SILT and silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Massive. Dry.		NO SEEPAGE	
1.3	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse and sub-rounded to sub-angular. Medium dense. Bedded. Dry.			

Total Depth = 1.3 m

COMMENT:	Logged By: JAS/MDP
	Checked Date:
	Sheet: 1 of 1

EXCAVATION LOG

EXCAVATION NUMBER: **SP 4**

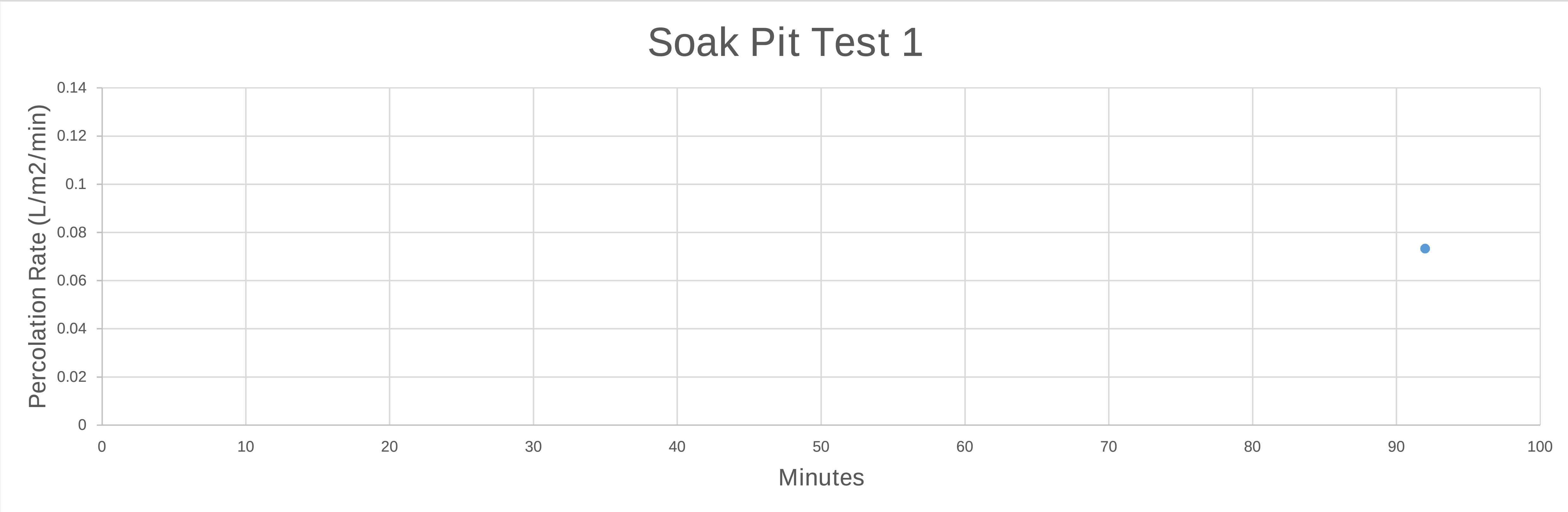
PROJECT:	Mt Iron Junction	JOB NUMBER:	170839

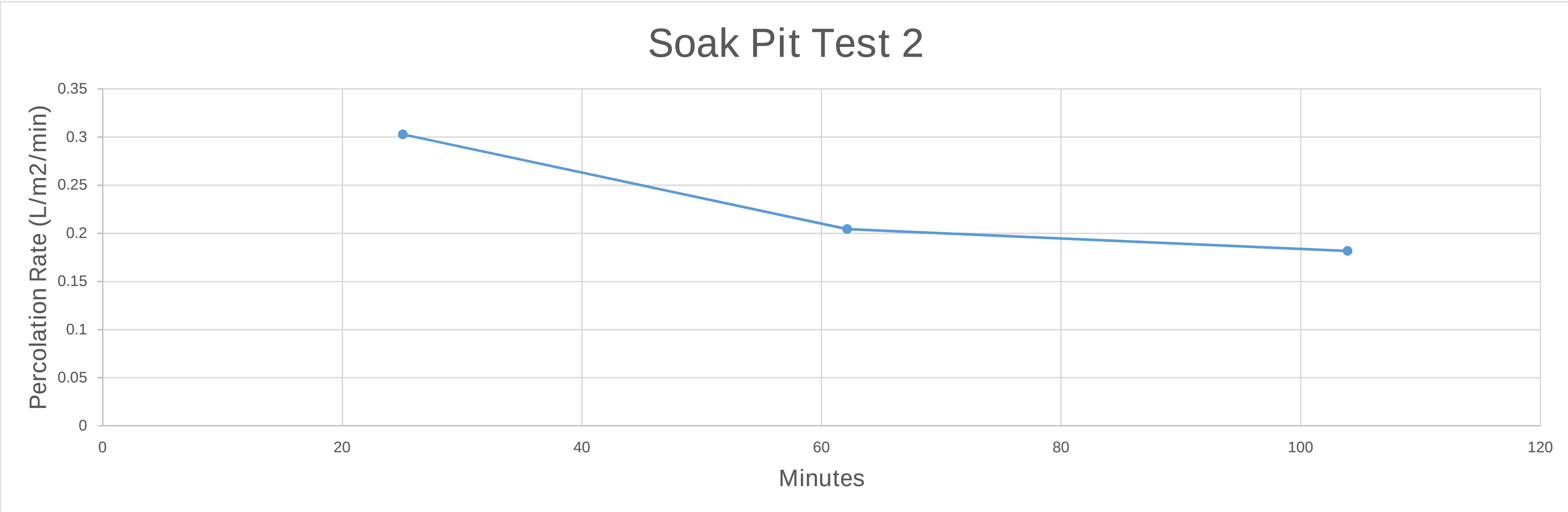
EASTING:		mE	EQUIPMENT:	8 T Excavator	OPERATOR:	Ben
NORTHING:		mN	INFOMAP NO.		COMPANY:	Diverse Works
ELEVATION:		m	DIMENSIONS:		HOLE STARTED:	18-Dec-17
METHOD:			EXCAV. DATUM:		HOLE FINISHED:	18-Dec-17

DEPTH (m)	SOIL / ROCK TYPE	GRAPHIC LOG	DESCRIPTION	USCS GROUP	GROUNDWATER / SEEPAGE	SCALA PENETROMETER
0.3	TOPSOIL/LOESS		Brown, organic SILT and silty SAND with minor rootlets. Sand is fine to medium. Silt is non-plastic. Firm. Massive. Dry.		NO SEEPAGE	
1.2	OUTWASH GRAVEL		Grey, sandy GRAVEL. Sand is fine to coarse. Gravel is fine to coarse and sub-rounded to sub-angular. Medium dense. Bedded. Dry.			

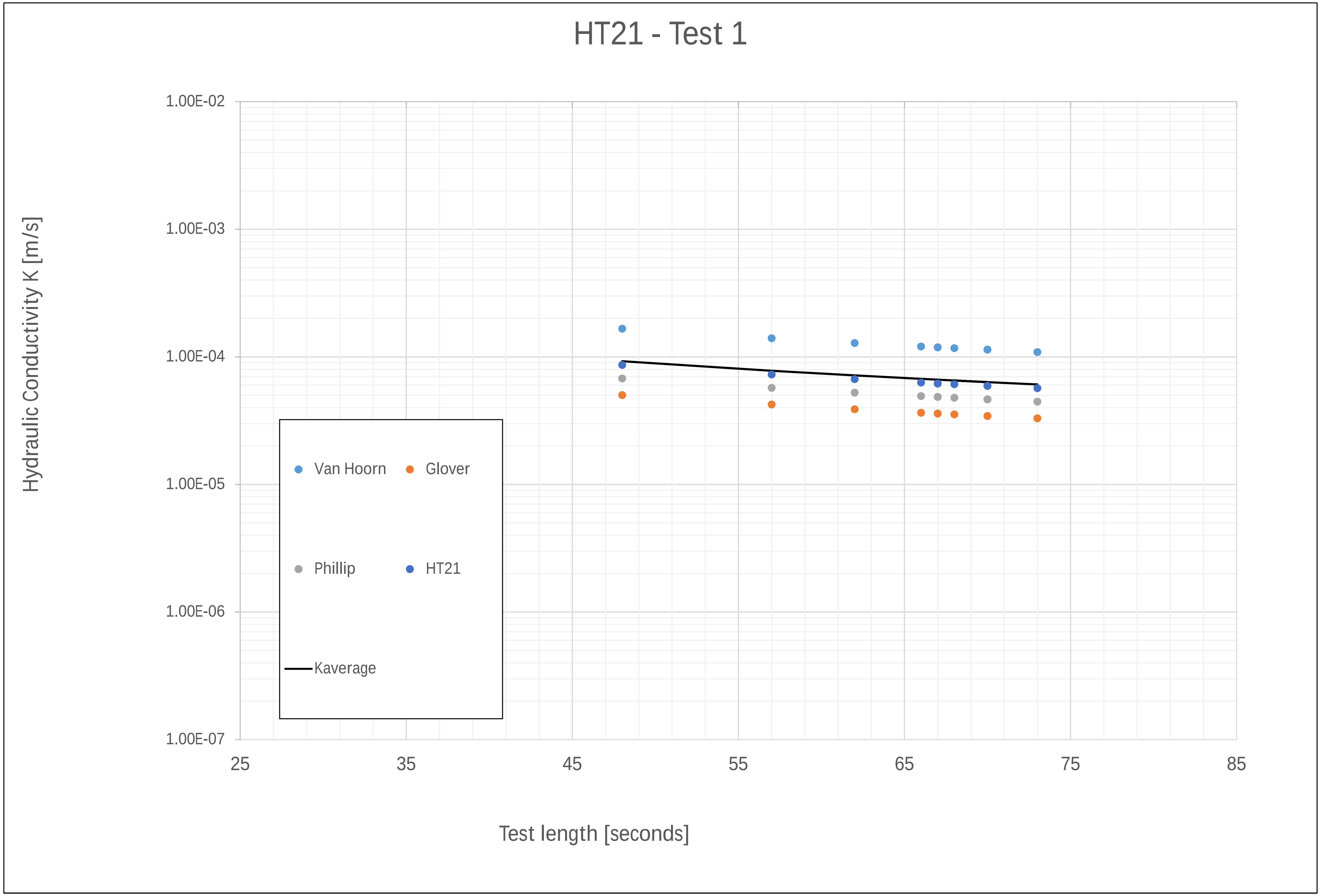
Total Depth = 1.2 m

COMMENT:	
	Logged By: JAS/MDP
	Checked Date:
	Sheet: 1 of 1

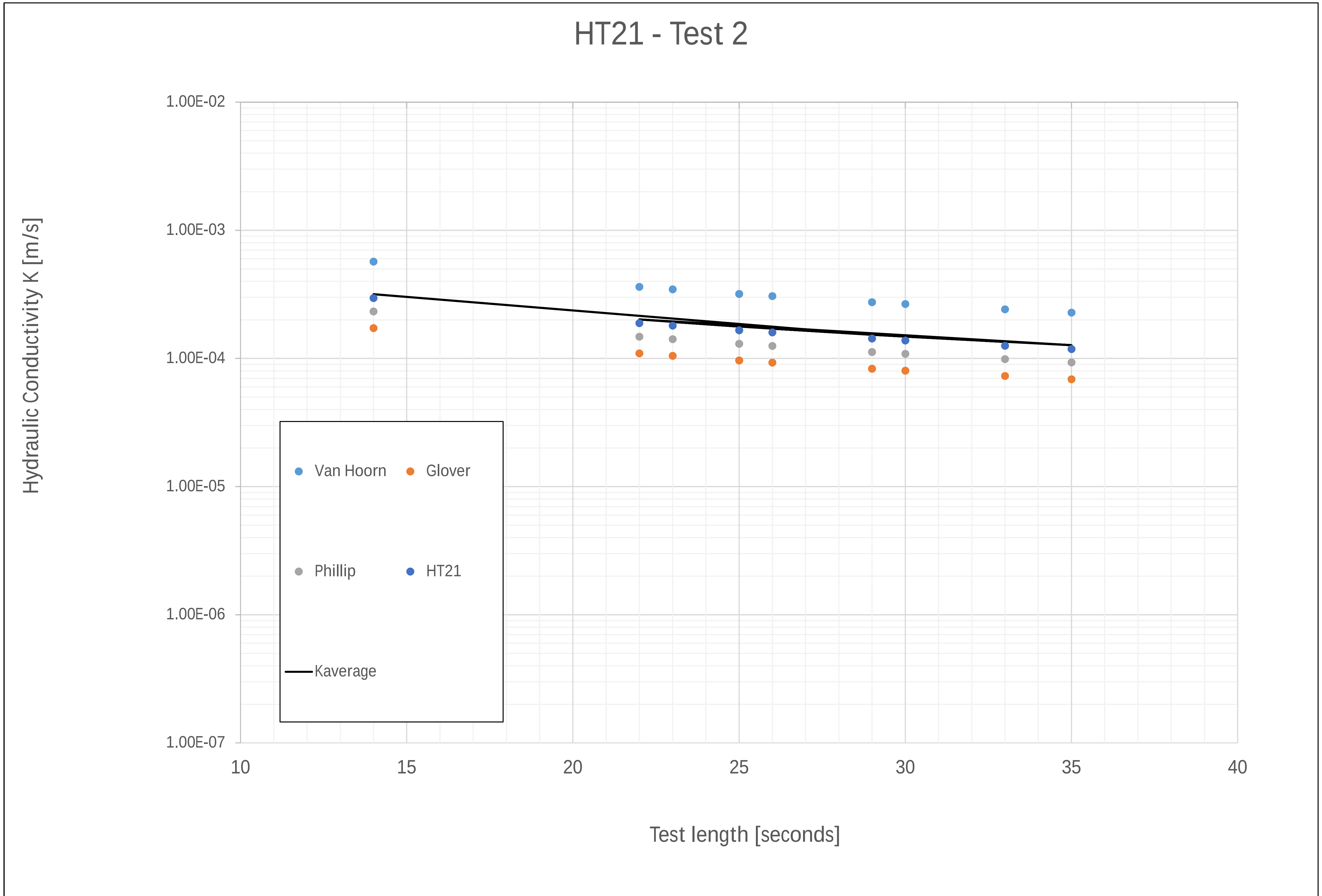




GEOSOLVE	PROJECT DESCRIPTION LOCATION	170839 - Mt Iron Junction Peremability Testing Wanaka	CLIENT TEST NUMBER DATE	Mt Iron Junction Limited SP 1 and 2 18/12/2017	LOGGED BY ANALYSED BY CHECKED BY	JAS/MDP MDP FAW



GEOSOLVE	PROJECT	170839 - Mt Iron Junction	CLIENT	Mt Iron Junction Limited	LOGGED BY	JAS/MDP
	DESCRIPTION	Peremability Testing	TEST NUMBER	HT21 1	ANALYSED BY	MDP
	LOCATION	Wanaka	DATE	18/12/2017	CHECKED BY	FAW



GEOSOLVE	PROJECT DESCRIPTION LOCATION	170839 - Mt Iron Junction Peremability Testing Wanaka	CLIENT TEST NUMBER DATE	Mt Iron Junction Limited HT21 2 18/12/2017	LOGGED BY JAS/MDP ANALYSED BY MDP CHECKED BY FAW

APPENDIX C STORMWATER REPORT

STORMWATER MODELLING REPORT

Mount Iron Junction

Client: Mt Iron Junction Ltd

Patersons Ref	**P240103**
Date	19/02/2026

Version	Date	Prepared by	Reviewed by	Comments
A	19/02/2026	Kuang-Chi Chu	Alan Hopkins	For Fast Track Consent (Substantial)

Table of Contents

Executive Summary

This report presents the preliminary stormwater modelling and design assessment for the proposed Mt Iron Junction development at Mountain and Junction Roads, Wānaka. The objective of this report is to demonstrate that the proposed stormwater management system achieves adequate containment and disposal of both primary (5 % AEP) and secondary (1 % AEP) storm events and no increase in offsite effects, consistent with the Queenstown Lakes District Council (QLDC) Land Development and Subdivision Code of Practice (2025).

Stormwater modelling has been completed using HEC-HMS v4.12, based on NIWA HIRDS V4 RCP 8.5 (2081–2100) rainfall intensities and temporal distributions. The analysis confirms that the proposed soakage and storage facilities can effectively contain and infiltrate design flows as parameterised below:

Key design parameters and outcomes:

- **Design basis:** QLDC CoP (2025) and TP108 methodology
- **Model:** HEC-HMS v4.12 hydrologic simulation
- **Design infiltration rate:** 666 mm/hr (tested 2,000 mm/hr with factor of safety = 3)
- **Infiltration mechanism:** 0.95 void ratio proprietary stormwater soakage and attenuation device (to be confirmed during detailed design).
- **Combined capacity:** Meets 1 % AEP (RCP 8.5 period 2081-2100) containment requirements
- Full stormwater drain down time within 24 hours after storm end.

The modelling confirms that the proposed system provides sufficient capacity to manage stormwater within the development footprint, with flexibility retained for refinement during detailed design. Site-specific infiltration testing is required prior to detailed design to verify infiltration performance and finalise basin geometry.

This report also confirms the location and suitability of emergency and super design event flow paths that will convey flows via the localised road network to low points and the existing off- site state highway flow paths.

1. Introduction

This stormwater modelling has been conducted for Mt Iron Junction Ltd to support a substantive application for fast-track approval for the residential development and ancillary commercial uses proposed for Lots 2 and 6 DP 605028 and Lot 3 DP 359869.

HEC-HMS hydrologic analysis has been used to size soakage infrastructure to dispose of stormwater up to (and including) the 1% AEP RCP 8.5 for the period 2081-2100 storm.

Design not covered in this report includes detailed design, stormwater treatment selection, specific design levels, earthworks modelling, and soak pit product specification.

1.1 The Site

The proposed development area (*Figure 1*) is legally described as Lots 2 and 6 DP 605028 and Lot 3 DP 359869 and comprises of 5.98 ha of land, or thereabouts. The north-west boundary of the site is bound by the Mount Iron reserve and an existing residential development to the north-east. The south the site is bounded by State Highway 84 (Wanaka – Luggate Highway) and state Highway 6; These highways converge at the middle south end of the site at the recently constructed five-legged roundabout.

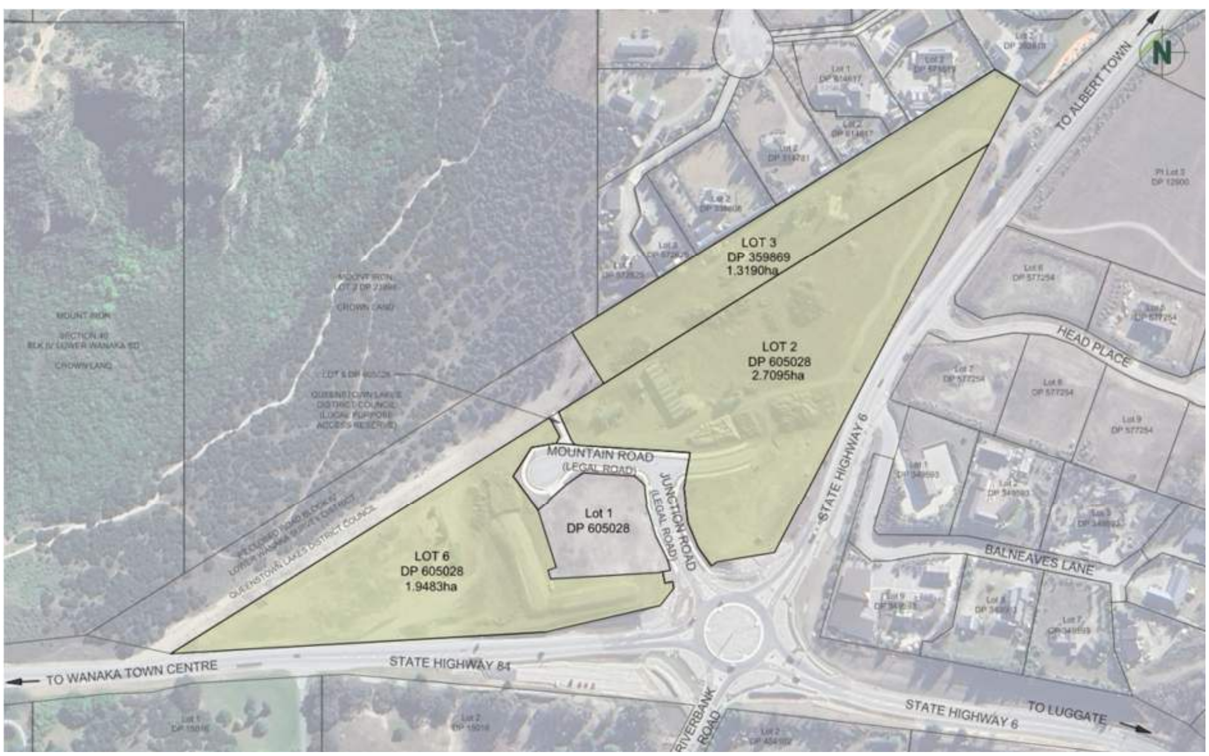


Figure 1 Location and general site context.

1.2 Referenced Documents

Table 1: Supporting documentation.

Title	Consultant	Appendix Ref	Date of Issue
Geotechnical Report – 237 Wanaka-Luggate Highway, Wanaka	Geosolve	See Infrastructure Report	March 2018

1.3 Predevelopment Site Condition and Overland Flow Paths

The existing catchments within and impacting the development site are shown in *Figure 2* below.

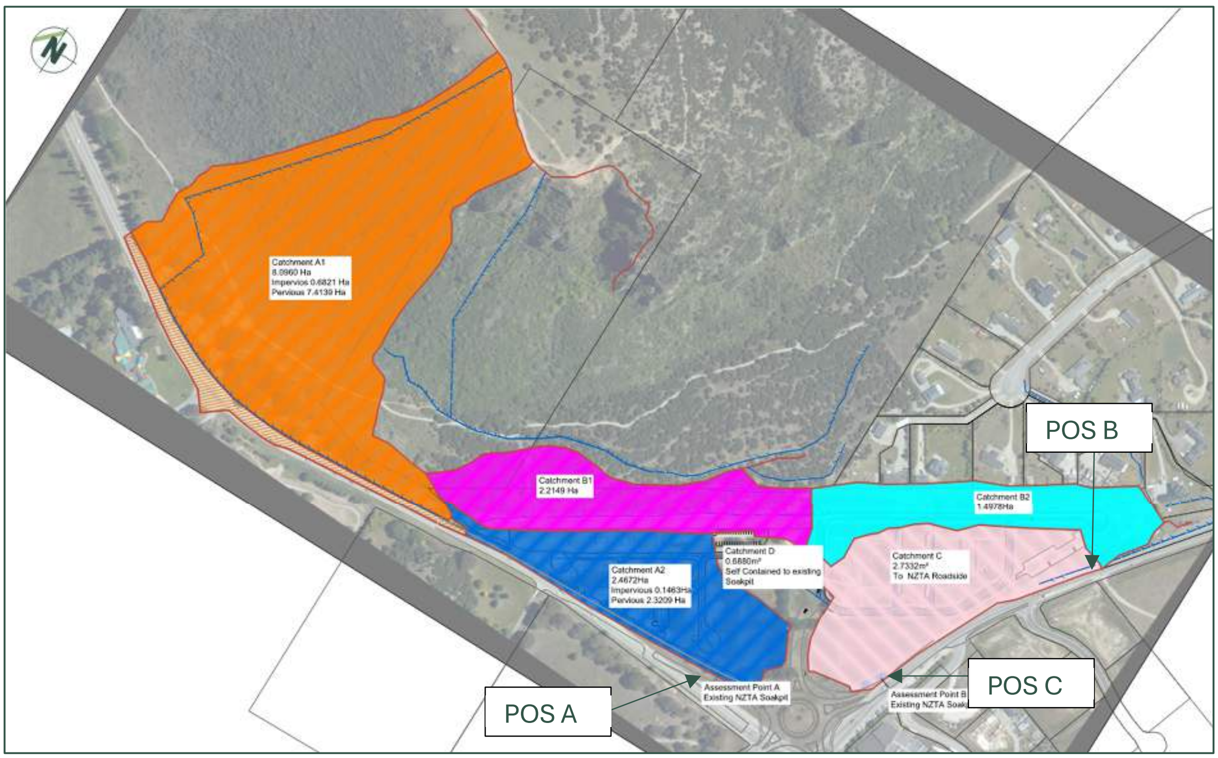


Figure 2: Existing predevelopment catchments impacting Mt Iron Junction development.

The subject site has been cleared of significant vegetation and is generally flat open ground with rough grass cover. The external contributing catchment B1 to the north contain undulating ground that generally falls gradually to the east and consists of poor rough grass and weed ground cover with areas of Kanuka scrub.

Predevelopment the south-western portion of the subject site, referred to as Catchment A2 (*dark blue*), falls gradually to the south towards the state highway water table and associated rock filled soak pit located north-west of the Mt Iron junction roundabout. This existing NZTA soak pit is referenced as Point of Study (POS) A.

Catchment A1 located to the north-west of the subject site currently drains via a flow path and state highway swale adjacent to Catchment A2 towards the Existing NZTA soak pit at Point of Study (POS) A.

This catchment does not enter the subject development site and therefore has been excluded from any modelling or associated assessment.

The eastern portion of the subject site, referred to as Catchments C (*pink*), falls gradually to the south-east towards the state highway water table and associated rock filled soak pit located north of the Mt Iron junction roundabout. This existing NZTA soak pit is referenced as Point of Study (POS) C.

The external catchment between the subject site and the base of Mt Iron referred to as Catchment B1(*magenta*), and a north-eastern strip within the subject (Lot 3 DP359869) referred to as Catchment B2 (*aqua*), fall very gradually through undulating ground to the north-east in the vicinity of the QLDC wastewater pumpstation. These flows join the State Highway 6 water table and flow north-east at a point referenced as Point of Study (POS) B.

The existing limited portion of Mountain Road and Junction Road that come off the fifth arm of the roundabout, and Lot 1 DP 605028 are identified as Catchment D. This catchment is considered outside the current development and excluded from this stormwater assessment. This area is understood to go to an existing contained soak pit sized for the 1% AEP event.

Flows exiting the site can be grouped into POS A-C, where A & C consist of existing NZTA soak pits which infiltrate stormwater into the ground, and POS C consists of a discharge road swale which conveys stormwater eastward, flowing parallel to SH 6. If either of the two state highway soak pits (POS A & C) are overwhelmed, they will head up and spill over the state highway carriageway and flow south to the Cardrona River.

2. Stormwater Design

The total combined internal and external catchment areas associated with the development are below 10ha. However, the proposed design solution utilises soakage and potentially some storage components and therefore as required under Section 4.3.5.1.4 of the QLDC COP, the stormwater modelling design for predevelopment and post development scenarios were conducted using HEC-HMS v4.12, following Auckland Regional Council's *TP108 – Guidelines for Stormwater Runoff Modelling in the Auckland Region*.

As the emergency (or tertiary) situation relies on the carrying capacity of the post development overland flow paths only with no soakage/storage components, and the total contributing catchments are less than 10ha, this scenario has been assessed using the Rational Method only.

Given the lack of existing QLDC stormwater pipe networks in the area, and limited capacity within the existing State Highway water table drains and soak pits, it is proposed to dispose of all primary and secondary flows i.e. the 100-year (1 % AEP) event from the subject site and associated contributing catchments to ground via on-site soakage. To enable this to occur in the most efficient manner possible, the sumps and localised pipe networks will be upsized to convey both the primary and secondary events. The road carriageways and any associated overland flow paths are designed to convey the 'emergency' event, which is taken as being full failure of the pipe network and associated soak pits up to the 100 year (1 % AEP) event. While the QLDC COP preference is to convey primary flows via the pipe network, and secondary flows via the road carriageway, the proposed combination of both event flows into the pipe network is deemed appropriate given the added complexity of collecting bulk secondary surface flows and conveying these to soak pits, it is also noted that suitable emergency surface conveyance will be provided and therefore any associated risk is mitigated.

2.1 Design Parameters

The design parameters which guide the stormwater modelling are advised from Section 4 of the Queenstown Lakes District Council's Engineering Code of Practice (2025) (i.e. QLDC ECoP 2025), and associated Auckland City Technical Publication 108 - TP108(1999):

- The provision of soak pits/infiltration galleries adequate to dispose of the runoff from the development during the critical 1% AEP storm event.
- Percolation testing shall be undertaken by a suitably qualified professional at the individual soak pit/infiltration gallery locations to adequately demonstrate that soakage is available in all areas proposed for soakage.
- The soak pit/infiltration gallery design shall be in general accordance with the "Acceptable Solutions and Verification Methods for New Zealand Building Code Clause: E1NM1 Surface Water,"
- The critical storm duration and ensuring the soak pits/infiltration galleries will drain within 24 hours of the end of the critical event.
- A reticulated primary system to collect and dispose of stormwater from all impervious areas within the development to the soak pits/infiltration galleries
- This shall include details of treatment solutions to avoid adverse water quality effects on receiving waters, low impact design solutions are encouraged. As a minimum there shall be provision for the interception of settle-able solids, hydrocarbons and floatable debris prior to discharge from all proposed catchments.
- The individual lateral connections shall be designed to provide gravity drainage for the entire area within each activity area.
- Ensure that there is no inundation of any buildable areas of any buildings, and no increase in run-off rates onto land beyond the site from the pre-development situation
- All soakage devices must have a drain down period (to empty) within 24 hours from the end of the design storm event, to ensure capacity for back-to-back design storm events.

2.2 Design Rainfall Intensities

As per Queenstown Lakes District Council's Land Development and Subdivision Code of Practice 2025:

<u>Section 4.3.5.1.3</u>

- Primary stormwater network must use HIRDS V4 RCP 6.0 for 2081-2100 rainfall intensities and depths <u>at a minimum</u>.
- Secondary stormwater network must use HIRDS V4 RCP 8.5 for 2081-2100 rainfall intensities and depths.

Design rainfall depths exported from NIWA HIRDS V4.

As the intention is capture and combine both primary and secondary flows within the localised pipe network and to keep the stormwater modelling consistent, pipe sizing and modelling have used HIRDS V4 RCP 8.5 for 2081-2100 rainfall intensities and depths. This approach also adds some general conservatism to the calculations for the pipe network.

2.3 Stormwater Temporal Pattern

As per Auckland Regional Council's TP108 methodology, rainfall depth data from NIWA HIRDS V4 with RCP 8.5 climate change factor for the period 2081-2100 was converted into 24 hour nested storm profiles for 5% and 1% AEP events; these are plotted as *Figure 3* and *Figure 4* below, respectively.

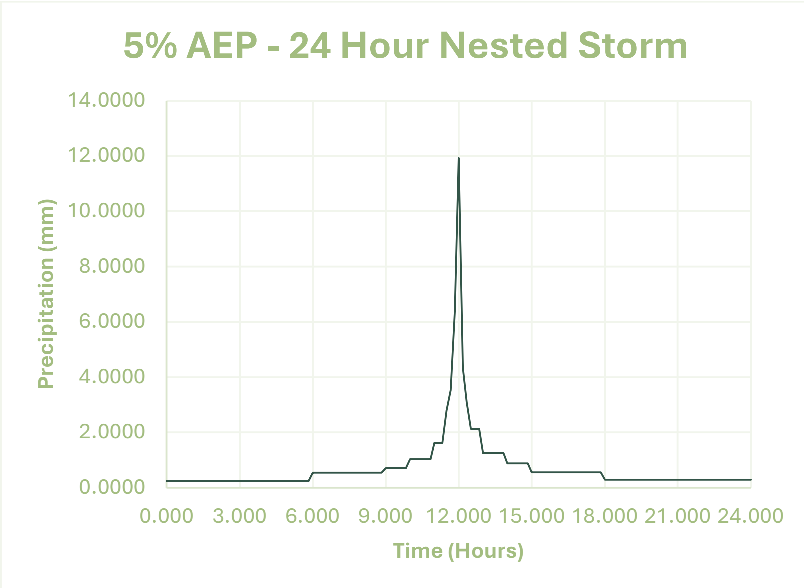


Figure 3: TP108 20% AEP, 24-hour nested storm profile.

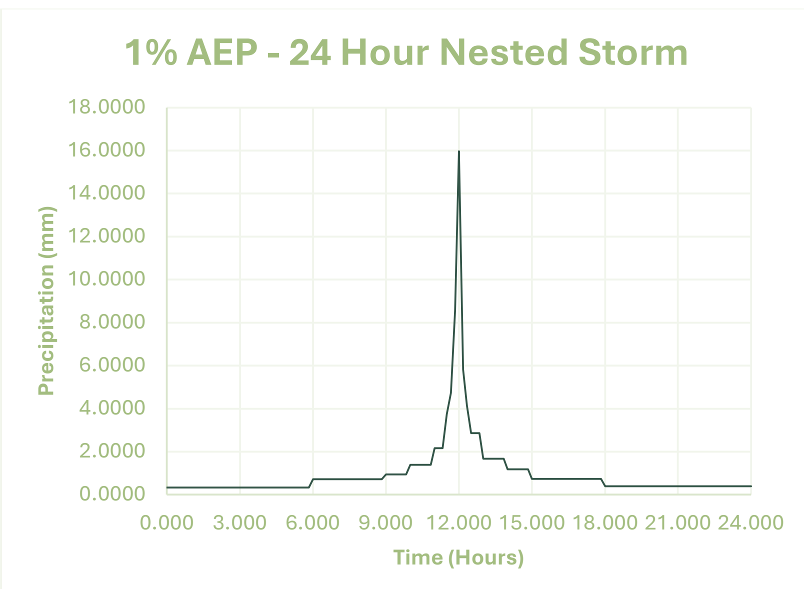


Figure 4: TP108 1% AEP, 24-hour nested storm profile.

2.4 Soil Profile, CN

Auckland Regional Council's *TP108 – Guidelines for Stormwater Runoff Modelling in the Auckland Region* have defined Hydrological Soil Groups into 4 categories ranging from A-D.

As per the Geosolve Ltd's Geotechnical Assessment (appended to Infrastructure Report), the geological stratigraphy on site consists of isolated uncontrolled fill, outwash gravel, outwash sand, and lake sediment.

Site observations include limited to no concentrated overland flow paths and the general indication is that the underlying ground is relatively free draining, while this could be categorised Group A, to provide a suitable level of conservatism Group B (alluvial sediments) was selected and the CN values (as shown in Table 2) were utilised in the HEC-HMS modelling and stormwater calculations.

2.5 Design Soakage Rate

As part of Geosolve Ltd's March 2018 geotechnical report, two soakage tests were conducted on site at locations shown in Figure 5 below.

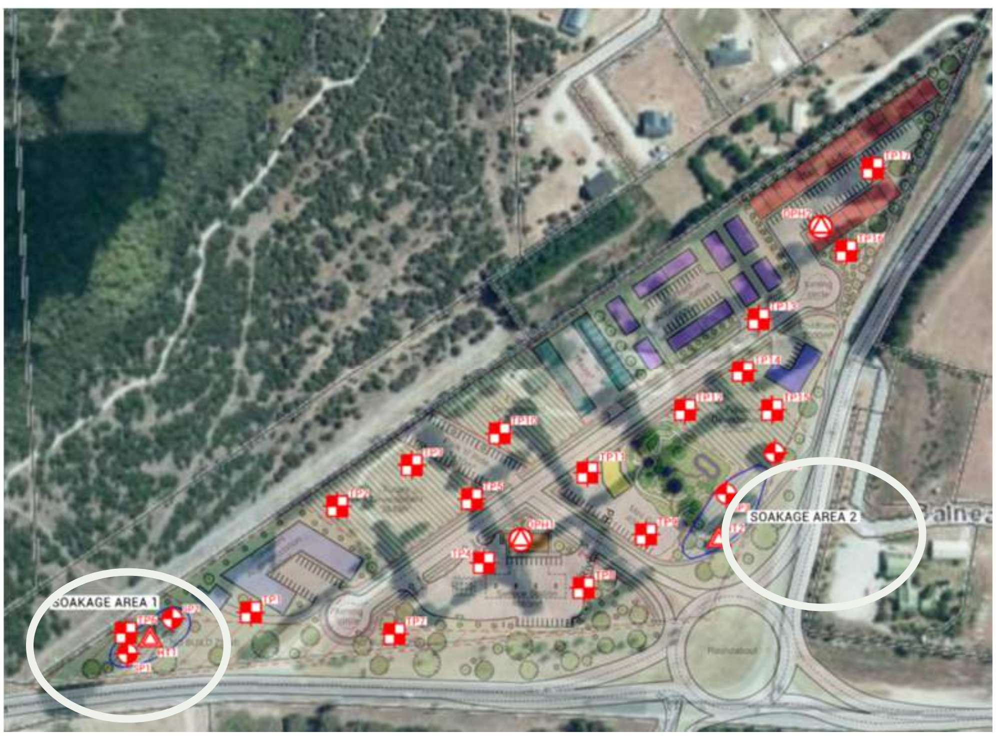


Figure 5. Geosolve Ltd's soakage testing location plan.

Unfactored soakage rates tested at Soakage Area 1 ranged between 0.07 – 0.18 L/s/m^2 (252 -648 mm/hr), while Soakage Area 2 was free draining.

Due to the inconsistent soakage rates obtained across the site, and neither of the existing test pit locations being specific to proposed soak pit locations, the stormwater model has adopted a maximum unfactored soakage rate of 2000 L/hr/m2, as allowable by QLDC's ECoP 2025 (section

4.3.8.11 soakage testing). This is consistent with a relatively free draining substrate and aligns with general site observations. However, to acknowledge the potential variability of the soakage over the site, a factor of safety of 3 has been conservatively applied to the unfactored soakage rate of 2000 L/hr/m2, for a design soakage rate of 666.67 mm/hr.

The soak pit design will be reviewed and refined following completion of site-specific soil infiltration testing for Mount Iron Junction in the exact location of the proposed soak pits. As per Geosolve's geotechnical report, the regional water table is expected to lie at depth below any future foundation levels and is not expected to be encountered during construction on site.

2.6 Proposed Scheme

Figure 6 below shows the current proposed scheme plan for the development of Mt Iron Junction. Refer also to **Appendix A** of this report for the subdivision scheme plans in full.

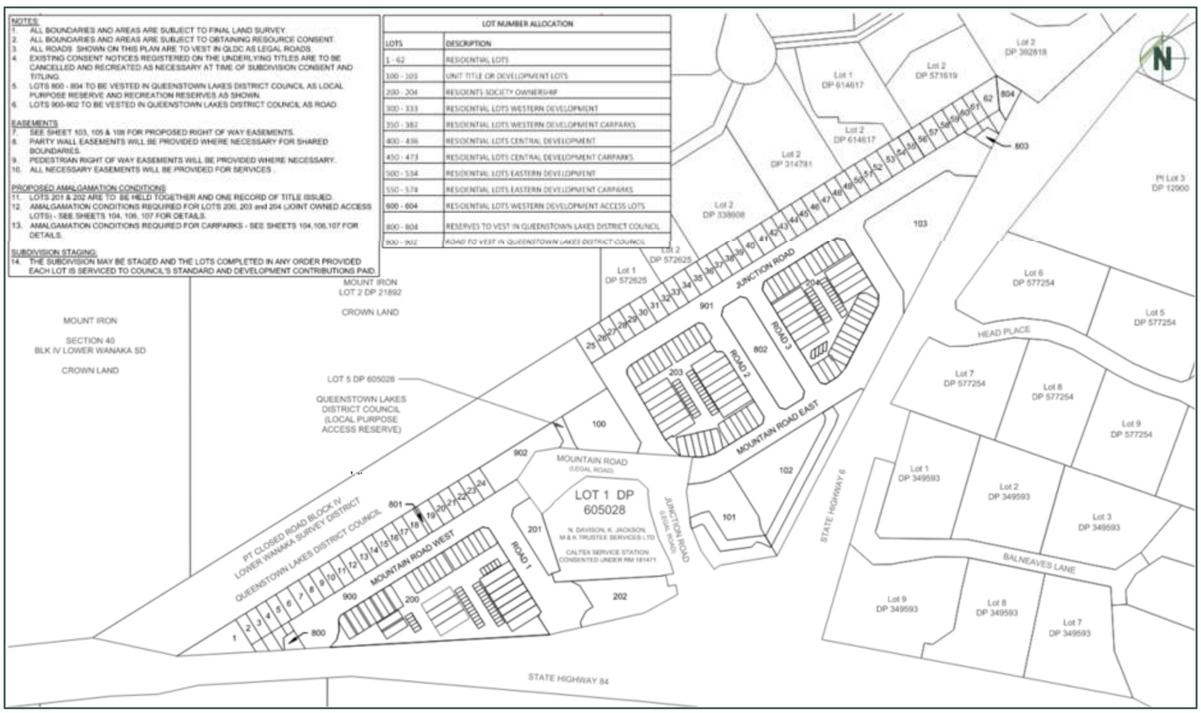


Figure 6: Proposed scheme plan for Mt Iron Junction development.

- Lots 1-62, 300-333, 400-436, and 500-534 are fee simple residential lots.
- Lots 350-384, 450-474, and 550-574 are fee simple carpark space lots that will be amalgamated with their relevant residential lot.
- Lots 200-203 are joint ownership access lots with fractional ownership by relevant residential lots.
- Lots 100-103 are bulk future development lots (potential future units/unit titled)
- Lots 800-807 are reserves to vest in Queenstown Lakes District Council (QLDC), including 3x local purpose drainage reserves containing s/w soakage pits (Soak Pits C, E, I).
- Lots 900-902 are roads to Vest in QLDC, these lots will include vested soak pits (Soak Pits J, H)
- Joint ownership access lots and private stormwater infrastructure (Soak Pits A, B, F, G) will be owned and maintained by a management company for the relevant lots serviced.

2.7 Stormwater Catchments (Post Development -100 year)

The proposed development described in section 2.6 above is split into multiple discrete stormwater catchments, as shown in *Figure 7* below.

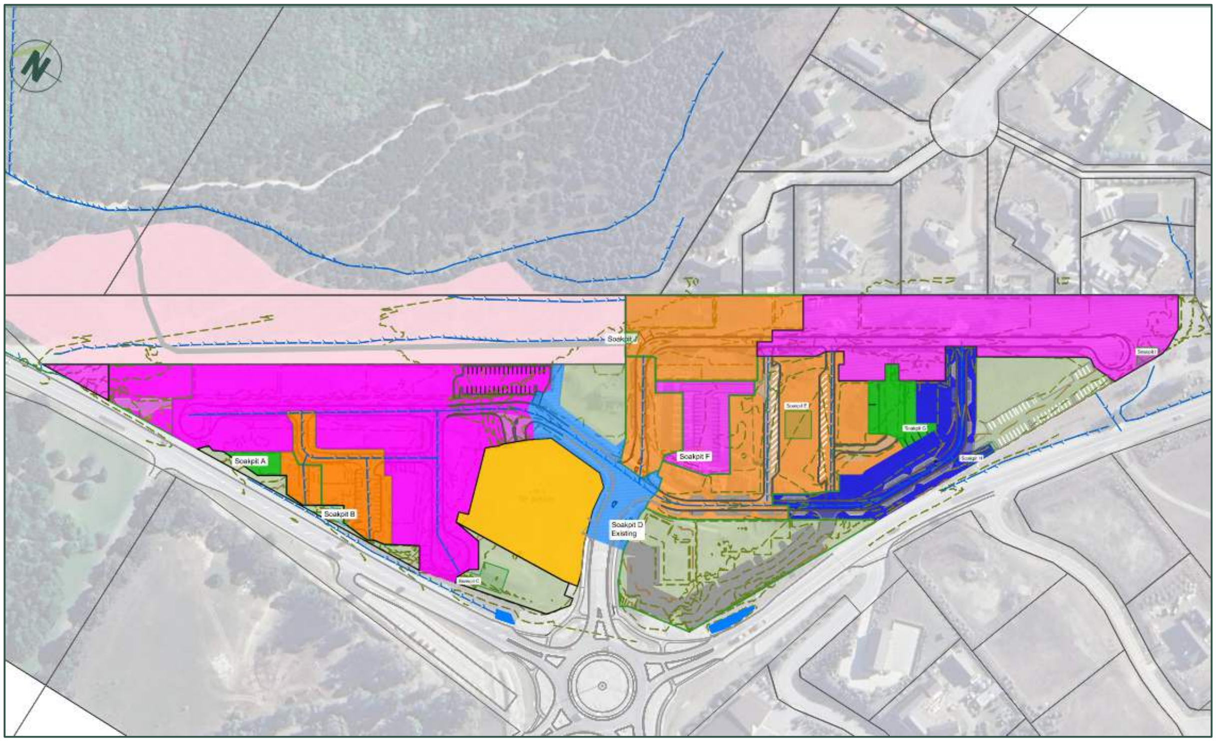


Figure 7: Post Development Catchments for HEC-HMS modelling. (see Appendix A for larger image).

Table 2: Summary Mt Iron Junction Catchment Area and Parameters.

Catchment	"Type of Surface or Land Use"	Post-Dev, C	Area, m^2	Area, Ha
Catchment J	Pervious	61	20844	2.0844
PD-A (Private)	Pervious	61	154	0.0154
PD-A (Private)	Impervious	98	174	0.0174
PD-B (Private)	Pervious	61	577	0.0577
PD-B (Private)	Impervious	98	2227	0.2227
PD-C	Pervious	61	3760	0.3760
PD-C	Impervious	98	11218	1.1218
PD-E	Pervious	61	7670	0.7670
PD-E	Impervious	98	9021	0.9021
PD-F (Private)	Pervious	61	819	0.0819
PD-F (Private)	Impervious	98	945	0.0945
PD-G (Private)	Pervious	61	408	0.0408
PD-G (Private)	Impervious	98	668	0.0668
PD-H	Pervious	61	1072	0.1072
PD-H	Impervious	98	3220	0.3220
PD-I	Pervious	61	1072	0.1072
PD-I	Impervious	98	7944	0.7944

The proposed post development stormwater catchments for Mt Iron Junction have been split into the above 9 catchment areas draining towards discrete soakage pits, where pervious and impervious areas have been tabulated as shown in

Table 2. These discrete catchments will drain stormwater towards the proposed soak pit and fully dispose of stormwater up to the 1% AEP event, the only exception being Catchment J where only ~40m^2 of land is available for soakage purposes. Excess stormwater above the capacity of the 40m^2 Catchment J soak pit will be reticulated downstream towards soak pit E, where more space and soakage capacity is available to fully dispose of the full 1% AEP stormwater flow.

All pits are also designed to have a full drain down time of less than 24 hours, after the end of the initial storm event.

2.8 Model Setup

2.8.1 Pre-Development Flow Model

Figure 8 below shows the pre-development model set up in HEC-HMS for modelling of the 5% and 1% AEP 24 hour storm event, with a climate change factor of RCP 8.5 for 2081-2100 period applied.

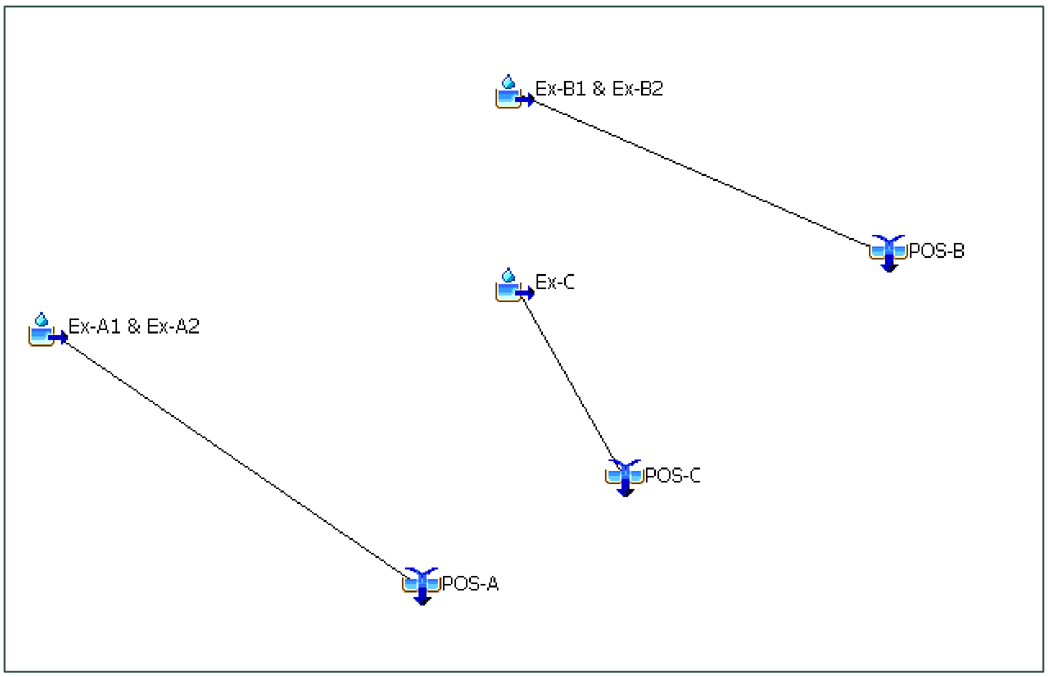


Figure 8: HEC-HMS stormwater flows pre-development model

The above modelled catchment areas are based on the predevelopment catchments and flow paths discussed under Section 1.3 above.

Point of study A (POS A) is the existing NZTA soak pit located to the north-west of the Mount Iron junction roundabout. POS A currently accepts flows from catchment Ex-A2 via sheet flow to the SH84 water table/swale. The EX-A2 flows originate from within the western area of the subject site, with flows sheet flowing across slightly graded and roughly grassed land before arriving at the SH84 northern water table and tracking in a concentrated flow to the Mt Iron roundabout soak pit. It is note that POS A and the associated State Highway soak pit also accept flows from an area referred to as EA-A1. Ex A1 drains the steep bedrock and scree slopes of Mt Iron to the north of the subject site. The flows from catchment Ex-A1 do not pass through the development land.

Point of study B (POS B) is the existing SH6 water table/swale in the vicinity of the QLDC wastewater pumpstation. POS C currently accepts flows from catchment Ex-B1 & B2 via low velocity surface flows that cross undulating ground that falls very gradually to the north-east.

Point of study C (POS C) is the existing NZTA soak pit located to the north of the Mount Iron junction roundabout. POS C currently accepts flows from catchment Ex-C via sheet flow to the SH6 water table/swale. The EX-A2 flows originate from within the western area of the subject site, with flows sheet flowing across slightly graded and roughly grassed land before arriving at the SH84 northern water table and tracking in a concentrated flow to the Mt Iron roundabout soak pit.

2.8.2 Post-Development Flow Model

Figure 9 below shows the post-development model set up in HEC-HMS for modelling the 1% AEP 24-hour storm event, with a climate change factor of RCP 8.5 for 2081-2100 period applied.

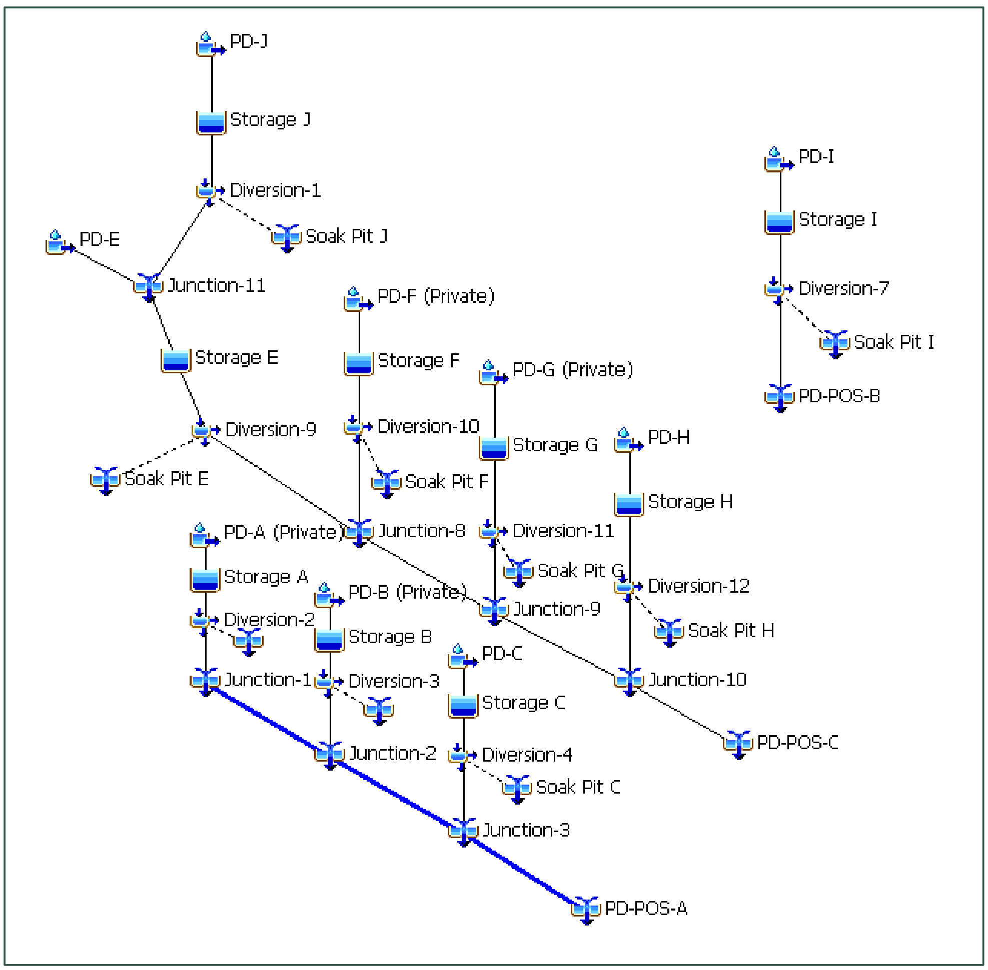


Figure 9. HEC-HMS stormwater flow post-development model.

Each of the proposed catchments (as shown in *Figure 7*) are arranged with a storage node downstream of the catchment node. The storage node discharges flow through a HEC-HMS diversion node, which simulates infiltration via the soak pit node. Any flow above the infiltration rate will discharge down to the Junction node and flow further downstream.

The proposed soak pit depth, extents and soakage rates to accommodate the 1% AEP storm are as shown below in Table 3 and 5 below. The soak pits have been modelled as underground soakage and attenuation galleries only, with no above ground attenuation infrastructure (e.g. basins). A nominal 0.5m depth has been allowed for to transition between the top of storage media to finished ground. 0.95 void ratio of soakage media has been factored into the storage calculations; product specification will be finalised during detailed design.

Table 3. Post-development soak pit depths.

Storage	Soak Pit Depth, m
Storage A	1.12
Storage B	1.56
Storage C	1.99
Storage E	2.34
Storage F	1.12
Storage G	1.12
Storage H	1.12
Storage I	1.56
Storage J	1.99

Table 4. Post-development soak pit dimensions and areas.

Soak Pit	Design Sr, mm/hr	Total Contributing Catchment Area, m^2	Soak Pit Base Length, m	Soak Pit Base Width, m	Proposed Area (0.95 Ratio), m^2
Soak Pit A	666.67	328	3.00	3.00	9.47
Soak Pit B	666.67	2804	10.00	7.00	73.68
Soak Pit C	666.67	14978	20.00	14.00	294.74
Soak Pit E	666.67	16691	21.00	20.00	442.11
Soak Pit F	666.67	1764	8.00	5.00	42.11
Soak Pit G	666.67	1076	6.00	5.00	31.58
Soak Pit H	666.67	4292	12.00	10.00	126.32
Soak Pit I	666.67	9016	15.00	15.00	236.84
Soak Pit J	666.67	20844	10.00	4.00	42.11

Full modelling inputs and details are detailed in **Appendix B**

3. Results

3.1 Post-Development Stormwater Flow

For the purposes of the Mt Iron development stormwater assessment, the design of the soak pit and attenuation volumes were determined from the secondary 1% AEP, 24-hour event (see Section 3.1.2.).

Primary event modelling used NIWA HIRDS v4 rainfall depths with the TP108 nested temporal pattern. The design storm was the 5% AEP, 24-hour event under RCP 8.5 (2081–2100).

Table 5 below shows the post-development primary and secondary flows:

Table 5. Post-development catchment peak discharge rates.

Post-Development Catchment	5% AEP, Peak Discharge, m^3/s	1% AEP, Peak Discharge, m^3/s
PD-A (Private)	0.00356	0.00533
PD-B (Private)	0.0389	0.05461
PD-C	0.19985	0.28382
PD-E	0.18326	0.27332
PD-F (Private)	0.01927	0.02877
PD-G (Private)	0.01279	0.01872
PD-H	0.05743	0.08148
PD-I	0.13347	0.18356
PD-J	0.12833	0.21141

As post-development 5% and 1% flows are to be fully captured and discharged in full to each catchment's proposed soak pit (with the exception of PD-J), there should be no stormwater flow discharging off-site, i.e. post-development flows will be significantly less than the pre-development scenario. This will be a net improvement to the existing downstream situation at POS A-C and the performance of the two NZTA State Highway soak pits.

3.2 Post-Development Stormwater Mitigation

To ensure post-development stormwater flows are fully disposed of via infiltration to ground and do not result in flows which leave their respective catchments, volume attenuation is proposed via underground storage chambers.

In terms of the HEC-HMS model, post-development catchments feed into a storage-discharge node with a single orifice outlet, the discharge orifice shall limit the discharge rates to below infiltration rate.

Based on the 666.67 mm/hr design soakage rate detailed in section 2.5, *Table 6* below details the attenuated flow rate and the infiltration rate for each soak pit.

Table 6. Soak pit infiltration rates vs 1% AEP Peak flow rates.

Post-Development Catchment	Infiltration Rate (m3/s)	1% AEP, Peak Discharge (m3/s)
Storage A	0.00175	0.00129
Storage B	0.01365	0.01334
Storage C	0.05458	0.05392
Storage E	0.08187	0.08023
Storage F	0.00780	0.00779
Storage G	0.00585	0.00579
Storage H	0.02339	0.02144
Storage I	0.04386	0.04221
Storage J	0.00780	0.18375

The system is adequately sized to contain and dispose of the 1% AEP storm if the 1% AEP Peak discharge rate is less than the infiltration rate for each respective soak pit.

The storage curves for each soak pit are included in **Appendix C**, *Figure 10* below details the Storage A curves of volume and flow over time. As peak storage does not exceed total available volume (storage A proposed volume of 106 m^3), the system does not overflow in the 1% AEP 24 hour storm event.

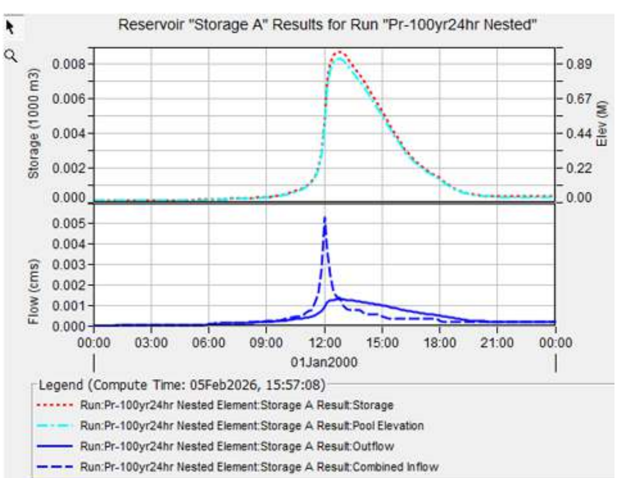


Figure 10. Storage A volume and flow over time.

4. Emergency/Tertiary Events

If the pipe network and associated soak pits are exceeded in the super design event (tertiary event) or network blockage failure occurs, the flows from the individual discrete catchments will be combined into larger catchments defined by the road network and will be conveyed via the road carriageways to existing discharge points from the site.

4.1 Stormwater Catchments (Emergency/Tertiary Events)

The post development emergency/tertiary event overland flow path catchments are illustrated in Figure 11.

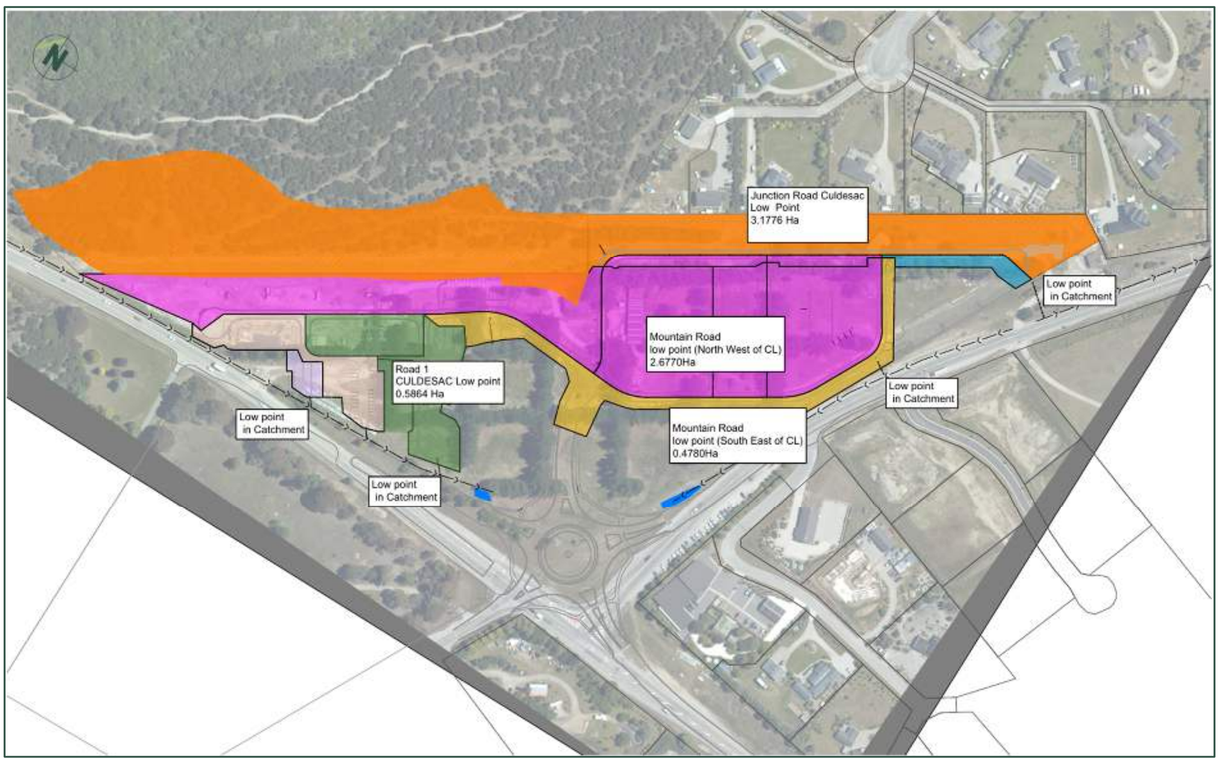


Figure 11: Emergency/Tertiary catchment and low points. (see Appendix A for larger image).

Junction Road Catchment (*Orange*)

Existing external catchment Ex-B1 enters the Mt Iron Junction development along the existing pervious flow path, which then enters the Junction Road carriageway along the indicative dashed green line shown below. These flows combine with flows from the northern side of Junction Road and flow east, exiting the development at the cul-de-sac head and discharging east off site via the existing state highway swale and predevelopment flow path. The total emergency/tertiary ‘Junction Road Cul-de-

Sac' catchment has an area of 3.1776 ha and includes both off-site Ex-B1 undeveloped areas and development runoff.

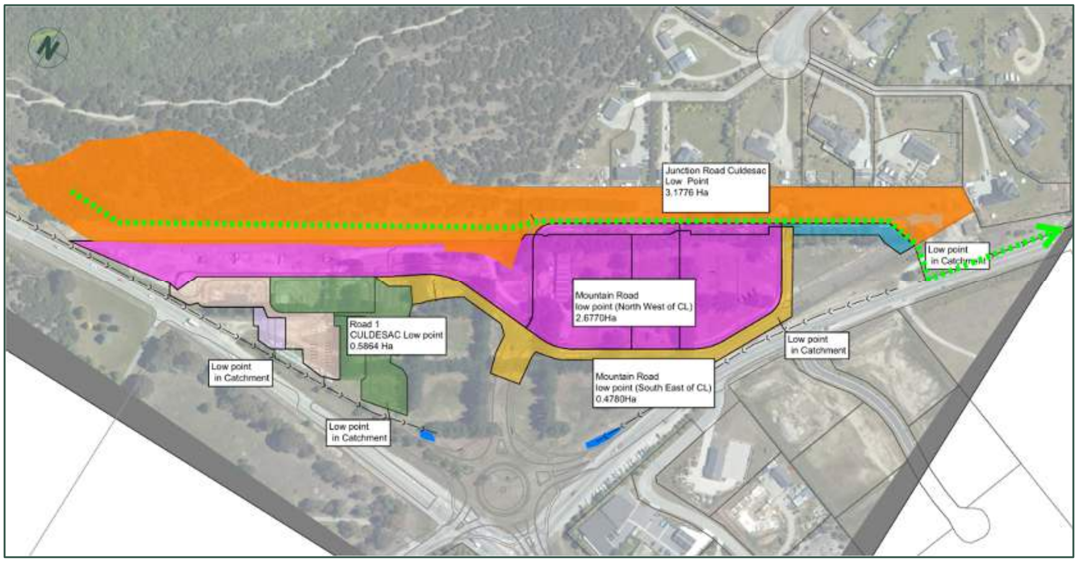


Figure 12: Emergency/Tertiary flow path from Junction Road Catchment.

Mountain Road Catchments

The emergency/tertiary flow from both the Mountain Road north and Mountain Road south catchments are conveyed south-east via the road carriageway towards the existing discharge point along the eastern boundary of the Mount Iron Junction development. Once it exits the site, this discharge flows within the state highway water table south to the existing rock soak pit in the vicinity of the Mount Iron roundabout. Once overwhelmed the existing state highway rock soak pit will head up and spill over the SH6 carriageway and flow east to the Cardrona River. The total emergency/tertiary 'Mountain Road' catchments have a combined area of 3.155 ha.

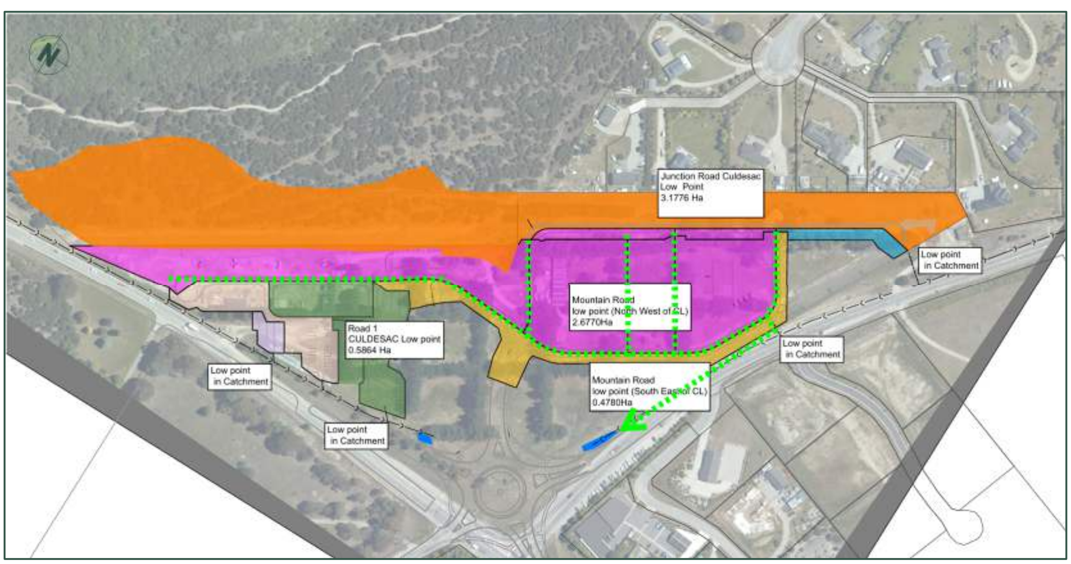


Figure 13: Emergency/Tertiary flow path from Mountain Road Catchment.

Road 1

The emergency/tertiary flow from Road 1 and associated minor catchments on the western portion of the development are conveyed south via the road carriageway and ROWs towards the State Highway

84 water table. These flows will combine with the off-site flows from catchment Ex A1 to the north-west and flow south-east to the existing rock soak pit in the vicinity of the Mount Iron roundabout. Once overwhelmed the existing state highway rock soak pit will head up and spill over the SH6 carriageway and flow east to the Cardrona River. The emergency/tertiary ‘Road 1’ catchment has an area of 0.5864 ha.

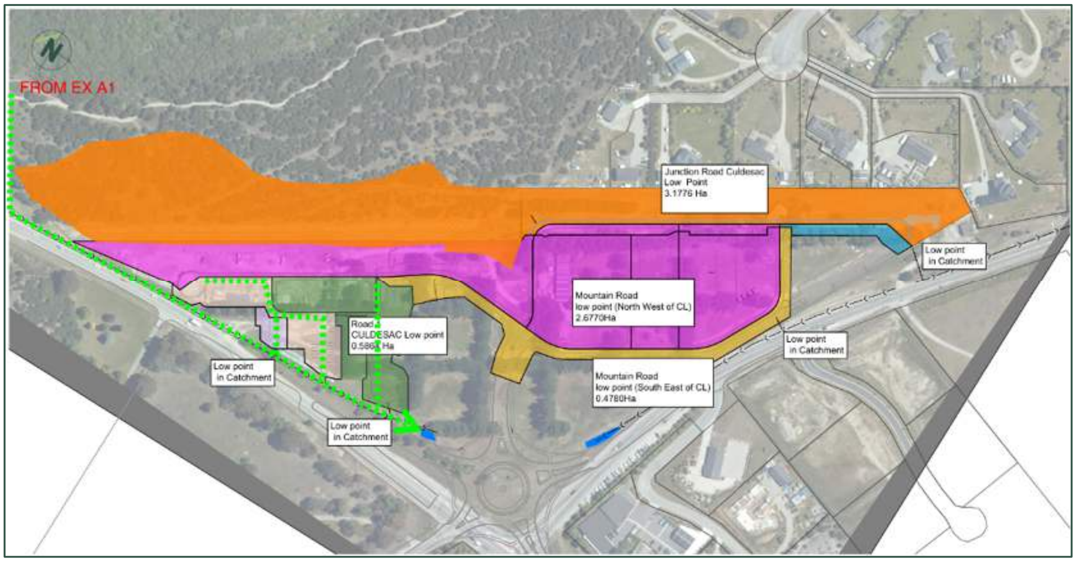


Figure 14: Emergency/Tertiary flow path from Road 1 & associated western catchments.

4.2 Emergency/Tertiary Conveyance

Based on the requirements of Section 4.3.4 of the QLDC COP, emergency flow paths need to be provided to cater for 100% blockage and failure of the pipe network and associated soak pits. The ability of the internal road carriageways to convey the 1% AEP RCP 8.5 peak flows to the existing downstream off-site points of discharge have therefore been assessed. It is noted however that due to the flat grades, limited external catchments, multiple discrete soak pits and localised pipe networks, and significant network inlet sumps, the chance of a full failure of the development primary and secondary piped network as a whole is considered extremely unlikely.

As this assessment includes no catchments greater than 10ha and no soakage or storage components, Section 4.3.5 of the QLDC COP accepts that this catchment flows can be assessed using the Rational Method.

The capacity of the road carriageway itself has been undertaken using Mannings trapezoidal channel calculations via FHWA Hydraulic Tool Box version 5.4, with the minimum carriageway grade set at 0.5%.

Mountain Road (North West of CL) 2.677ha

Peak rational 100 year flows from this catchment have been estimated at 308 l/s with a TOC of 10 minutes. As below this exceeds the capacity of the northern half of the Mountain Road carriageway and will therefore spill to southern half of the Mountain Road carriageway.

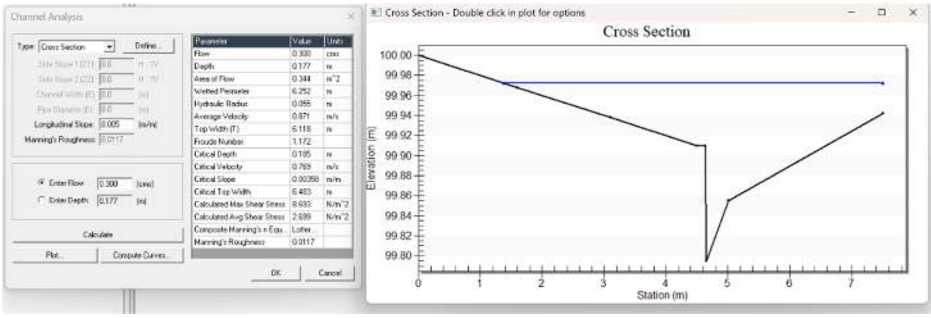


Figure 15: Mountain Road carriageway north-west of centreline emergency flow carrying capacity.

To simulate the above combining with the Mountain Road (South-East of CL) flows to the south, the peak rational 100 year flows of 55 l/s from the 0.4780ha Mountain Road (South-East of CL) have been combined with the Mountain Road (North West of CL) 308 l/s. This gives a peak of 363 l/s. As below this can be contained within the total carriageway.

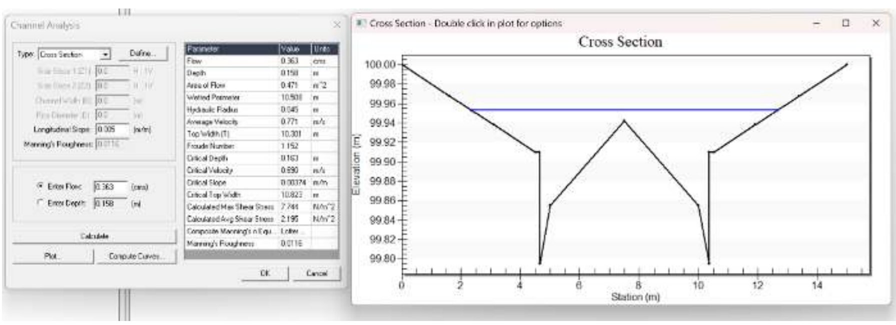


Figure 16: Mountain Road full carriageway emergency flow carrying capacity (at 363 l/s).

Junction Road (TOC 50min) 2.4251ha

The TOC for this catchment is considerably longer at around 50 minutes due to the inclusion of the relatively flat external catchment Ex-B1. Peak rational 100 year flows have been estimated at 151 l/s. As below this exceeds the capacity of the northern half of the Junction Road carriageway and will therefore spill into the Mountain Road catchment.

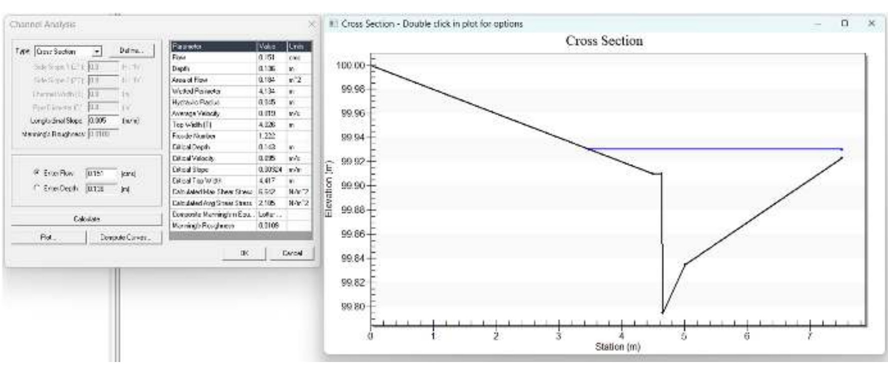


Figure 17: Junction Road carriageway north of centreline emergency flow carrying capacity.

To simulate the above Junction Road flows combining with the full Mountain Road flows to the south, and as an additional worst case the external Junction Road catchment short tracking into the development and reducing the TOC to 10minutes, the worst case of all of Junction and Mountain Road catchment flows combined with a 10 minute TOC has been considered. While deemed highly unlikely, this equates to a peak 628 l/s. As below this can be contained within the total carriageway at any point within the Mountain and Junction Road network.

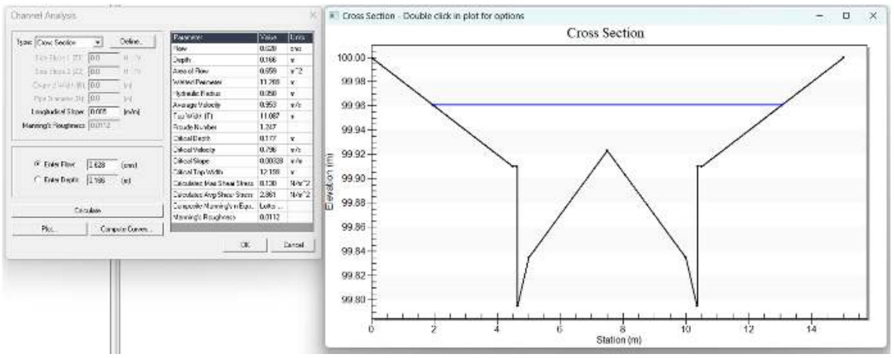


Figure 18: Junction & Mountain Road full carriageway emergency flow carrying capacity (at 628 l/s).

Road 1 0.5864ha

Peak rational 100 year flows from this relatively small catchment have been estimated at 68 l/s with a TOC of 10 minutes. Given the relatively small peak flow volume this will easily be contained within the Road 1 carriageway and therefore no specific capacity assessment is deemed required.

Overall, the above assessment confirms suitable capacity within the road carriageway to convey super design events and full failure of the pipe and soak pit networks to the existing discharge points to the surrounding state highway table drains. The maximum potential flow depths and velocities comply with Section 4.3.4.1b of the QLDC COP as they are less than 0.4 m^2/s and are therefore acceptable.

5. Conclusion

Stormwater modelling (HEC-HMS) undertaken for the Mt Iron Junction development demonstrates that the proposed soak pit systems provide adequate capacity to manage and soak to ground both primary (5 % AEP) and secondary (1 % AEP) design events under future rainfall conditions (RCP 8.5, 2081–2100). All three external points of study (POS A-C) have been confirmed as having less flows originating from the development in the post development 20 year and 100-year scenarios and therefore off-site stormwater runoff impacts are mitigated.

Under emergency or super design tertiary event any flows will spill to the local road carriageway and will track to the existing low points and off-site existing state highway flow paths. The capacity of the internals road carriageways to convey these flows up to the 1% ARI RCP 8.5 event volumes from the relevant contributing catchments have been checked and confirmed sufficient.

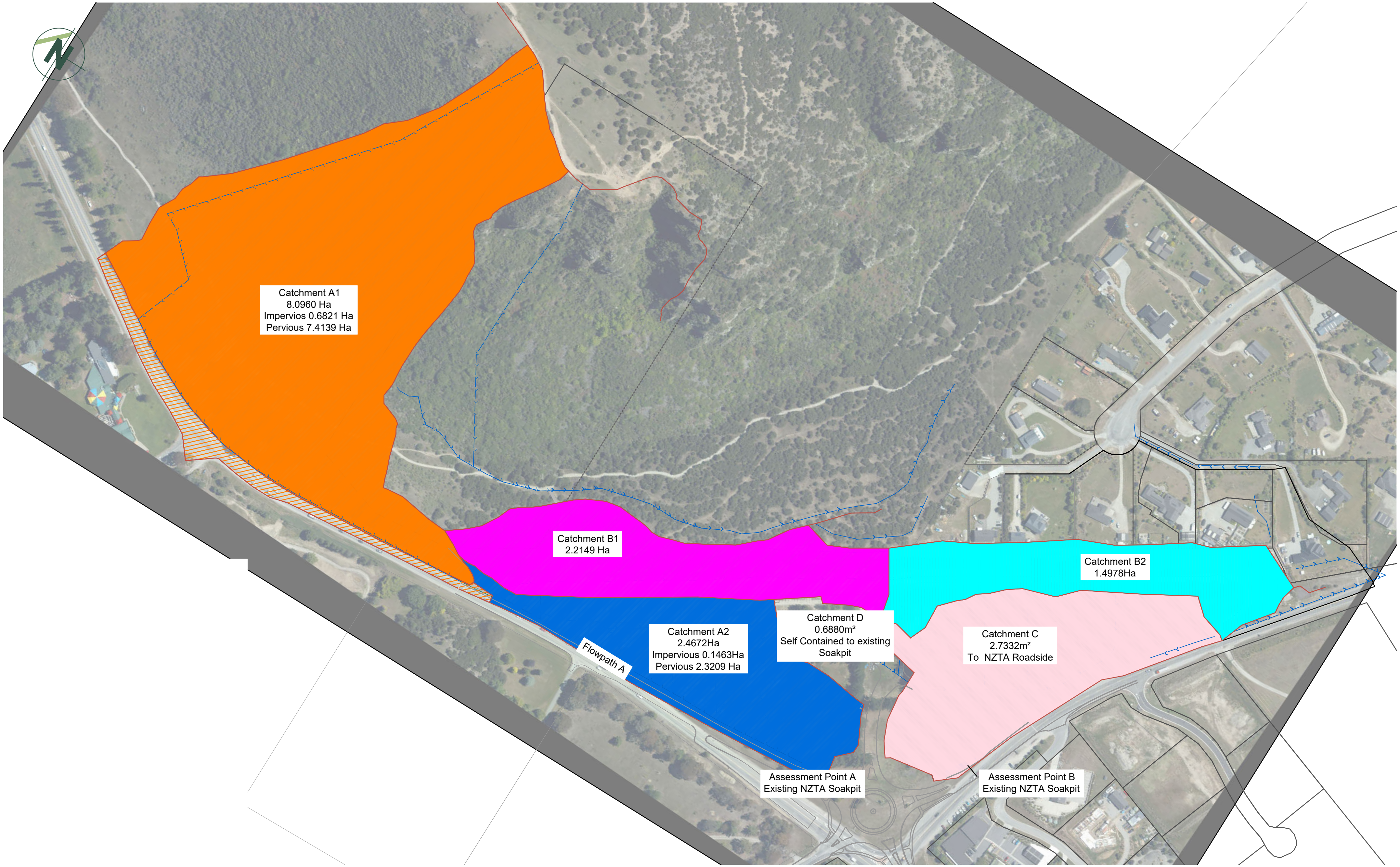
The proposed soak pits consist of underground soakage galleries of varying depths (1.12m – 2.34m). Void ratio of storage volumes are tentatively sized on a proprietary device of void ratio 0.95. This will be confirmed in detailed design. The design soakage rate used for all catchment soak pits is 666.67 mm/hr, noting that catchment specific infiltration rates will need to be conducted across the development to confirm this rate for detailed design and also potentially improve the stormwater modelling.

Further recommended work under detailed design should include:

- Site-specific infiltration testing to confirm local soil permeability
- Confirmation of soakage device type (proprietary or otherwise) and associated refinement of soak pit geometry (or possibly basin), side slopes, and void ratios
- Confirmation of suitable pretreatment devices to ensure soak pits maintain performance
- Verification of infiltration and storage performance under final site conditions

Subject to the above refinements, the proposed stormwater management approach is expected to comply with QLDC's Land Development and Subdivision Code of Practice (2025) and provide a robust, future-resilient solution for Mount Iron Junction.

APPENDIX A – CATCHMENT PLANS



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Land Professionals

CLIENT	DRAWING TITLE
MT IRON JUNCTION LTD	ENGINEERING DESIGN
JUNCTION ROAD	PRE DEVELOPMENT CATCHMENTS
WANAKA	

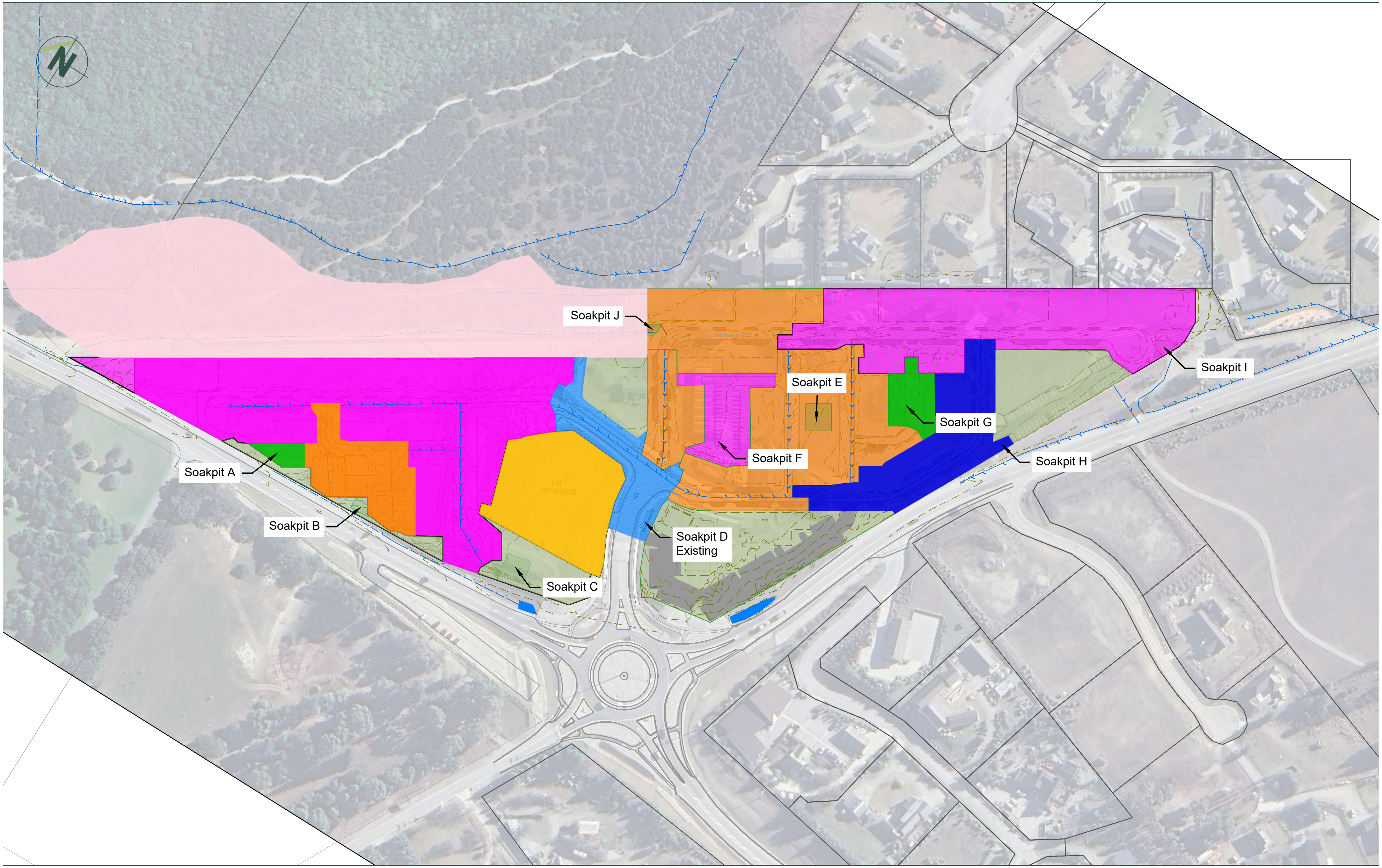
DATUM INFORMATION	
COORDINATE SYSTEM	NZGD 2000
	LINDIS PEAK CIRCUIT
DATUM	NZVD2016
ORIGIN OF COORDINATES	
ORIGIN OF LEVELS	

REV	DRAWN	DATE	NOTE

STATUS **FOR INFORMATION**

SURVEYED	
DESIGNED	PHJ
DRAWN	PHJ
REVIEWED	TO BE REVIEWED
APPROVED	TO BE APPROVED

PROJECT	P240103
DRAWING NO	006
SHEET	430
REVISION	1
SCALE (A3)	1:3000



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Land Professionals

CLIENT
MT IRON JUNCTION LTD
JUNCTION ROAD
WANAKA

DRAWING TITLE
ENGINEERING DESIGN
POST DEVELOPMENT
STORMWATER CATCHMENTS

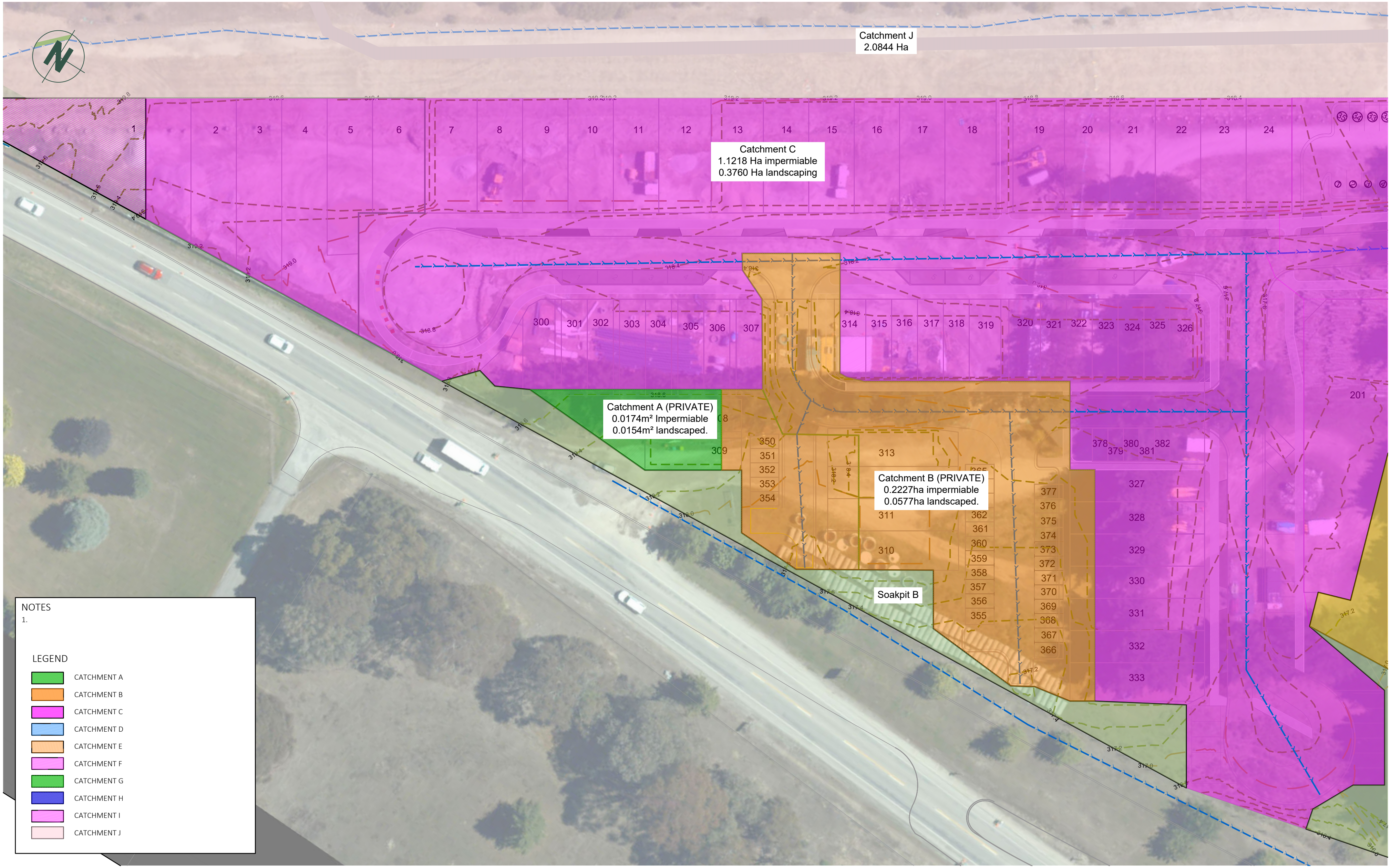
DATUM INFORMATION	
COORDINATE SYSTEM	NZGD 2000 LINDIS PEAK CIRCUIT
DATUM	NZVD2016
ORIGIN OF COORDINATES	
ORIGIN OF LEVELS	

REV	DRAWN	DATE	NOTE

STATUS FOR INFORMATION

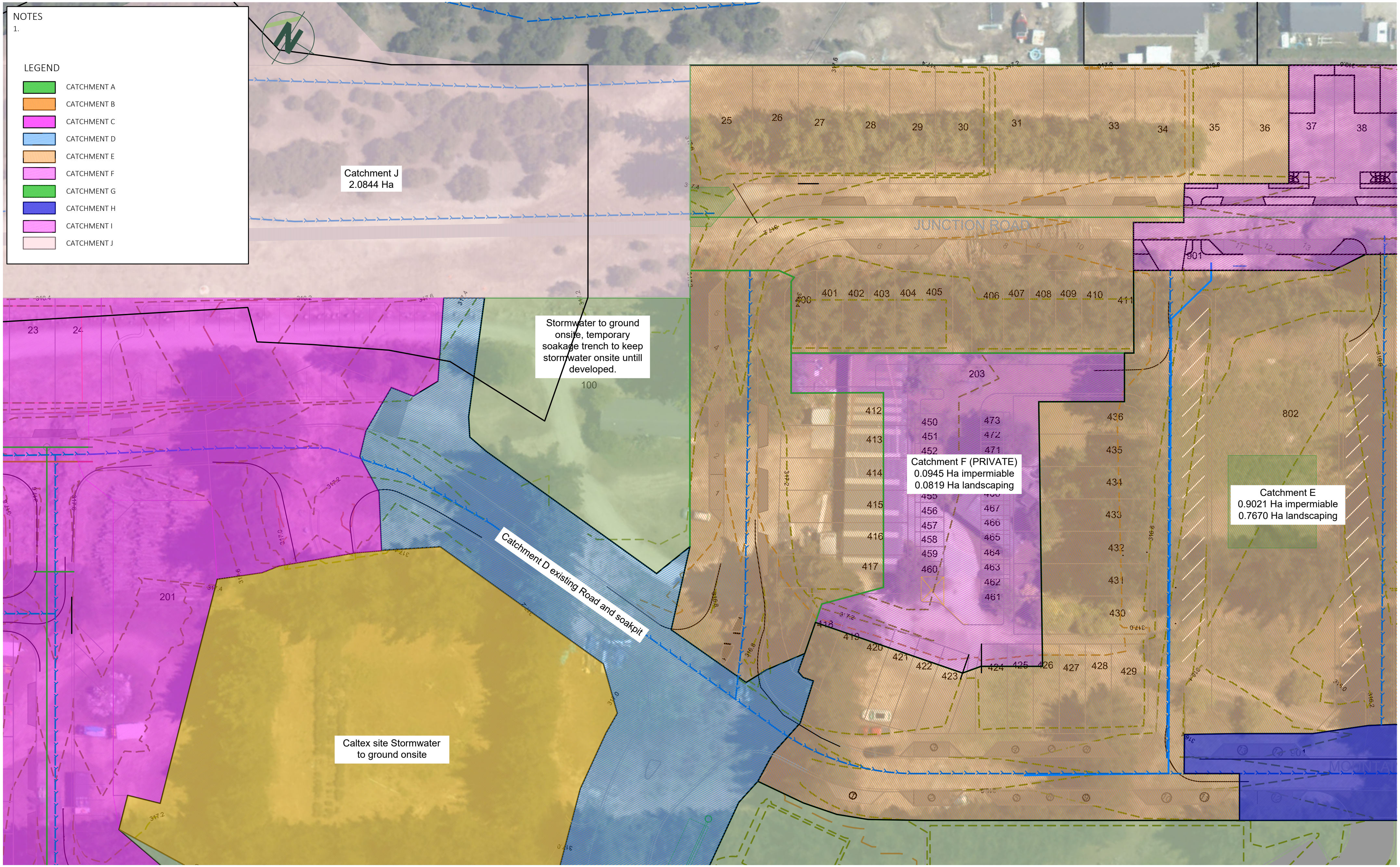
SURVEYED	-
DESIGNED	PHJ
DRAWN	PHJ
REVIEWED	TO BE REVIEWED
APPROVED	TO BE APPROVED

PROJECT	P240103
DRAWING NO	006
SHEET	435
REVISION	1
SCALE (A3)	1:2000



patersons. Land Professionals

CLIENT	DRAWING TITLE	DATUM INFORMATION		REV	DRAWN	DATE	NOTE				
MT IRON JUNCTION LTD	ENGINEERING DESIGN	COORDINATE SYSTEM	NZGD 2000					SURVEYED	LIDAR	PROJECT	P240103
JUNCTION ROAD	POST DEVELOPMENT CATCHMENTS		LINDIS PEAK CICUIT					DESIGNED	PHJ	DRAWING NO	006
WANAKA	SHEET A	DATUM	NZVD2016					DRAWN	PHJ	SHEET	435A
		ORIGIN OF COORDINATES						REVIEWED	TO BE REVIEWED	REVISION	1
		ORIGIN OF LEVELS		STATUS		FOR INFORMATION		APPROVED	TO BE APPROVED	SCALE (A3)	1:600



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Land Professionals

CLIENT	DRAWING TITLE
MT IRON JUNCTION LTD	ENGINEERING DESIGN
JUNCTION ROAD	POST DEVELOPMENT CATCHMENT
WANAKA	SHEET B

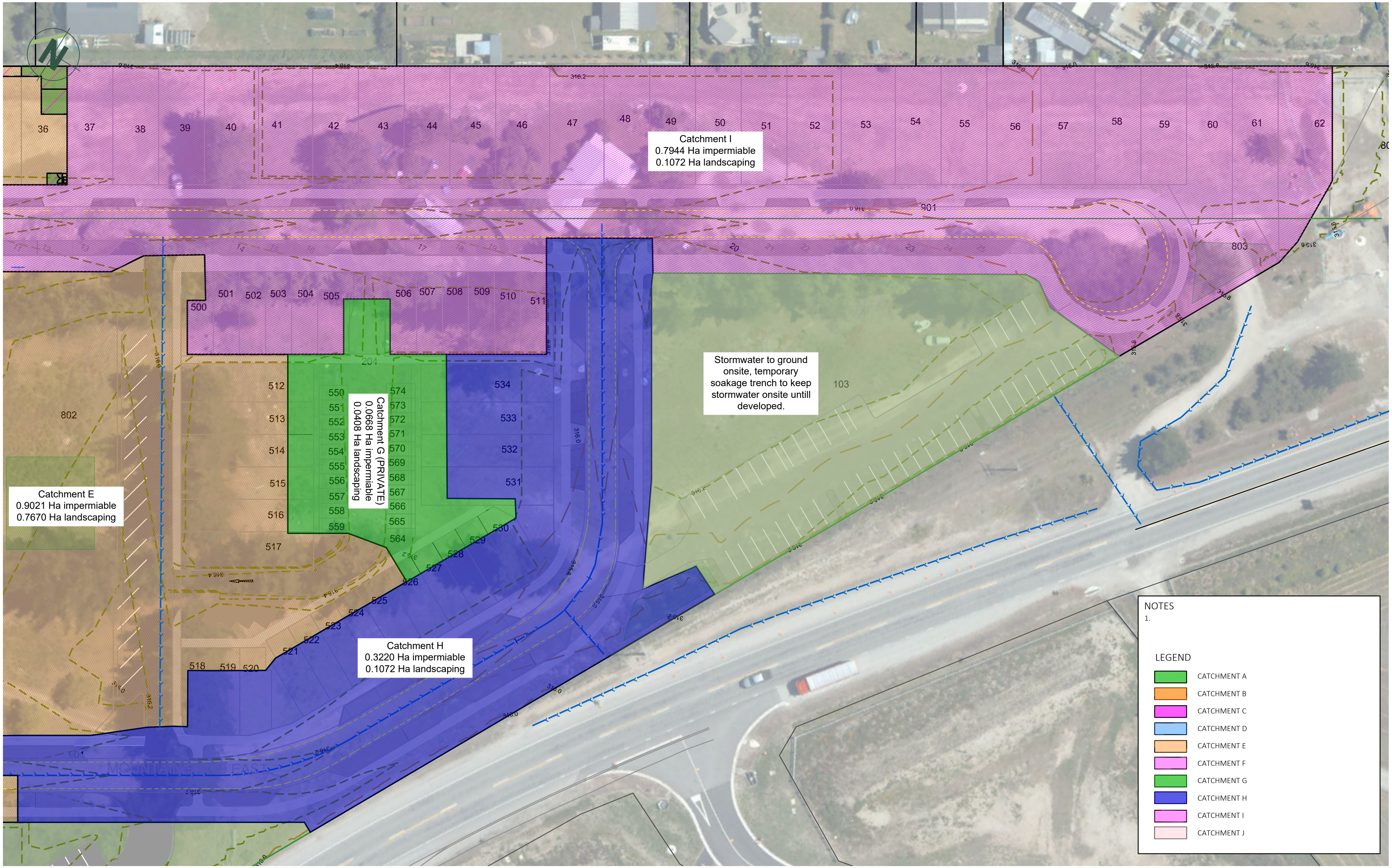
DATUM INFORMATION
COORDINATE SYSTEM NZGD 2000 LINDIS PEAK CICUIT
DATUM NZVD2016
ORIGIN OF COORDINATES
ORIGIN OF LEVELS

REV DRAWN DATE NOTE

STATUS FOR INFORMATION

SURVEYED	LIDAR
DESIGNED	PHJ
DRAWN	PHJ
REVIEWED	TO BE REVIEWED
APPROVED	TO BE APPROVED

PROJECT	P240103
DRAWING NO	006
SHEET	435B
REVISION	1
SCALE (A3)	1:600



patersons.
Land Professionals

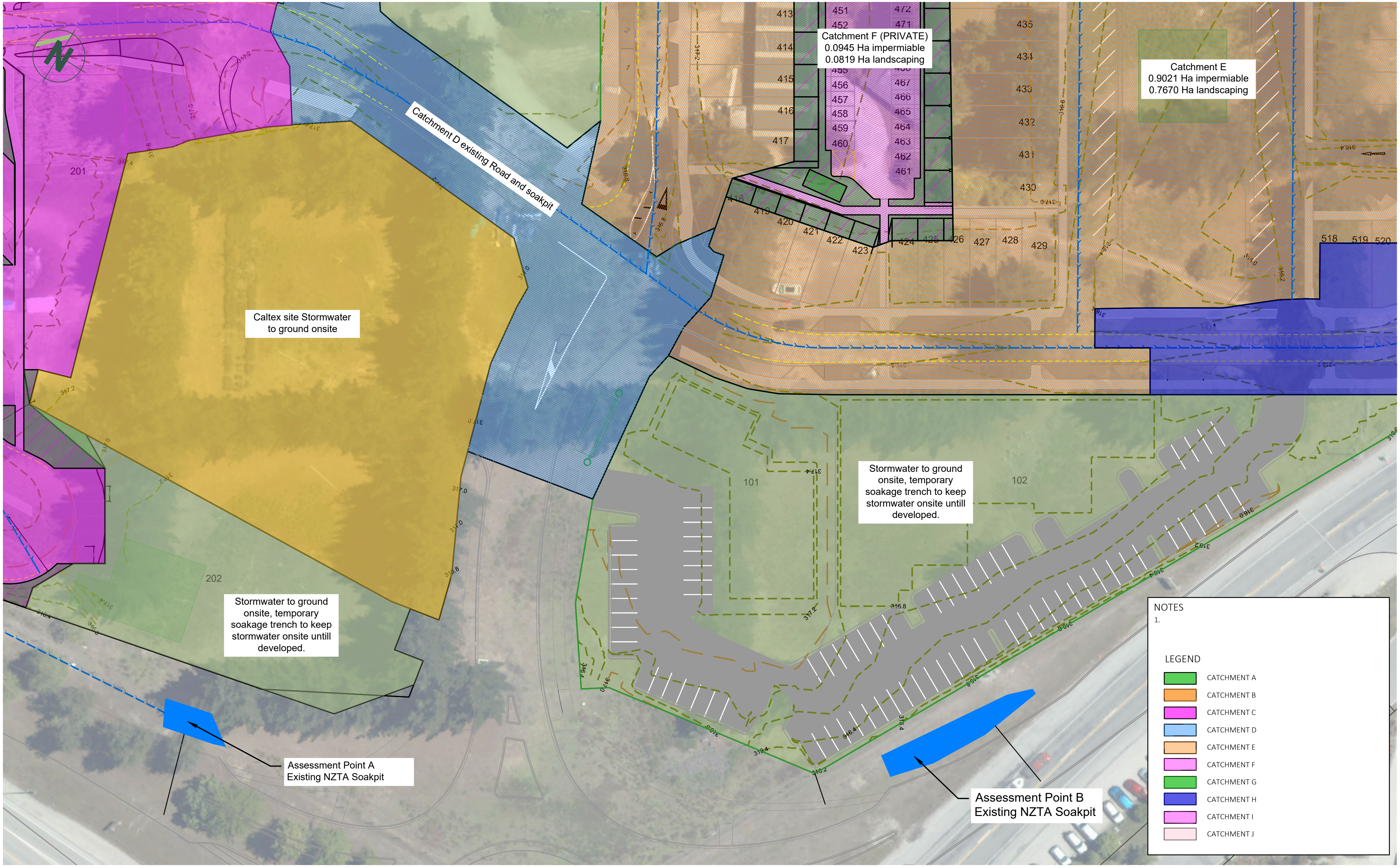
CLIENT	DRAWING TITLE	DATUM INFORMATION
MT IRON JUNCTION LTD	ENGINEERING DESIGN	COORDINATE SYSTEM NZGD 2000 LINDIS PEAK CICUIT
JUNCTION ROAD	POST DEVELOPMENT CATCHMENT	DATUM NZVD2016
WANAKA	SHEET C	ORIGIN OF COORDINATES
		ORIGIN OF LEVELS

REV DRAWN DATE NOTE

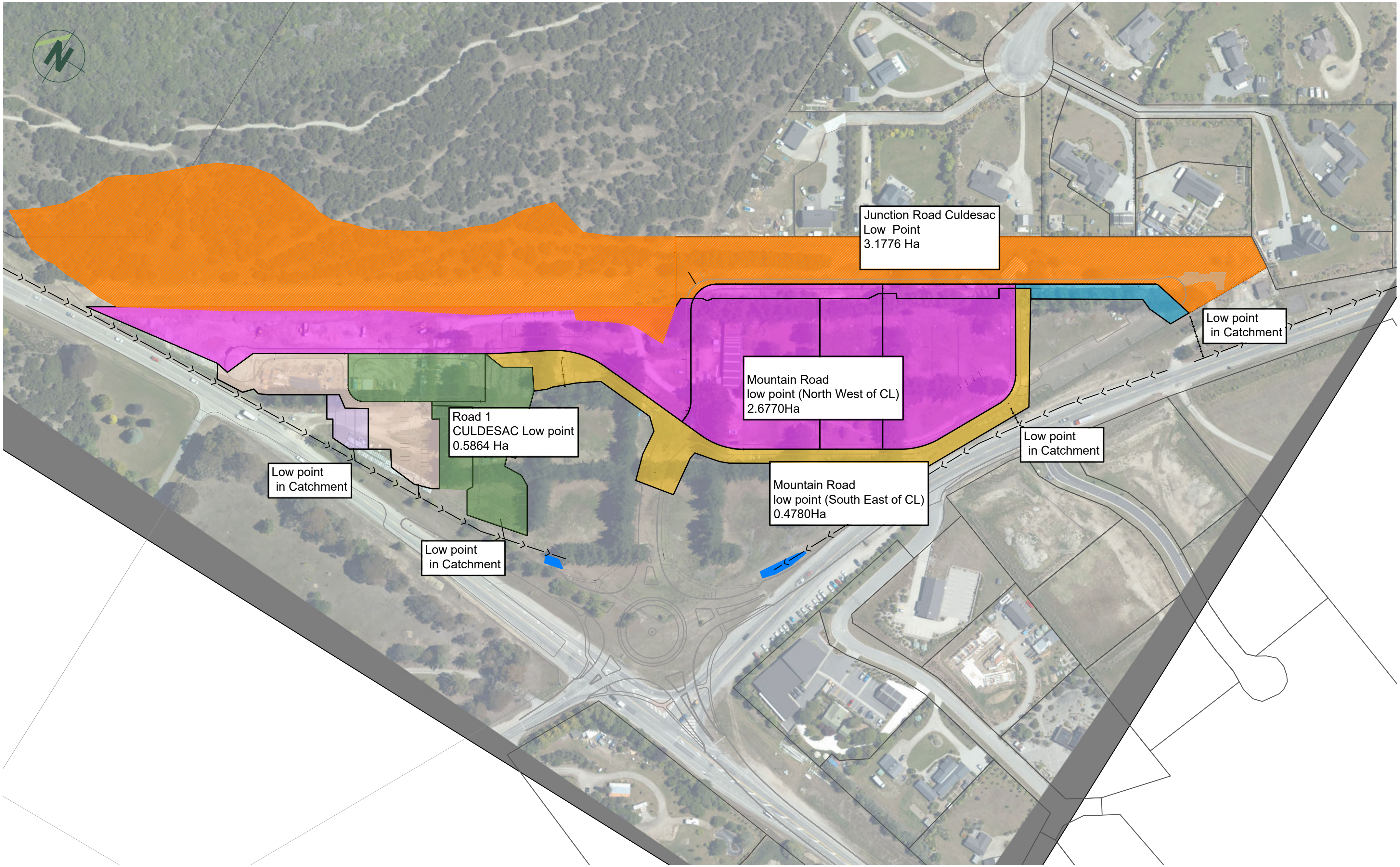
STATUS FOR INFORMATION

SURVEYED	LIDAR
DESIGNED	PHJ
DRAWN	PHJ
REVIEWED	TO BE REVIEWED
APPROVED	TO BE APPROVED

PROJECT	P240103
DRAWING NO	006
SHEET	435C
REVISION	1
SCALE (A3)	1:600



patersons. Land Professionals	CLIENT: MT IRON JUNCTION LTD, JUNCTION ROAD, WANAKA	DRAWING TITLE: ENGINEERING DESIGN, POST DEVELOPMENT CATCHMENT, SHEET C	
DATUM INFORMATION	COORDINATE SYSTEM: NZGD 2000 LINDIS PEAK CICUIT	DATUM: NZVD2016	ORIGIN OF COORDINATES / ORIGIN OF LEVELS
REV / DRAWN / DATE / NOTE	STATUS: FOR INFORMATION		
SURVEYED: LIDAR	DESIGNED: PHJ	DRAWN: PHJ	REVIEWED: TO BE REVIEWED; APPROVED: TO BE APPROVED
PROJECT: P240103	DRAWING NO: 006	SHEET: 435D	REVISION: 1; SCALE (A3): 1:600



patersons. Land Professionals

CLIENT	MT IRON JUNCTION LTD JUNCTION ROAD WANAKA	DRAWING TITLE	ENGINEERING DESIGN FLOOD CATCHMENTS
DATUM INFORMATION			
COORDINATE SYSTEM	NZGD 2000 LINDIS PEAK CICUIT		
DATUM	NZVD2016		
ORIGIN OF COORDINATES			
ORIGIN OF LEVELS			

REV	DRAWN	DATE	NOTE

STATUS FOR INFORMATION

SURVEYED	LIDAR
DESIGNED	PHJ
DRAWN	PHJ
REVIEWED	TO BE REVIEWED
APPROVED	TO BE APPROVED

PROJECT	P240103
DRAWING NO	006
SHEET	436
REVISION	1
SCALE (A3)	1:2000

APPENDIX B – MODELLING INPUTS

PRE-DEVELOPMENT

Drainage Area

Element Name	Element Type	Drainage Area (KM2)
Ex-C	Subbasin	0.027332
Ex-B1 & Ex-B2	Subbasin	0.037127
POS-B	Junction	0.037127
POS-C	Junction	0.027332
Ex-A1 & Ex-A2	Subbasin	0.105632
POS-A	Junction	0.105632

SCS Curve Number

Subbasin	Initial Abstraction (MM)	Curve Number	Impervious (%)
Ex-A1 & Ex-A2	4.6	63.9	0.0
Ex-B1 & Ex-B2	5	61	0.0
Ex-C	5	61	0.0

SCS Unit Hydrograph

Subbasin	Graph Type	Lag Time (MIN)
Ex-C	Standard (PRF 484)	7.1
Ex-B1 & Ex-B2	Standard (PRF 484)	8
Ex-A1 & Ex-A2	Standard (PRF 484)	6.67

POST-DEVELOPMENT

Drainage Area

Element Name	Element Type	Drainage Area (KM2)	Description
PD-I	Subbasin	0.009016	
Storage I	Reservoir	0.009016	
Diversion-7	Diversion	0.009016	
PD-H	Subbasin	0.004292	
Storage H	Reservoir	0.004292	
PD-G (Private)	Subbasin	0.001076	
Storage G	Reservoir	0.001076	
PD-F (Private)	Subbasin	0.001764	
Storage F	Reservoir	0.001764	
PD-J	Subbasin	0.020844	
Storage J	Reservoir	0.020844	
Diversion-1	Diversion	0.020844	
PD-E	Subbasin	0.016691	
Junction-11	Junction	0.037535	
Storage E	Reservoir	0.037535	
Diversion-9	Diversion	0.037535	
Diversion-10	Diversion	0.001764	
Junction-8	Junction	0.039299	
Diversion-11	Diversion	0.001076	
Junction-9	Junction	0.040375	
Diversion-12	Diversion	0.004292	
Junction-10	Junction	0.044667	
PD-POS-C	Junction	0.044667	
PD-POS-B	Junction	0.009016	
PD-A (Private)	Subbasin	0.000328	
Storage A	Reservoir	0.000328	
Diversion-2	Diversion	0.000328	
Junction-1	Junction	0.000328	
Reach-1	Reach	0.000328	
PD-B (Private)	Subbasin	0.002804	
Storage B	Reservoir	0.002804	
Diversion-3	Diversion	0.002804	
Junction-2	Junction	0.003132	
Reach-2	Reach	0.003132	
PD-C	Subbasin	0.014978	
Storage C	Reservoir	0.014978	
Diversion-4	Diversion	0.014978	
Junction-3	Junction	0.018110	
Reach-3	Reach	0.018110	
PD-POS-A	Junction	0.018110	

SCS Curve Number

Subbasin	Initial Abstraction (MM)	Curve Number	Impervious (%)
PD-A (Private)	2.3	80.6	0
PD-B (Private)	1.0	90.4	0
PD-C	1.3	88.7	0
PD-E	2.3	81	0
PD-F (Private)	2.3	80.8	0
PD-G (Private)	1.9	84.0	0
PD-H	1.2	88.8	0
PD-I	0.6	93.6	0
PD-J	5	61	0.0

SCS Unit Hydrograph

Subbasin	Graph Type	Lag Time (MIN)
PD-A (Private)	Standard (PRF 484)	6.67
PD-B (Private)	Standard (PRF 484)	6.67
PD-C	Standard (PRF 484)	6.67
PD-E	Standard (PRF 484)	6.67
PD-F (Private)	Standard (PRF 484)	6.67
PD-G (Private)	Standard (PRF 484)	6.67
PD-H	Standard (PRF 484)	6.67
PD-I	Standard (PRF 484)	6.67
PD-J	Standard (PRF 484)	6.67

APPENDIX C – MODELLING OUTPUTS

5% AEP FLOW OUTPUTS

Project: Mount Iron SW   Simulation Run: Pr-20yr24hr Nested

Start of Run: 01Jan2000, 00:00   Basin Model: Proposed
End of Run: 02Jan2000, 00:00   Meteorologic Model: 20-yr 24hr Nested
Compute Time: 05Feb2026, 15:53:12   Control Specifications: 24hr

Show Elements: All Elements   Volume Units: MM / 1000 M3 (selected)   Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (m3/s)	Time of Peak	Volume (1000 m3)
Diversion-1	0.02084	0.10416	1 January 2000, 12:10	0.38455
Diversion-10	0.00176	0	31 December 1999, 24:00	0
Diversion-11	0.00108	0	31 December 1999, 24:00	0
Diversion-12	0.00429	0	31 December 1999, 24:00	0
Diversion-2	0.00033	0	31 December 1999, 24:00	0
Diversion-3	0.0028	0	31 December 1999, 24:00	0
Diversion-4	0.01498	0	31 December 1999, 24:00	0
Diversion-7	0.00902	0	31 December 1999, 24:00	0
Diversion-9	0.03753	0	31 December 1999, 24:00	0
Junction-1	0.00033	0	31 December 1999, 24:00	0
Junction-10	0.04467	0	31 December 1999, 24:00	0
Junction-11	0.03753	0.27011	1 January 2000, 12:00	1.47934
Junction-2	0.00313	0	31 December 1999, 24:00	0
Junction-3	0.01811	0	31 December 1999, 24:00	0
Junction-8	0.0393	0	31 December 1999, 24:00	0
Junction-9	0.04038	0	31 December 1999, 24:00	0
PD-A (Private)	0.00033	0.00356	1 January 2000, 12:00	0.02131
PD-B (Private)	0.0028	0.0389	1 January 2000, 12:00	0.23341
PD-C	0.01498	0.19985	1 January 2000, 12:00	1.19357
PD-E	0.01669	0.18326	1 January 2000, 12:00	1.09479
PD-F (Private)	0.00176	0.01927	1 January 2000, 12:00	0.11516
PD-G (Private)	0.00108	0.01279	1 January 2000, 12:00	0.07617
PD-H	0.00429	0.05743	1 January 2000, 12:00	0.34324
PD-I	0.00902	0.13347	1 January 2000, 12:00	0.81311
PD-J	0.02084	0.12833	1 January 2000, 12:00	0.80327
PD-POS-A	0.01811	0	31 December 1999, 24:00	0
PD-POS-B	0.00902	0	31 December 1999, 24:00	0
PD-POS-C	0.04467	0	31 December 1999, 24:00	0
Reach-1	0.00033	0	31 December 1999, 24:00	0
Reach-2	0.00313	0	31 December 1999, 24:00	0
Reach-3	0.01811	0	31 December 1999, 24:00	0
Soak Pit A	0	0.00099	1 January 2000, 12:40	0.0212
Soak Pit B	0	0.0107	1 January 2000, 12:40	0.23245
Soak Pit C	0	0.04309	1 January 2000, 12:50	1.184
Soak Pit E	0	0.05911	1 January 2000, 13:00	1.4718

Soak Pit F	0	0.00601	1 January 2000, 12:30	0.1147
Soak Pit G	0	0.0045	1 January 2000, 12:30	0.07595
Soak Pit H	0	0.01708	1 January 2000, 12:30	0.34201
Soak Pit I	0	0.03432	1 January 2000, 12:40	0.80946
Soak Pit J	0	0.0078	1 January 2000, 11:20	0.4004
Storage A	0.00033	0.00099	1 January 2000, 12:40	0.0212
Storage B	0.0028	0.0107	1 January 2000, 12:40	0.23245
Storage C	0.01498	0.04309	1 January 2000, 12:50	1.184
Storage E	0.03753	0.05911	1 January 2000, 13:00	1.4718
Storage F	0.00176	0.00601	1 January 2000, 12:30	0.1147
Storage G	0.00108	0.0045	1 January 2000, 12:30	0.07595
Storage H	0.00429	0.01708	1 January 2000, 12:30	0.34201
Storage I	0.00902	0.03432	1 January 2000, 12:40	0.80946
Storage J	0.02084	0.11196	1 January 2000, 12:10	0.78495

1% AEP FLOW OUTPUTS

Project: Mount Iron SW   Simulation Run: Pr-100yr24hr Nested

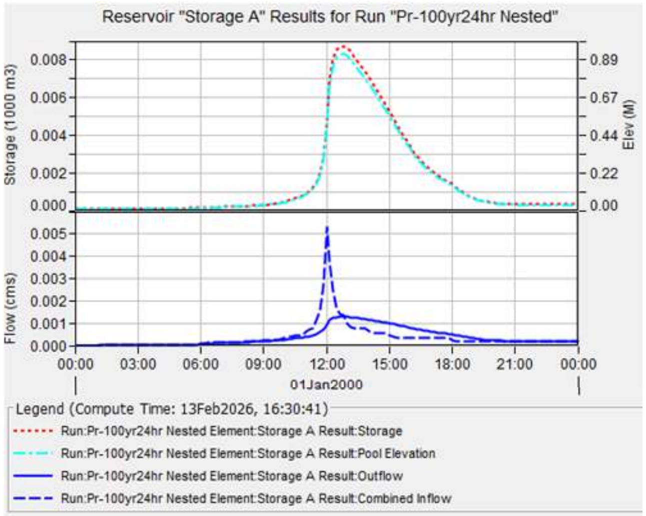
Start of Run: 01Jan2000, 00:00   Basin Model: Proposed
End of Run: 02Jan2000, 00:00   Meteorologic Model: 100-yr 24hr Nested
Compute Time: 05Feb2026, 15:57:08   Control Specifications: 24hr

Show Elements: All Elements   Volume Units: MM 1000 M3   Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (m3/s)	Time of Peak	Volume (1000 m3)
Diversion-1	0.02084	0.17595	1 January 2000, 12:10	0.81608
Diversion-10	0.00176	0	31 December 1999, 24:00	0
Diversion-11	0.00108	0	31 December 1999, 24:00	0
Diversion-12	0.00429	0	31 December 1999, 24:00	0
Diversion-2	0.00033	0	31 December 1999, 24:00	0
Diversion-3	0.0028	0	31 December 1999, 24:00	0
Diversion-4	0.01498	0	31 December 1999, 24:00	0
Diversion-7	0.00902	0	31 December 1999, 24:00	0
Diversion-9	0.03753	0	31 December 1999, 24:00	0
Junction-1	0.00033	0	31 December 1999, 24:00	0
Junction-10	0.04467	0	31 December 1999, 24:00	0
Junction-11	0.03753	0.42237	1 January 2000, 12:00	2.44437
Junction-2	0.00313	0	31 December 1999, 24:00	0
Junction-3	0.01811	0	31 December 1999, 24:00	0
Junction-8	0.0393	0	31 December 1999, 24:00	0
Junction-9	0.04038	0	31 December 1999, 24:00	0
PD-A (Private)	0.00033	0.00533	1 January 2000, 12:00	0.03175
PD-B (Private)	0.0028	0.05461	1 January 2000, 12:00	0.33074
PD-C	0.01498	0.28382	1 January 2000, 12:00	1.70736
PD-E	0.01669	0.27332	1 January 2000, 12:00	1.62829
PD-F (Private)	0.00176	0.02877	1 January 2000, 12:00	0.17142
PD-G (Private)	0.00108	0.01872	1 January 2000, 12:00	0.11164
PD-H	0.00429	0.08148	1 January 2000, 12:00	0.49058
PD-I	0.00902	0.18356	1 January 2000, 12:00	1.13171
PD-J	0.02084	0.21141	1 January 2000, 12:00	1.30061
PD-POS-A	0.01811	0	31 December 1999, 24:00	0
PD-POS-B	0.00902	0	31 December 1999, 24:00	0
PD-POS-C	0.04467	0	31 December 1999, 24:00	0
Reach-1	0.00033	0	31 December 1999, 24:00	0
Reach-2	0.00313	0	31 December 1999, 24:00	0
Reach-3	0.01811	0	31 December 1999, 24:00	0
Soak Pit A	0	0.00129	1 January 2000, 12:50	0.03154
Soak Pit B	0	0.01334	1 January 2000, 12:40	0.32901
Soak Pit C	0	0.05392	1 January 2000, 13:00	1.68758
Soak Pit E	0	0.08023	1 January 2000, 13:10	2.41739
Soak Pit F	0	0.00779	1 January 2000, 12:40	0.17056
Soak Pit G	0	0.00579	1 January 2000, 12:30	0.11125

Soak Pit H	0	0.02144	1 January 2000, 12:40	0.48837
Soak Pit I	0	0.04221	1 January 2000, 12:50	1.12526
Soak Pit J	0	0.0078	1 January 2000, 10:30	0.44939
Storage A	0.00033	0.00129	1 January 2000, 12:50	0.03154
Storage B	0.0028	0.01334	1 January 2000, 12:40	0.32901
Storage C	0.01498	0.05392	1 January 2000, 13:00	1.68758
Storage E	0.03753	0.08023	1 January 2000, 13:10	2.41739
Storage F	0.00176	0.00779	1 January 2000, 12:40	0.17056
Storage G	0.00108	0.00579	1 January 2000, 12:30	0.11125
Storage H	0.00429	0.02144	1 January 2000, 12:40	0.48837
Storage I	0.00902	0.04221	1 January 2000, 12:50	1.12526
Storage J	0.02084	0.18375	1 January 2000, 12:10	1.26547

STORAGE NODE GRAPHS – 1% 24 hour nested storm profiles



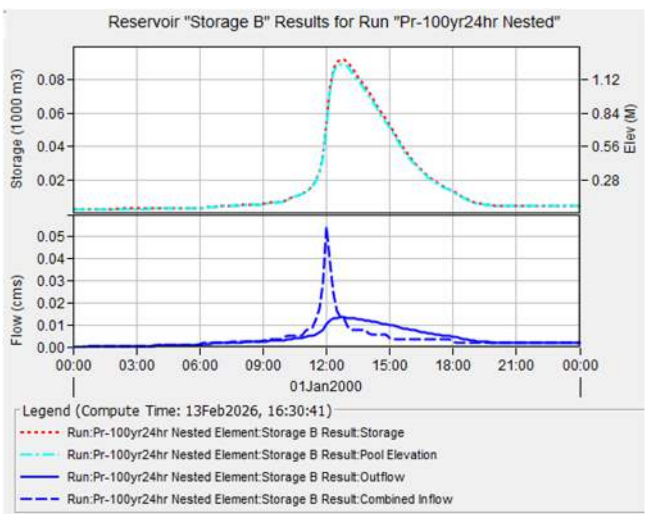
Project: Mount Iron SW Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage A

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	13Feb2026, 16:30:41	Control Specifications:	24hr

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	0.00533 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.00129 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:50
Inflow Volume:	0.03175 (1000 M3)	Peak Storage:	0.00874 (1000 M3)
Discharge Volume:	0.03154 (1000 M3)	Peak Elevation:	0.92309 (M)



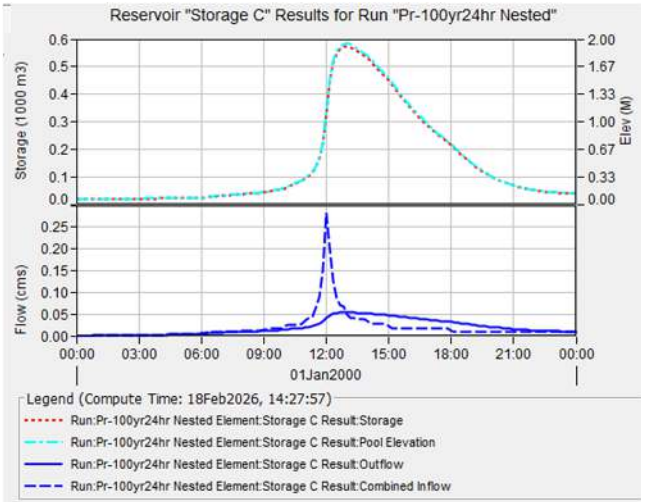
Project: Mount Iron SW Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage B

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	13Feb2026, 16:30:41	Control Specifications:	24hr

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	0.05461 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.01334 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:40
Inflow Volume:	0.33074 (1000 M3)	Peak Storage:	0.09183 (1000 M3)
Discharge Volume:	0.32901 (1000 M3)	Peak Elevation:	1.24674 (M)



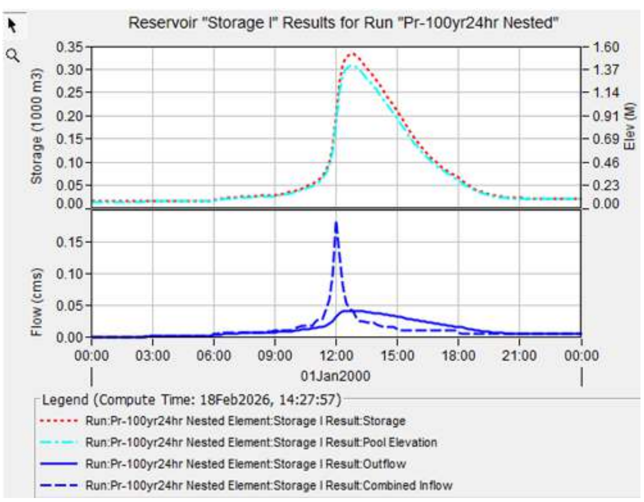
Project: Mount Iron SW   Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage C

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	18Feb2026, 14:27:57	Control Specifications:	24hr

Volume Units: MM 1000 M3

Computed Results

Peak Inflow:	0.28382 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.05392 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 13:00
Inflow Volume:	1.70736 (1000 M3)	Peak Storage:	0.57466 (1000 M3)
Discharge Volume:	1.68758 (1000 M3)	Peak Elevation:	1.94982 (M)



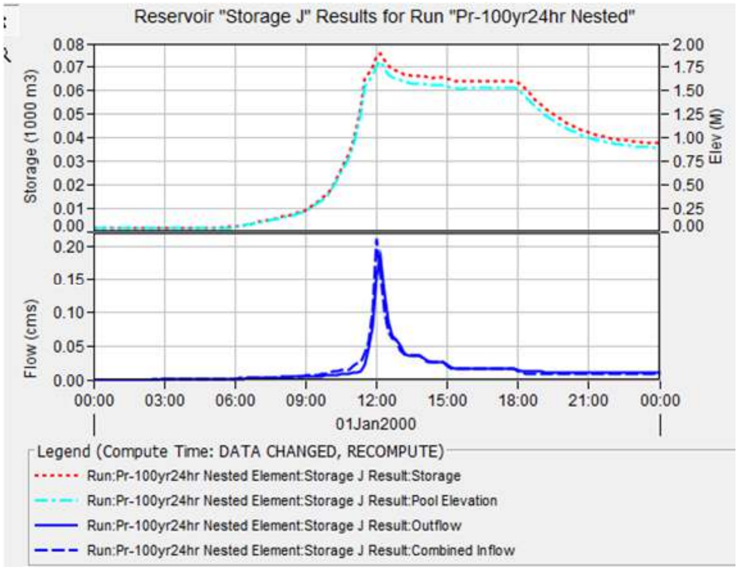
Project: Mount Iron SW   Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage I

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	18Feb2026, 14:27:57	Control Specifications:	24hr

Volume Units: MM 1000 M3

Computed Results

Peak Inflow:	0.18356 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.04221 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:50
Inflow Volume:	1.13171 (1000 M3)	Peak Storage:	0.33305 (1000 M3)
Discharge Volume:	1.12526 (1000 M3)	Peak Elevation:	1.40613 (M)



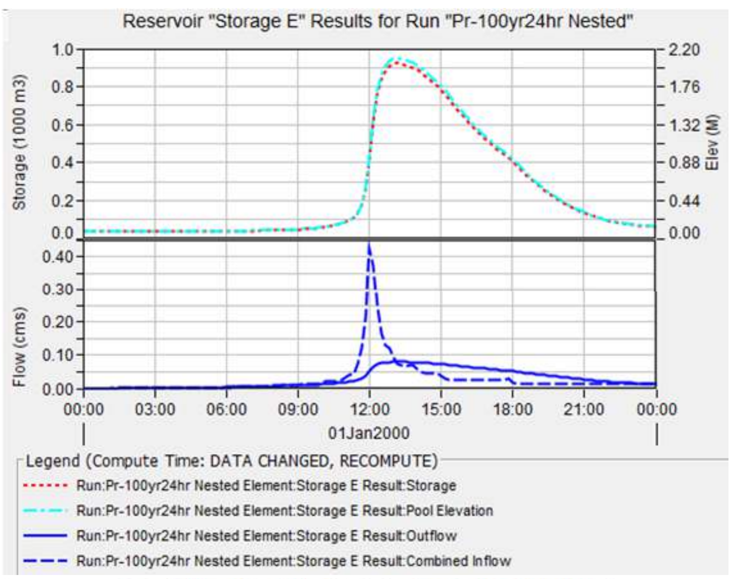
Project: Mount Iron SW Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage J

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	24hr

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	0.21141 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.18375 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:10
Inflow Volume:	1.30061 (1000 M3)	Peak Storage:	0.07607 (1000 M3)
Discharge Volume:	1.26547 (1000 M3)	Peak Elevation:	1.80649 (M)



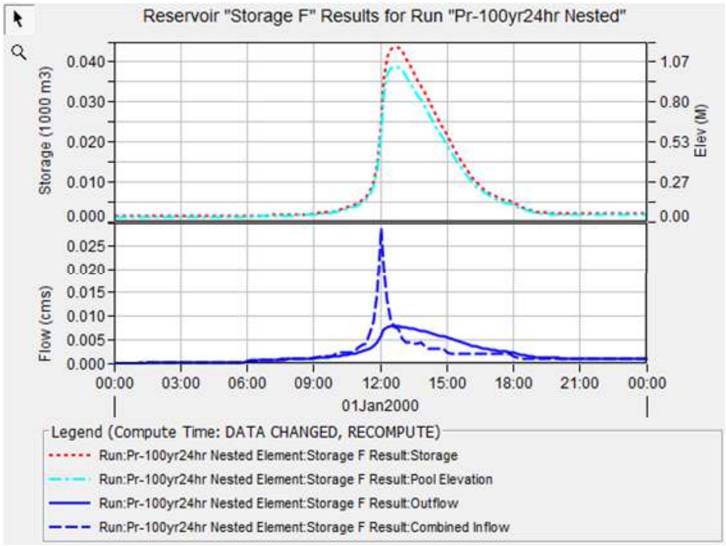
Project: Mount Iron SW Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage E

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	24hr

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	0.42237 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.08023 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 13:10
Inflow Volume:	2.44437 (1000 M3)	Peak Storage:	0.92419 (1000 M3)
Discharge Volume:	2.41739 (1000 M3)	Peak Elevation:	2.09048 (M)



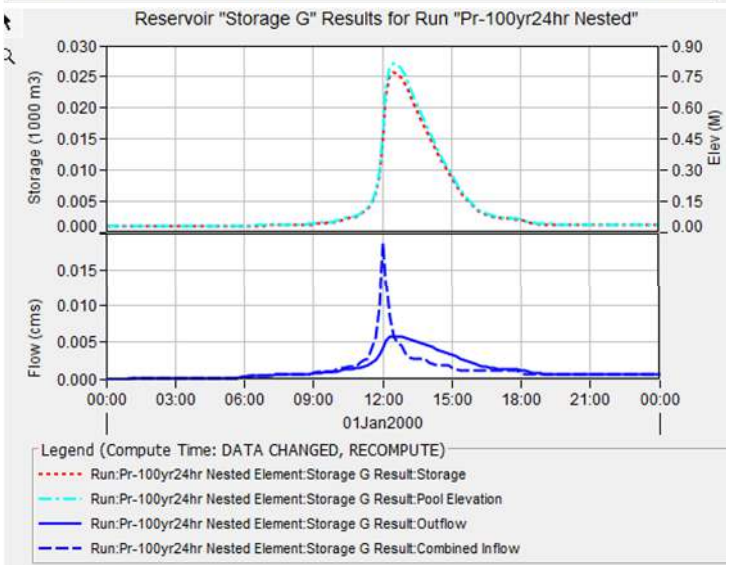
Project: Mount Iron SW   Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage F

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	24hr

Volume Units: MM 1000 M3

Computed Results

Peak Inflow:	0.02877 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.00779 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:40
Inflow Volume:	0.17142 (1000 M3)	Peak Storage:	0.04369 (1000 M3)
Discharge Volume:	0.17056 (1000 M3)	Peak Elevation:	1.03669 (M)



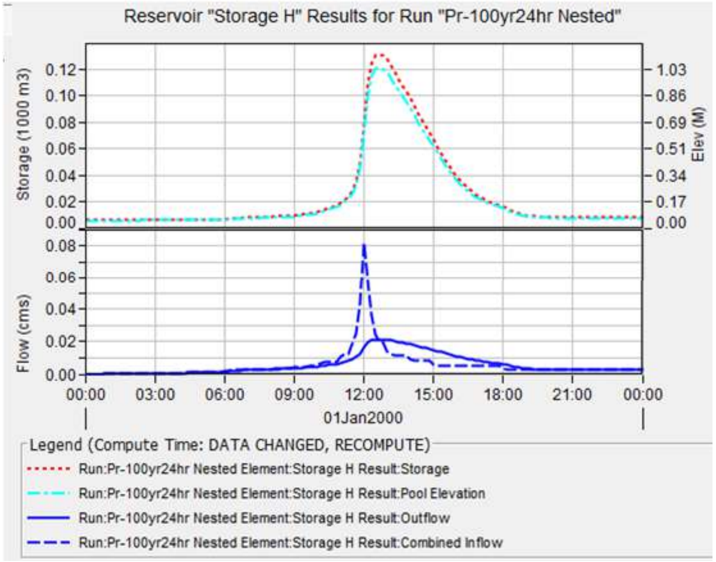
Project: Mount Iron SW   Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage G

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	24hr

Volume Units: MM 1000 M3

Computed Results

Peak Inflow:	0.01872 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.00579 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:30
Inflow Volume:	0.11164 (1000 M3)	Peak Storage:	0.02579 (1000 M3)
Discharge Volume:	0.11125 (1000 M3)	Peak Elevation:	0.81609 (M)



Project: Mount Iron SW Simulation Run: Pr-100yr24hr Nested
Reservoir: Storage H

Start of Run:	01Jan2000, 00:00	Basin Model:	Proposed
End of Run:	02Jan2000, 00:00	Meteorologic Model:	100-yr 24hr Nested
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	24hr

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	0.08148 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:00
Peak Discharge:	0.02144 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:40
Inflow Volume:	0.49058 (1000 M3)	Peak Storage:	0.13115 (1000 M3)
Discharge Volume:	0.48837 (1000 M3)	Peak Elevation:	1.03810 (M)

APPENDIX D POWER SUPPLY CONFIRMATION

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

07/11/2025 REF: 129212

Mark Laming
Power Solutions
mark@powerltd.co.nz

Dear Mark,

ELECTRICITY SUPPLY AVAILABILITY FOR A PROPOSED 250 LOT SUBDIVISION
10 MOUNTAIN ROAD, ALBERT TOWN .

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

NAFrear

Niel Frear
CUSTOMER INITIATED WORKS MANAGER

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

APPENDIX E TELECOMMUNICATION CONFIRMATION

Chorus NZ Ltd
4 Graham Street
Auckland CBD

Peter Joyce
Senior Surveyor
Patersons
Wanaka

26th November 2025

Hi Peter,

Thank you for providing an indication of your development plans in this area.

I can confirm that we have UFB fibre infrastructure in the area that you are proposing to develop. Chorus will be able to extend our network to provide connection availability to **"Mt Iron Junction Lot: 2, Deposited Plan: 605028, Otago".**

However, please note that this undertaking would of course be subject to Chorus understanding the final total property connections that we would be providing, roll-out of future property releases/dates and what investment may or may not be required from yourselves and Chorus to deliver any further infrastructure to and throughout the site in as seamless and practical way as possible.

The costs for any future works involved can only be finalised at the time that you are ready to proceed.

Chorus is happy to work with you on this project as the network infrastructure provider of choice. What this ultimately means is that the end customers (business and home owners) will have their choice of any retail service providers to take their end use services from once we work with you to provide the physical infrastructure.

Please reapply with a detailed site plan when you are ready to proceed.

Thanks

Kind Regards,

Danny Masterson
Business Development Manager
[redacted]
Chorus NZ Ltd