Technical Report No 3

Christchurch Southern Motorway Stage 2 and Main South Road Four Laning

Assessment of Stormwater Disposal and Water Quality

November 2012

Quality Assurance Statement			
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This Technical Report has been produced in support of the Assessment of Environmental Effects (AEE) for the Main South Road Four Laning and Christchurch Southern Motorway Stage 2 Project. It is one of 20 Technical Reports produced (listed below), which form Volume 3 of the lodgement document. Technical information contained in the AEE is drawn from these Technical Reports, and cross-references to the relevant reports are provided in the AEE where appropriate.

A Construction Environmental Management Plan (CEMP) has been prepared to provide the framework, methods and tools for avoiding, remedying or mitigating environmental effects of the construction phase of the Project. The CEMP is supported by Specialised Environmental Management Plans (SEMPs), which are attached as appendices to the CEMP. These SEMPs are listed against the relevant Technical Reports in the table below. This Technical Report is highlighted in grey in the table below. For a complete understanding of the project all Technical Reports need to be read in full along with the AEE itself; however where certain other Technical Reports are closely linked with this one they are shown in bold.

Schedule of Technical Reports for the AEE

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1	Design philosophy statement	4	
2	Traffic and transportation effects report	11	Construction Traffic Management Plan
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For further information on the structure of the lodgement documentation, refer to the 'Guide to the lodgement documentation' document issued with the AEE in Volume 1.

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Glossary of terms

Abbreviations used in this report

Abbreviation	Description
MSRFL	Main South Road Four Laning
CSM2	Christchurch Southern Motorway – Stage 2
ADT	Average Daily Traffic
AEE	Assessment of Environmental Effects
AEP	Annual Exceedance Probability
APHA	American Public Health Association
ARI	Average Recurrence Interval
ASTM	American Society for Testing Materials
Beca	Beca Infrastructure Ltd
BPO	Best Practicable Option
CCC	Christchurch City Council
CPW	Central Plains Water Enhancement Scheme
CSM	Christchurch Southern Motorway
CSM1	Christchurch Southern Motorway – Stage 1
CS&EMP	Contractors Social and Environmental Management Plan
E&SC	Erosion and Sediment Control
E&SCP	Erosion and Sediment Control Plan
ECan	Environment Canterbury the publicity name of Canterbury Regional Council
GHD	GHD Limited
GIS	Geographic Information Systems
HAIL	Hazardous Activities and Industries List
HIRDS	High Intensity Rainfall Data System
ICMP	Integrated Catchment Management Plan
LLUR	Listed Land Use Register
LiDAR	Light Detection And Ranging (A method of gathering contour data from aerial methods)
MfE	Ministry for the Environment

Abbreviation	Description
NRRP	Natural Resources Regional Plan
NZTA	NZ Transport Agency
OLFP	Overland flow path
RMA	Resource Management Act (1991)
RoNS	Roads of National Significance
SCS	United States Soil Conservation Service
SIMT	South Island Main Trunk (Rail)
SDC	Selwyn District Council
SH	State Highway
SWAP	South West Area Plan (CCC, April 2009)
WQL	Water Quality Level
WWDG	Waterways, Wetlands and Drainage Guide (CCC, 2003)

Executive Summary

The Project is for the upgrading of Main South Road to provide a four-lane median separated expressway along this existing arterial route (MSRFL) and for the construction, operation and maintenance of the Christchurch Southern Motorway Stage 2 (CSM2), a four-lane median separated motorway. There are associated changes to local roads and inclusion of three interchanges. This report describes the existing environment, the stormwater infrastructure proposed for the Project and the effect that it has on the environment.

Existing Environment

The local topography is gently undulating, with the surrounding land being predominantly rural, with some rural-residential, commercial and industrial areas. Constraints affecting the stormwater design of the proposed works are associated with the existing South West Area Plan (SWAP) (CCC, April 2009) water environment, the existing groundwater levels and protections zones, existing wells and the stockwater race network.

Proposed Infrastructure

The stormwater design philosophy includes separation of runoff from the Project, from the surrounding environment. Issues that are addressed in the design include the realignment of stockwater races, the land drainage function of the races, passage of overland flows and the effects of future groundwater level rises from the Central Plains Water Enhancement Scheme.

The collection and disposal system will typically consist of roadside swales or channels and stormwater disposal points at regular intervals along the Project. First flush basins and treatment ponds will also be required in some areas associated with the ECan 'less than 6 m to groundwater" zone identified in the Natural Resources Regional Plan. This zone is to the eastern end of the Project and is generally within the CCC boundary.

In addition, the discharge of stormwater to land will occur at numerous locations along the Project. This will be via infiltration through the base of the swales, via soak pits, through drainage pits, overland flow soak pits, Project ponds and from the base of overland flow and secondary siphon structures.

A further important element to the stormwater design is at Halswell Junction Road and Robinsons Road.

- At Robinson Road overpass (where Robinson Road will pass under CSM2) the runoff from carriageway water will be intercepted and treated then discharged to ground by gravity. However in future years when the effects of the Central Plains Water Enhancement Scheme occur and groundwater rises above a predefined critical level, surface water runoff will be pumped to land disposal some 300 m to the south. As groundwater rises further then an intervention strategy will be required and a second system of pumping of groundwater will commence. This will groundwater discharge will be to the adjacent stockwater race.

- At Halswell Junction Road there are numerous changes to the stormwater system. For the Project runoff, this includes the capture and diversion of flows to new ponds to be known as the Maize Maze Pond and the Ramp Ponds or to the existing Mushroom Pond. In addition there will be changes to CCC network including diversion of flows from Montgomery's Drain to the recently formed Owaka Basin, connections to Wilmers Quarry Basin and overflow connection back to Montgomery's Drain and modification of the overland flow path on Upper Knights Stream. In rare extreme flood events and when ponds are full, a spillway would operate and discharge surplus volume also to Montgomery's Drain. Following large rain events there will be controlled discharge from the ponds to drain them to allow sufficient volume for a subsequent storm event.

- At Halswell Junction Road, groundwater levels in future are projected to rise and due to the effects of Central Plains Water in conjunction with annual variation in groundwater levels. The combined effect will be to have groundwater levels rise above pond base level with the potential for groundwater inflows back into the ponds. In order to prevent this, a groundwater intervention strategy is proposed to intercept rising groundwater and to divert this flow by gravity to the Upper Knights Stream some 500 m downstream.

Environmental Impacts

The Project has the potential to impact on the existing environment with regards to water quality (stormwater runoff, groundwater and surface water), potential flooding issues, changes in the land drainage function of stockwater races and the water supply in groundwater wells.

The existing State Highway and local road network in the vicinity of the Project provides little in the way of formal stormwater quality treatment. Currently untreated runoff can also enter the environment via the stockwater race network. The Project design philosophy includes separation of runoff from the Project. It will be treated as it flows through the grass verges and along the treatment swales, prior to soakage to land. This stormwater treatment process will improve the receiving environment water quality.

The design of the two treatment ponds (Maize Maze and Ramp) mitigates the effects of contaminants generated in road runoff prior to discharge to the receiving environment.

The stormwater infrastructure has been designed to largely comply with rules in the NRRP and applies good industry practices. As such, it is considered that overall the effect of the discharges on water quality will be less than minor.

The design standard for the Highway drainage system is the 100 year Average Recurrence Interval (ARI) rainfall event including an allowance for climate change and as such it is not expected that there will be any adverse flooding effects as a result of the Project.

As a result of the Project there is a direct effect of closing of existing bores and wells beneath the Project footprint. On occasion, the bores can be closed, however for the balance, the NZTA will have an obligation to have drilled and tested new wells to service the owners of the severance land.

In addition there are a number of bores and wells in close proximity to the Project (i.e. those wells within 100m of the limit of the Project designation). An assessment of those wells has been carried out to ascertain what the potential impact the Project has on those wells. Closing and capping of some of these effected wells will also be required along with developing of new replacement wells outside of the influence of the Project. The new well construction will need to be undertaken in a timely manner to ensure the impacts of continuation of supply have been adequately allowed for.

Mitigation Measures

Mitigation measures are proposed to avoid or mitigate potential adverse effects discussed above.

Overall there are a number of aspects of the design philosophy which have been implemented to mitigate environmental effects including: the design standard applied, the dispersed drainage and disposal system, overland flow siphons and stockwater race conveyance pipes.

Another measure includes the implementation of a Construction Environmental Management Plan including erosion & sediment control measures to address how any discharges will be dealt with to minimise the impact on the environment.

Summary

Overall, the environmental impact of the proposed infrastructure will be minor due to the proposed mitigation measures.

1 Introduction

The Project is for the construction, operation and maintenance of the Christchurch Southern Motorway Stage 2 (CSM2), a four-lane median separated motorway. The Project includes the widening and upgrading of Main South Road to provide a four-lane median separated expressway along this existing arterial route (MSRFL). The Project also includes underpasses for multiple roads at (Weedons, Waterholes/Hamptons, Trents Roads, Shands, Marshs, Springs and Halswell Junction Roads), as well as an overpass at Robinsons Road. Additionally, associated on and off ramps at Halswell Junction Road, Trents Road, State Highway 1, Weedons Ross Road, a rear access road between Weedons and Robinson Roads. Associated local road changes and upgrades are also components of the Project.

Collectively the CSM2 and MSRFL and the associated works outlined above will be referred to as the Project.

This report describes the stormwater infrastructure proposed for the Project and the effect that it will have on the environment. In order to highlight the effects, a description of the existing environment is provided along with a description of the design philosophy. The various options, for the design philosophy considered in deriving the proposed design solution, are outlined, followed by a detailed description of the Project. Other topics covered in this report include: (i) an analysis of the Environment Canterbury regional rules relevant to the stormwater aspects of the proposal, erosion and sediment control, construction management, residual effects, residual risks, mitigation measures and (ii) a brief summary of consultation undertaken.

The report also describes the groundwater environment and its associated effects on stormwater management for the Project and the existing wells and boreholes that are or potentially affected by the project.

2 Proposal Description

The NZ Transport Agency (NZTA) seeks to improve access for vehicles and freight to and from the south of Christchurch via State highway 1 (SH1) to the Christchurch City centre and Lyttelton Port by constructing, operating and maintaining the Christchurch Southern Corridor. The Government has identified the Christchurch motorway projects, including the Christchurch Southern Corridor, as a road of national significance (RoNS).

The proposal forms part of the Christchurch Southern Corridor and is made up of two sections: Main South Road Four Laning (MSRFL) involves the widening and upgrading of Main South Road (MSR), also referred to as SH1, to provide for a four-lane median separated expressway; and the construction of the Christchurch Southern Motorway Stage 2 (CSM2) as a four-lane median separated motorway. The proposed construction, operation and maintenance of MSRFL and CSM2, together with ancillary local road improvements, are referred to hereafter as ‘the Project’.

2.1 MSRFL

Main South Road will be increased in width to four lanes from its intersection with Park Lane north of Rolleston, for approximately 4.5 km to the connection with CSM2 at Robinsons Road. MSRFL will be an expressway consisting of two lanes in each direction, a median with barrier separating oncoming traffic, and sealed shoulders. An interchange at Weedons Road will provide full access on and off the expressway. MSFRL will connect with CSM2 via an interchange near Robinsons Road, and SH1 will continue on its current alignment towards Templeton.

Rear access for properties fronting the western side of MSRFL will be provided via a new road running parallel to the immediate east of the Main Trunk rail corridor from Weedons Ross Road to just north of Curraghs Road. For properties fronting the eastern side of MSRFL, rear access is to be provided via an extension of Berketts Drive and private rights of way.

The full length of MSRFL is located within the Selwyn District.

2.2 CSM2

CSM2 will extend from its link with SH1 / MSRFL at Robinsons Road for approximately 8.4 km to link with Christchurch Southern Motorway Stage 1 (CSM1, currently under construction) at Halswell Junction Road. The road will be constructed to motorway standard comprising four lanes, with two lanes in each direction, with a median and barrier to separate oncoming traffic and provide for safety.[1] Access to CSM2 will be limited to an interchange at Shands Road, and a half-interchange with eastward facing ramps at Halswell Junction Road. At four places along the motorway, underpasses (local road over the motorway) will be used to enable connectivity for local roads, and at Robinsons / Curraghs Roads, an overpass (local road under the motorway) will be provided. CSM2 will largely be constructed at grade, with a number of underpasses where elevated structures provide for intersecting roads to pass above the proposed alignment.

CSM2 crosses the Selwyn District and Christchurch City Council boundary at Marshs Road, with approximately 6 km of the CSM2 section within the Selwyn District and the remaining 2.4 km within the Christchurch City limits.

2.3 Key design features

The key design features and changes to the existing road network (from south to north) proposed are:

- a new full grade separated partial cloverleaf interchange at Weedons Road
- a new roundabout at Weedons Ross / Jones Road
- a realignment and intersection upgrade at Weedons / Levi Road
- a new local road running to the immediate east of the rail corridor, to the west of Main South Road, between Weedons Ross Road and Curraghs Road
- alterations and partial closure of Larcombs Road intersection with Main South Road to left in only
- alterations to Berketts Road intersection with Main South Road to left in and left out only
- a new accessway running to the east of Main South Road, between Berketts Road and Robinsons Road
- an overpass at Robinsons and Curraghs Roads (the local roads will link under the motorway)
- construction of a grade separated y-junction (interchange) with Main South Road near Robinsons Road
- a link road connecting SH1 with Robinsons Road
- a short new access road north of Curraghs Road, adjacent to the rail line
- a new roundabout at SH1 / Dawsons Road / Waterholes Road

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[1] CSM2 will not become a motorway until the Governor-General declares it to be a motorway upon request from the NZTA under section 71 of the Government Roading Powers Act 1989 (GRPA). However, for the purposes of this report, the term "motorway" may be used to describe the CSM2 section of the Project.

- an underpass at Waterholes Road (the local road will pass over the motorway)
- an underpass at Trents Road (the local road will pass over the motorway)
- the closure of Blakes Road and conversion to two cul-de-sacs where it is severed by CSM2
- a new full grade separated diamond interchange at Shands Road
- an underpass at Marshs Road (the local road will pass over the motorway)
- providing a new walking and cycling path linking the Little River Rail Trail at Marshs Road to the shared use path being constructed as part of CSM1
- an underpass at Springs Road (the local road will pass over the motorway)
- a new grade separated half interchange at Halswell Junction Road with east facing on and off ramps linking Halswell Junction Road to CSM1, and
- closure of John Paterson Drive at Springs Road and eastern extension of John Paterson Drive to connect with the CSM1 off-ramp via Halswell Junction Road roundabout (east of CSM2).

The proposed alignment is illustrated in Figure 1 and encompasses the MSRFL and CSM2 alignments between Rolleston and Halswell Junction Road.

2.4 Surface Water, Stockwater, Wells and Groundwater

The project also impacts on the passage of surface water, stockwater races and their associated land drainage function and upon the groundwater. A number of the issues are interlinked.

The existing (or historical) groundwater regime has the groundwater (in general) below the zone of influence. However the Central Plains Water project has been consented and is part of the planning landscape. The extent and the timing of future groundwater levels remain uncertain. However in order to mitigate for the predicted effects then implementation of a groundwater intervention is required and this is proposed as part of the Project. This intervention strategy at Robinsons Road would take the form of pumping of groundwater and disposal to the stockwater race. At Halswell Junction Road, the intervention strategy is an infiltration trench and drainage by gravity to Upper Knights Stream some 0.5 km downstream. It is understood that 90% of the effects would be felt with 2 - 4 years of CPW completion[2]. As such we would expect much or all of the groundwater intervention works to be installed at the time of the initial Project construction.

The route crosses a number of existing stockwater races that are under the control of Selwyn District Council. The report addresses how each of the stockwater function will be maintained along with the land drainage component of the race network.

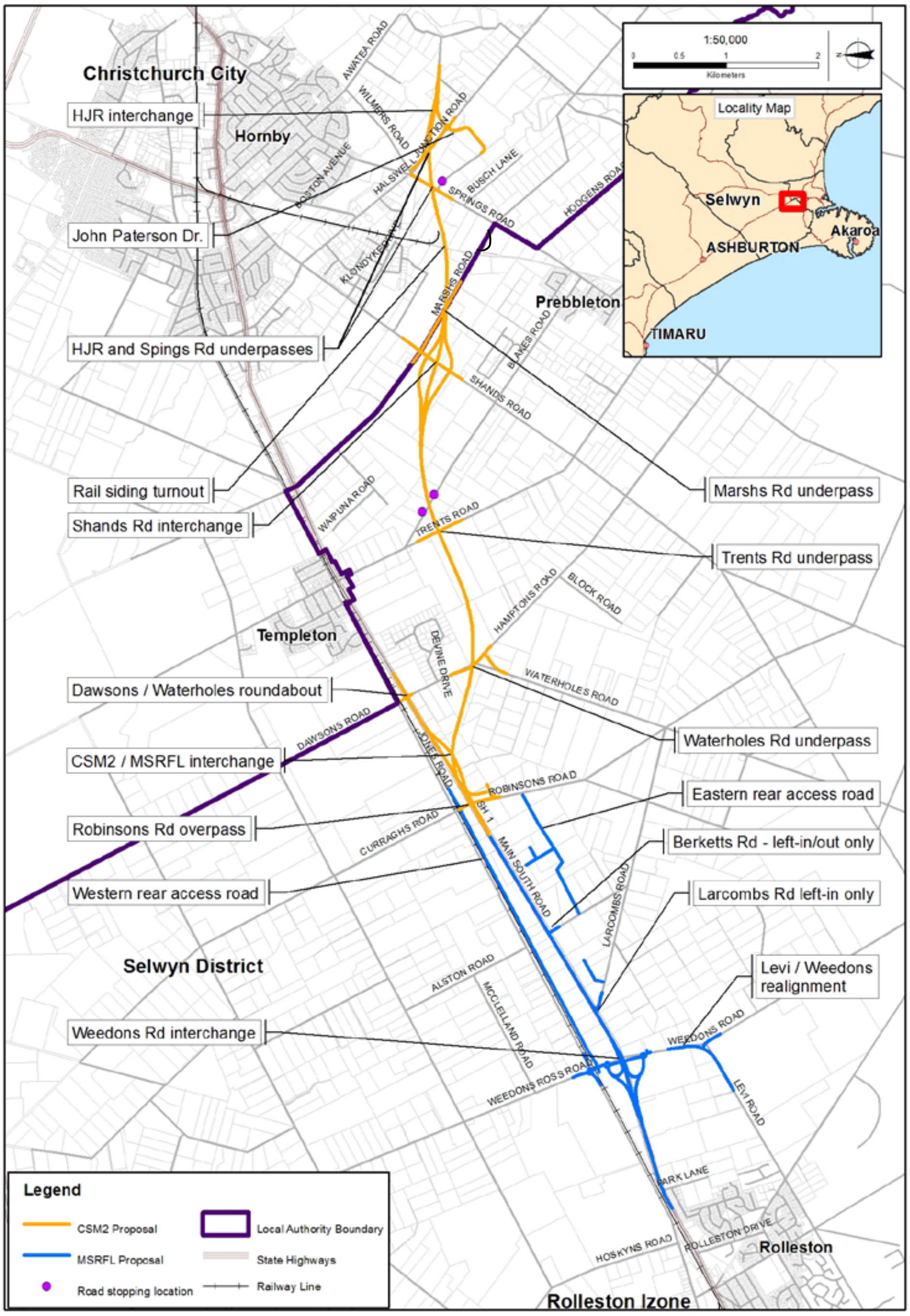
[2] Weir (CPW hydrogeologist) discussion with Mark Utting, Beca hydrogeologist, pers comm. August 2012

Overland flows have the potential to flow once the rainfall rate exceeds the ability of the ground to soak that water away. There is evidence of ponding but little evidence of large overland flow. The potential exists for these overland rotes to flow, which needs to be catered for.

There are no open water courses along the route. As such all Project stormwater runoff needs to collected, receive some treatment before discharge to ground, all without impact upon existing users of the adjacent land.

With discharges to ground there is a potential for impact of that discharge to have on groundwater and upon adjacent wells used for extraction. The report addresses how these impacts will be managed.

Figure 1 Proposal location map



2.5 Stormwater during Construction

Stormwater requires careful management during the construction phase of the Project and specific erosion and sediment control measures. The erosion and sediment control works have been the subject of preliminary design and both temporary and permanent stormwater infrastructure have been considered concurrently in this work.

Erosion and sediment control during construction are addressed in a specific management plan (Specialised Environmental Management Plan (SEMP) 002) appended to the Draft Construction and Environmental Management Plan (Volume 4 of the application documents). SEMP 002 should be read in conjunction with this report.

3 Existing Environment

The Project alignment will be described from Rolleston to Christchurch and with increasing chainage.

3.1 Introduction

The MSRFL follows the existing SH1. The Project continues adjacent to Park Lane and extends for 4.5 km from its southern extent, adjacent to Park Lane, to the north east where it joins with CSM2 approximately 400 m west of Robinsons Road. CSM2 is approximately 8.4 km long and extends from MSRFL east to the Halswell Junction Road / Springs Road intersection. This intersection is at the western extent of the CSM1 Project. For the purpose of this report the Project has been divided into the following sections from south to north:

- MSRFL – Adjacent Park Lane to Weedons Ross Road (Chainage 1350 m – 3100 m)
- MSRFL – Weedons Ross Road to CSM2 (Chainage 3100 m – 5875 m)
- CSM2 – MSRFL/CSM2 to Blakes Road (Chainage 0 m – 3800 m)
- CSM2 – Blakes Road to before Springs Road (Chainage 3800 m – 7100 m)
- CSM2 – Before Springs Road to CSM1 (Chainage 7100 m – 8400 m)

Appendix A contains Figure 11 which shows the proposed route and stockwater changes. The changes relate to the Project design which is set out in Chapter 6. Each section listed above is discussed in detail below after general comments that relate to the environment of the entire alignment.

3.2 General Environment

The topography of the Project area is gently undulating, sloping generally from south west to north east. The majority of the proposed route is within the Selwyn District, with a short section within the Christchurch City boundary to the north and east of Marshs Road.

The surrounding land is predominantly rural, but also includes residential, commercial and industrial zoned areas. Land use in the rural areas includes grazing (predominantly sheep and beef), stud farms, market gardens, nurseries, orchards, crops, and viticulture. A map of the land zoning is provided in Figure 12 of **Appendix A**.

Commercial areas include the shops at Templeton on SH1, Trents Road Winery, and businesses along Halswell Junction Road. There is also the industrial area to the northwest of Rolleston, and between Shands Road and Halswell Junction Road. Occasional commercial sites are dotted along SH1 between Rolleston and Waterholes Road.

Residential developments in the vicinity of the Project include the settlements of Rolleston, Templeton, Claremont Estate near Templeton, Aberdeen, Prebbleton and the outlying suburbs of Hornby.

There are various constraints that have had an effect on the civil and stormwater design of the proposed works. These include the existing South West Area Plan (SWAP) (CCC, April 2009) water environment, the groundwater protection zones for the area, and the existing wells and stockwater race network.

3.2.1 Geology

Technical Report 11 "Geotech Engineering and Geo-hazard Report" describes the underlying geology of the Project area as alluvial gravels and glacial outwash comprising various levels of sandy and silty loams. The gravels are of medium to high permeability and suited to the disposal of stormwater via soakage. This is advantageous to the stormwater design for the Project given the absence of surface waterways. Percolation testing has been carried out at various locations along the route and further testing will be carried out in areas of settlement and swale infiltration locations as the design phase progresses.

For a full description of the geology and geotechnical investigations undertaken refer to the Assessment of Environmental Effects Geotechnical Engineering and Natural Hazards Report (GHD, 2012).

The general topography for the Project area is characterised by flat alluvial plains. The alluvial material has been subdivided by Brown (1992) into alluvial sand and silt of historic river flood channels and underlying alluvial gravel and sand (and silt overbank deposits), both of the Yaldhurst Member of the Springston Formation. These have been laid by alluvial processes over the past 10,000 years.

The Springston Formation - Yaldhurst Member underlies the majority of the CSM2 and MSRFL route and consists of shallow low plasticity silts and clays, intermixed with fine sands. These soils are typically overlain by 0.1 – 0.3 m of topsoil and generally extend to a depth of between 0.1 to 2.2 m below ground level, although they were consistently encountered to depths of 1.5 to 3.5 m between the Project chainage 4500 m to 6600 m.

The silts have variable clay content, being defined as medium to low plasticity silt based upon Atterberg Limit tests. As the silts generally behave as fine to medium grained soils they have been described as predominantly non cohesive with a loose density. Where more cohesive material was encountered it was described as having soft to firm consistency. The delineation between the lower boundary of this unit and underlying sandy gravels is clearly defined.

The sandy gravel and sandy silty gravel of the Springston Formation - Halkett Member underlie the whole Project area, either from surface or below the shallower Yaldhurst Member where encountered. The Halkett Member was encountered at depths from between 0.1 – 2.2 m below ground level to the full depth of the investigation holes at 21.5 m below ground level.

A simplified soil profile has been adopted for the purposes of developing geotechnical parameters and design philosophies. These soil profiles vary according to the section of the Project and are derived from the exploratory hole located in that section. The profiles adopted are described as:

- Top soil
- Sandy silt
- Sandy gravel and
- Silty sandy gravel.

In terms of permeability, the surface profile indicates varying levels of permeability; however we expect the permeability to increase with increasing depth. Surface permeability's can vary significantly. The Assessment of Groundwater Effects report (**Appendix C**) confirms typical values of 10^{-5} to 10^{-8}m/s. The surface materials typically have lower permeability and the depths of the lower permeability material also vary. Surface lower permeable materials typically have depths between 0.5 to 3.5 m. However at depth and across much of the Project, zones of 1×10^{-3} to 8×10^{-3}m/s permeability rates can be expected. As such the soak holes will need to extend downwards until such more permeable zones can be encountered.

3.2.2 Existing Catchment

The Project alignment crosses the Canterbury Plains to the south of Christchurch. The ground appears near flat, but does rise gradually in elevation towards the foothills and Southern Alps. The plains have formed over geological time as river outwash fan. Evidence of this can be observed with the occasional ancient watercourse crossing the Project alignment. Over the Project alignment the ground level near Rolleston is RL = 45 m initially rising to RL = 51 beside Weedons Ross Road then falling to RL = 23 m beside Halswell Junction Road.

The majority of the catchment crossed by the CSM2 and MSRFL route does not directly contribute to any natural watercourse (T. Oliver & I. Haslop, ECan, pers comm., Oct 2010), (Andrew Mazey, SDC, pers comm., Aug 2011). This conclusion was reached in discussion with ECan and Selwyn District Council (SDC) staff (above) and is illustrated by the absence of watercourses in the vicinity of the Project. Surface water typically ponds in local depressions on the land surface and soaks to ground or evaporates. In larger events overland flows have the potential to flow along surface depressions (an overland flow path is shown in Figure 2). These overland flow paths are often intercepted by field drains, irrigation channels and the existing stockwater race network, which eventually discharge to the Halswell River or to land via engineered soak pits.

Discussions with SDC, ECan and the NZTA staff have confirmed that little anecdotal information on historic flooding is available but some surface water ponding has been observed within the catchment.

Figure 2 Existing Overland Flow Paths and Depressions

The SDC advises that stockwater races perform a land drainage function during heavy rainfall events. During or prior to such events, the upstream stockwater race intakes are closed or shut off. SDC advises that runoff can exceed water race capacity and some localised flooding does occur. The natural catchment upstream of the proposed MSRFL alignment is intercepted by SH1 and the railway embankment. Both of these structures form impediments to overland flows, particularly the railway embankment, and there is little existing stormwater infrastructure in place to allow for the passage of flood flows through or under Jones Road and the rail embankments. There is significant capacity for ponding upstream of these embankments.

All of CSM2 is within the Halswell catchment. The drainage and overland flow from the land surrounding Halswell Junction Road typically drains to land/soakage. In rainfall events where overland flow is generated it will discharge directly to the Halswell River via Montgomery's Drain and Upper Knights Stream (shown in Figure 3). Upper Knights Stream carries little or no flow except (1) at the end of large storm events when overland flow enters the drain, (2) when Halswell Junction Road Pond fills and spills from the service spillway.

The Halswell Junction Road pond is located North of the Roundabout with Springs Road. It is operated by CCC and collects stormwater from the industrial catchment to the North West. The overflow from this pond currently connects to Montgomery's Drain at the upstream end. This pond is included CCC South West Area Plan (SWAP) proposed stormwater infrastructure network.

The CSM Stormwater Management Review (NZTA & Opus, 2008) notes that periodic flows are not established in Upper Knight's Stream until approximately 2.3 km further downstream of the proposed alignment. ECan has stated that the Halswell River is sensitive to any increases in peak discharge rate or volume as there is a history of flooding. The ECan Engineer (Ross Vesey, pers. comm., Sept 2010) responsible for the Halswell River highlighted that flooding of the river was driven by slow response groundwater inflow. He also noted that the local community is actively engaged in its management and would be concerned with any additional contributions to the Halswell catchment area. Timing of potential discharges from the proposed stormwater system will be critical to minimising the effect of the discharges.

Figure 3 Montgomery's Drain Adjacent to Halswell Junction Road

The CCC has developed a South West Area Plan (SWAP) for this 8000 ha sector of Christchurch. This Plan was adopted by Council in April 2009. The SWAP outlines the proposed development of the area to the south west of Christchurch in the surrounds of Halswell, Wigram and Hornby. In conjunction with the plan there are a series of implementation projects including implementation of a Stormwater Management Plan (SMP).

The former Integrated Comprehensive Management Plan (ICMP (CCC, 2008)) was developed into a Stormwater Management Plan (SMP) and was prepared in support of a CCC application for network discharge consent. The application has been lodged, advertised and a hearing held in March 2012. There were no appeals and the ECan CRC120223 consent is now operative.

The SMP has been developed to establish naturalised waterways to improve water quality, better manage flood risks and enhance natural habitats. A network of naturalised stormwater facilities will be built throughout the area but the timing will be dependent on the progress of urban development within the catchment. The naturalised stormwater waterway will use soil adsorption, sedimentation and detention basins, wet ponds, swales and wetlands to treat and manage stormwater run-off before it enters the rivers and waterways. The extent of the proposed naturalisation can be seen in Plan 1 of the SWAP (CCC 2009) and replicated in this report in **Appendix A** as Figure 17.

In relation to the CSM2 Project, the Project alignment cuts diagonally across the flood plain and has the potential to divert surplus overland flow back to the Upper Knights Stream and hence into the upper reaches of the Halswell River. There is a history of flooding in the Halswell catchment where the critical duration storm is up to 60 hours in length (T.Oliver & I.Haslop, ECan, pers comm., Oct 2010) (R.Eastman & G.Harrington – CCC, pers comm. Mar 2012). The SMP prepared for this catchment recognises these longer duration events and promotes a series of measures that will permit development but restrict SW discharge rates.

60 hours is the duration at which no increase in runoff volume is permitted above existing volumes. This indicates that any discharges of stormwater from the Project system will have to be delayed for extended periods, given the slow response of the Halswell River catchment.

The CCC has identified Upper Knights Stream as a watercourse for corridor enhancement as part of the SWAP (CCC, 2009). The current state of the ecology of the Upper Knights Stream may not be representative of the ecological value of the stream in future years after the implementation of the SWAP. CSM Stormwater Management Review (NZTA & Opus, 2008) described the existing freshwater ecological state of the Stream aquatic communities as degraded. In terms of this Project, measures to improve the quality of discharge in line with the expected outcomes from the SWAP are described in Chapter 6.

The SWAP proposes to construct larger, interconnected stormwater devices including the Owaka Basin. The Owaka Basin is currently being excavated for the CCC. The Project stormwater design will maintained the functionality of the CCC system.

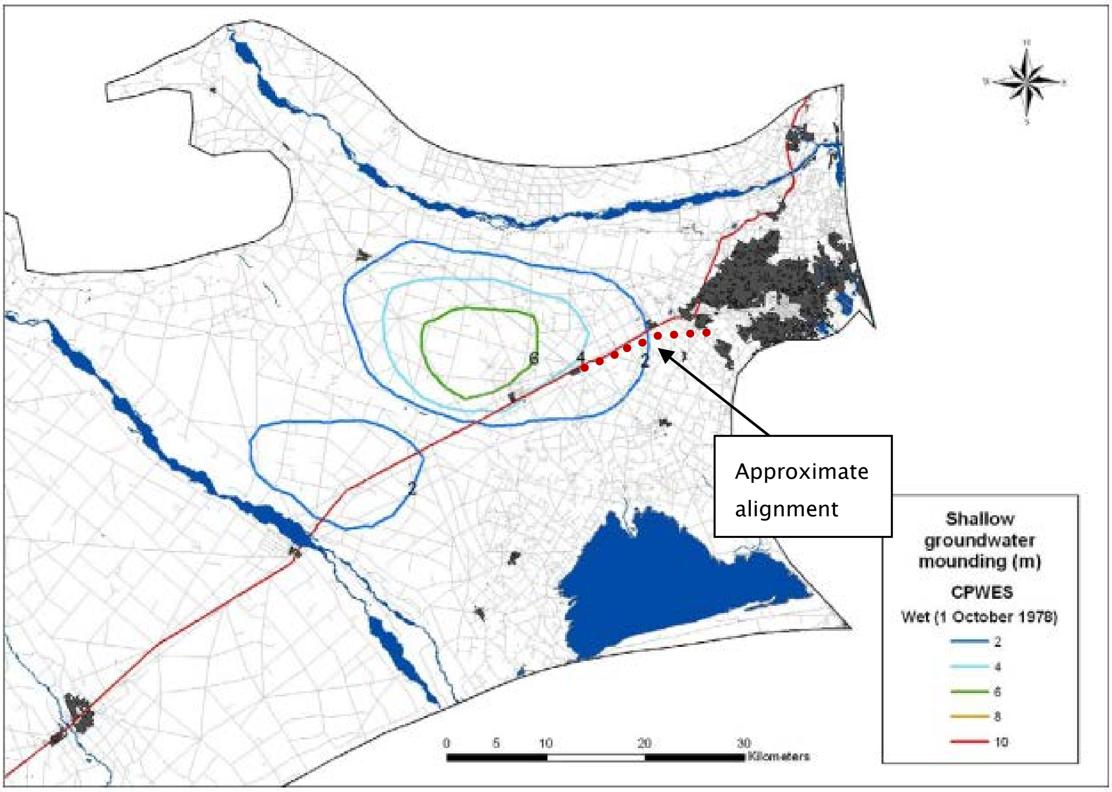
3.2.3 Groundwater

Groundwater in the Canterbury region varies, with the water table generally sloping towards the sea. Within the confines of the Project the water table gradient slopes form north west to south east, draining to Lake Ellesmere/Te Waihora (Golder Associates 2011) and some discharges to the east with some shallow discharge to local streams, including Upper Knights Stream. Typically the historical groundwater along the Project alignment is in the order of 12 – 15 m below ground at the Rolleston end and gradually rising to 3 m nearer Halswell Junction Road. Geotechnical investigations undertaken as part of the Project have not found any evidence of significant long term effects on groundwater levels due to the Canterbury earthquakes. For the entire route the groundwater aquifers are unconfined or semi-confined (as defined by the ECan planning maps as provided in Figure 13 of **Appendix A**). A confined aquifer is defined in the Natural Resources Regional Plan (NRRP) as an aquifer overlain with a lower permeability or impermeable layer where the water in the aquifer is under pressure. Geotechnical investigations undertaken as part of the Project have found no evidence of aquifer confinement.

As part of the Scheme Assessment Report, (SAR) (GHD/Beca, Mar 2010) has investigated the groundwater levels along the alignment of MSRFL and CSM2 to establish a design groundwater level. This allowed identification of any issues that would arise with the Project vertical alignment (and consequently the stormwater disposal system) being set below existing ground levels. This has been compared with other reported information. A design team target of the Project is to ensure the effective disposal of stormwater runoff whilst achieving the 1 m clearance between the disposal system and the design groundwater level as specified in the NRRP. The design groundwater levels have also taken into account the effects on groundwater levels arising from the Central Plains Water (CPW).

Evidence provided by CPW (Weir 2009) at the hearing for that project, on the effects of the scheme, indicates that there will be mounding of the groundwater (as shown in Figure 4). This mounding is predicted to be in the order of 4 m at the Rolleston end of the Project and over 1 m at the Christchurch end, at the CSM1 boundary. An allowance has been made for CPW effects in determining the Project's design groundwater level.

Figure 4 CPW Mounding Effects (Reproduced from Expert Evidence of Weir (2009))



In summary, the following steps have been undertaken to establish the design groundwater level profile for the Project during the SAR phase of the work:

1. The six existing ECan boreholes (as shown in Figure 5) adjacent to the Project area have been used to record water levels in the unconfined aquifer. These boreholes have been measured at

varying frequencies (both temporal and spatial variation) but typically monthly intervals. The record lengths vary, with the longest extending back to the 1950s.

2. As part of the Project, a further nine boreholes with piezometers were installed to various depths as set out in Geotechnical Factual Report (GHD-Beca, 2011) prepared for the Project, all with frequent (15 minute period) data loggers for the month of January. The piezometers recorded groundwater variations in the upper groundwater horizon free from influence from any deeper aquitard zones.

3. The ECan records were extrapolated using correlations of monthly 'all season' and 'non-irrigation season' maxima between the sites. There were rapid drawdown effects observed in some of the 'irrigation season' values. However there were only small differences between 'all season' and 'non-irrigation season' maxima with the peak value observed at each site were the 'non-irrigation season' maxima. This methodology allowed an estimation of the historical high groundwater levels at each site dating back to 1950.

4. The differences between the historical high levels and the levels recorded at the ECan sites during the period when the piezometric sites were in operation were established.

5. Maximum 'historical groundwater level highs' were established at the local piezometric sites closest to the ECan wells by adjusting the data for the difference established above.

6. Calculation of the historical groundwater level highs for the local piezometric sites along the alignment between the two locations, adjusted for the ECan sites, was undertaken and increased according to the comments from an independent peer review by Pattle Delamore Partners (2011).

7. A design groundwater level was established from the historical groundwater highs and the effects of CPW at each of the piezometric sites.

8. The design groundwater level was adjusted further using the findings of a Beca investigation into design groundwater levels for the CSM1 Project by utilising a level of 18.3 m RL (Beca 2011) at the CSM1 interface for the Project.

At the completion of this work package there was sufficient information to determine groundwater level for establishing project parameters. However there was still a level of uncertainty and a further work package to develop a groundwater model and test of these assumptions. This additional work package is attached as **Appendix C** to this report.

The key findings of the ground water report are

- The peak predicted level at Robinsons Road overpass following the full effects as allowed for by CPW is RL = 39.6 m. This is above the low point on the road centreline (RL = 39.48). There is a further prediction that for 5% of time the expected level will be above RL = 37.4 m.

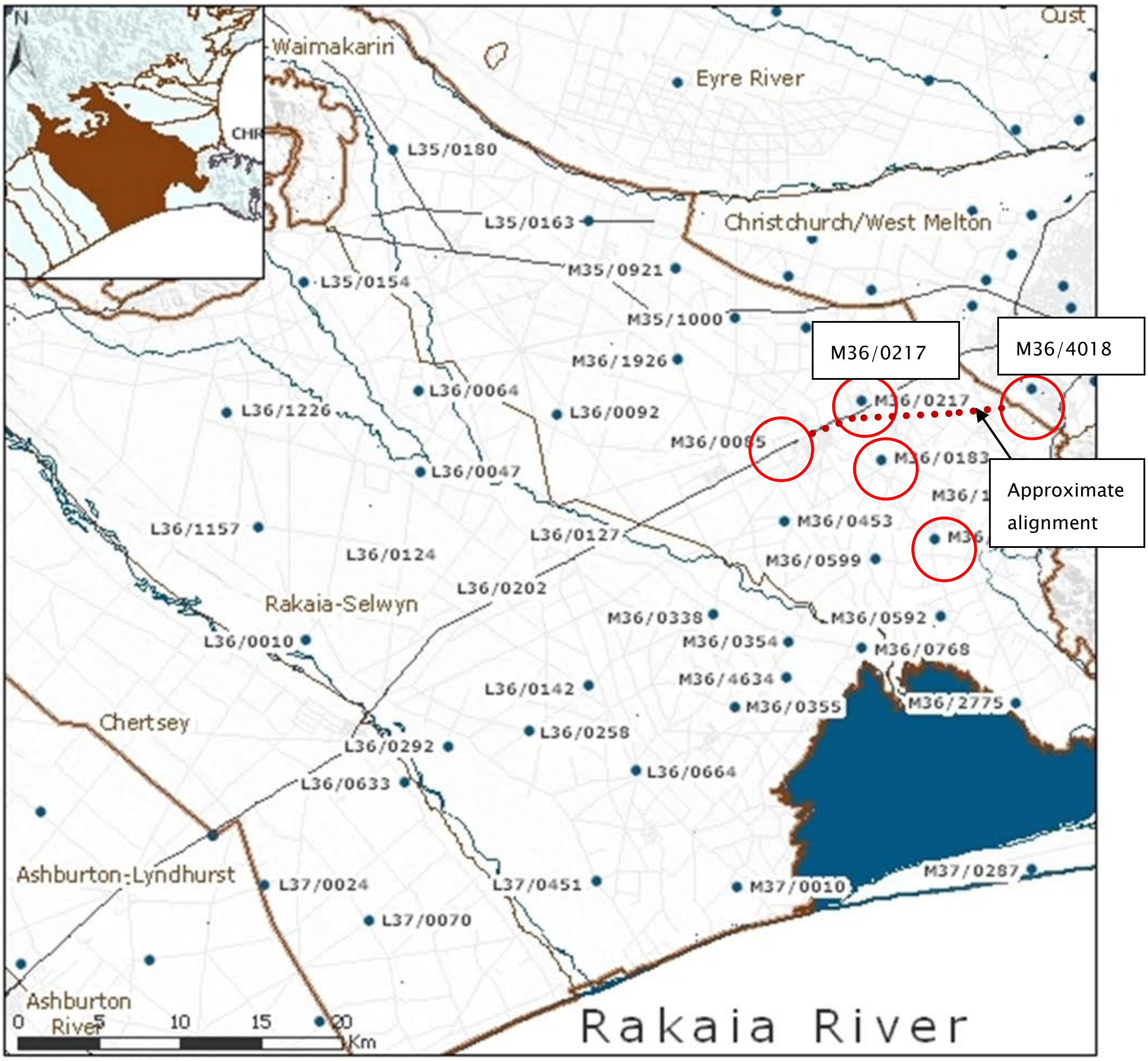
- The peak predicted level at Halswell Junction Road underpass following the full effects as allowed for by CPW is RL = 19.4 m. There is a further prediction that for 5% of time the expected level will be above RL = 18.8 m. This is below the low point on the CSM2 centreline (RL = 21.48) however this is above the Maize Maze Pond design invert level of RL = 18.75 m. There are further and similar implications on the Project Ramp Pond.
- The predicted groundwater highs have similar implications to the NZTA CSM1 Mushroom Pond and as well as on CCC Halswell Junction Road Pond, Owaka Basin, Lee Basin and Wilmers Quarry Disposal Area.

The implications and remedial works are discussed in further detail in subsequent sections of this report.

The CSM1 design groundwater values related only to the Mushroom Ponds (CSM1) and their immediate surrounds. The Beca study undertook a frequency analysis and investigated the correlation between groundwater highs and significant rainfall events to establish a design groundwater level for the design rainfall event. This study was developed prior to the CPW decision and as such does not take account of the future effects of the CPW project.

Full details regarding the establishment of the design groundwater level can be found in the GHD/Beca Stormwater Management and Disposal Options Report (2011). Figure 13 of that report is replicated in this report as **Figure 14** in **Appendix A**.

Figure 5 Groundwater Recording Sites (Reproduced from ECan Website)



Source: ECan Website October 2010

3.2.4 Stockwater Races

There are a number of stockwater races owned and operated by the SDC within the proposed route. An example is shown below in Figure 6. Refer to Figure 13 in **Appendix A** for a map showing the layout of the stockwater race network within 1 km of the Project. These are an important asset to the local users, being used for stockwater and irrigation, as well as providing for drainage of runoff. The stockwater races supply water through the area with some discharging to urban watercourses in Prebbleton approximately 3 km north west of the proposed alignment. The larger races discharge to streams in the Upper Halswell River Catchment, providing base flow to the Halswell River, while the smaller are controlled according to demand and these races terminate and drain to soak pits.

Figure 6 Existing Stockwater Race along Marshs Road

The proposed Project crosses nine existing water races. Two of these are along MSRFL and seven along CSM2. The race that flows through the Digga Link site (Weedons Road) crosses SH 1 downstream of that site and flows parallel with the MSRFL route on the eastern side and within the road reserve for approximately 2100 m. Currently this race also collects SH1 road run off.

In response to large rainfall events, SDC typically closes the inlets to the stockwater race network to increase the network capacity available to carry flood flows. This helps to reduce flooding of the race network and highlights the land drainage function of the network. Little quantitative information is known of the dry and wet weather flows in the system, however, qualitative comments from SDC staff (V. Rollinson, pers comm., Aug 2011) confirm that the system responds to wet weather to the extent that on occasion, localised flooding occurs (even when the upstream race gates are closed or restricted).

The SDC has advised that the larger stockwater races which drain to the Prebbleton urban watercourses and on to the Halswell River will need to remain in operation (M. England, A.Mazey, V. Rollinson, pers comm., Aug 2012). The remaining races could be closed by unanimous agreement from rate paying users and in discussions with the stockwater race committees. SDC will not permit discharge of surface water from the Project to the stockwater race network. SDC has also stated that the races can be culverted, if required, to accommodate the Project.

The SDC team also advise that the Marshs Road stockwater race is to become a land drainage race downstream of Shands Road. This race has no registered stockwater takes within the CCC boundary and as such can accommodate the change in race designation to “land drainage race”.

3.2.5 Existing Stormwater Infrastructure

There is little existing formal stormwater drainage infrastructure along the length of the Project, the exceptions being:

- Isolated soak pits along MSRFL.
- The swale and soak pit system constructed adjacent to the passing lanes outside of Rolleston. This system is discussed in more detail below in Section 3.3.1.
- The stockwater race network.
- More recent works constructed as part of the CSM1 and Halswell Junction Road upgrade works, being the Mushroom Pond and Lee Pond.
- The works proposed in the SWAP being the Owaka Basin and the culvert beneath CSM1 to accommodate discharge from the Owaka Basin to the Wilmers Road Quarry Disposal Area.

The site specific existing stormwater infrastructure is discussed in more detail in Sections 3.3 and 3.4. An example of a grassed swale along MSRFL is shown in Figure 7.

Figure 7 Existing Stormwater Infrastructure along Main South Road

3.2.6 Old Landfills and Quarry Sites

Within the CSM2 section of motorway, there is an old quarry pit on Springs Road to the south west of the junction with Halswell Junction Road. This parcel of land is currently owned by the Crown and has been identified in the past as a potential site for a stormwater retention basin. We understand this site would require significant land remediation to address contamination from past uncontrolled dumping. The invert of this pit is close to or below the calculated design groundwater level further limiting its potential for a disposal site. The site has not been proposed as a disposal area due to the risk of land remediation and potential difficulties with consenting for stormwater discharge to land. The site is on the ECan contaminated sites register.

A full discussion of this site can be found in Technical Report No. 16 Contaminated Land Assessment (NZTA, Sept 2011).

A second area of potential contamination which may influence the highway drainage system is at the northwest corner of the Robinsons Road intersection and is discussed in the TP16 (NZTA, 2012) report. There is an existing business where farming machinery and machinery parts are traded and stored. A risk of contaminated site runoff being generated has been identified and is discussed further in Section 6.3.1 and in the TP16 report. The site is also on the contaminated sites register LLUR[3] (ECan, 2012).

3.2.7 Wells

Based on information supplied by the ECan GIS data team (October 2010), there are a number of groundwater takes, predominantly for crop irrigation, that may be affected by the works. A map of the existing wells within 1000 m of the SAR alignment is provided in Figure 15 of **Appendix A**. The depth of wells range from 17 m (M36/0283 at Blakes Road and M36/2230 at Main South Road) to 177 m (M36/2405 at 108 Trents Road), however, the wells are typically from less than 20 m to 50 m in depth. There are wells at a depth of 6 m located in the Prebbleton area. This GIS data has been obtained from the ECan website.

As part of more recent abstraction of data from the ECan GIS database (22 August 2012), 87 active wells have identified as potentially affected as they are within a 100 m of the designation, and a further 16 active wells (not owned by the NZTA) directly affected as the ECan GIS database shows these wells being located within the Project extents. The wells identified are predominately used for domestic supply, stockwater, irrigation and to a lesser extent commercial use. Four directly affected wells within the Project extent have associated resource consents for groundwater take and use.

Of the 16 wells within the Project area there will be a variety of treatment. Some of these will come under NZTA control and will be closed and capped. Alternative supplies may need to be provided for the domestic and stockwater supply takes beneath the Project footprint, if required, for use on severed land. Some of the consents on the balance of wells will remain with the landowner for transfer to new wells outside the project footprint. Should the wells for consented takes be re-drilled at a significantly different location or depth this may require a change of conditions to the groundwater take resource consent held by the owner or occupier of the land.

A further 87 wells within a 100 m of the designation boundary have been identified. The selection of wells is to ensure capture of information where the location may be uncertain. In the detailed design phase each bore identified as being within 100 m of the designation will need to be identified and assessed to understand potential effects from the project, and to ensure there is adequate separation from the well and the locations of the Project temporary and permanent surface water to ground soakage locations. As the detail of this work is not currently known, and the accuracy of the ECan GIS database is uncertain, a consent condition will be required to identify the project discharge locations and confirm locations of the wells within the 100 m of designation boundary. For those wells where there are adverse effects that cannot otherwise be remediated or mitigated, then remedial work would be required to cap the existing well and develop a new well outside the zone of influence of the Project.

[3] The Listed Land Use Register (LLUR) is an electronic database which Environment Canterbury uses to store specific information about sites that have a past or present land use detailed on the Hazardous Activity and Industries List

A list of potentially affected wells within the designation plus 100 m buffer is attached to **Appendix E**.

3.2.8 Ecology

The following information is summarised from the Aquatic Ecology Assessment (GHD, August 2012) and Terrestrial Ecology Assessment (MWH, August 2012) reports TP17 and TP18. The fish communities at the five stockwater race sites sampled adjacent to Marshs, Weedons, Hamptons, Trents and Robinsons Roads, as part of the aquatic ecology survey, were depauperate (lacking species diversity) and limited to three species. The species were the native common and upland bullies (Gobiomorphus cotidianus and Gobiomorphus breviceps) and brown trout (Salmo trutta). No fish were caught in the Marshs Road stockwater race. Brown trout were caught at Weedons Ross Road and Robinsons Road races, upland bullies at Robinsons Road, Hamptons Road and Trents Road races and common bullies at all sites except Marshs Road. Both common and upland bullies are common throughout New Zealand waterways.

Upland bullies (along with short fin eels) were found to be the most common and abundant species in a survey of the waterways associated with CSM1 (EOS Ecology 2008) and within the South-West Christchurch area study (EOS Ecology et al. 2005). In addition, the Waterways and Wetlands Drainage Guide (WWDG) (CCC 2003) identifies these species as being common in Christchurch waterways. Upland bullies can be found upstream of substantial in-stream obstructions as it is a non-migratory species and therefore does not require access to the sea. Brown trout are also primarily a freshwater species and common bullies can also spend their entire lives in freshwater. These two species have the potential to migrate through the stockwater race network and as such fish passage should be maintained where practicable.

A full discussion on the ecology of the existing environment traversed by the Project can be found in the respective Technical Report No. 17 & 18.

3.2.9 Average Daily Traffic (ADT) Volumes

The ADT is the total number of vehicles on a road travelling in both directions on an average weekday. It provides an assessment of how “busy” a road is with the movement of people and freight. The predicted 2041 ADT volumes varies along Main South Road between 17,000 at Halswell Junction Road, peaking at 36,200 west of Marshs Road and dropping to 32,400 at Weedons Ross Road.

A full discussion on the traffic predictions for the existing environment in the vicinity of the Project can be found in Technical Report No. 2, Assessment of Traffic and Transportation Effects (GHD/BECA, May 2012). Table 6-1 is reproduced in part from TP2 and shows the predicted increase in traffic volumes with the CSM1 Project but not including the effect of the CSM2 and MSRFL Project. The purpose of providing this information in the Stormwater Report is to provide background of the potential contaminant sources. The major source of contaminants is from vehicles.

Table 1 Reproduced Table 7-1 from Technical Report 2[4]

Road and Location	Project			Baseline		
	2016	2026	2041	2016	2026	2041
Brougham St: West of Selwyn St	47,750	50,750	54,500	46,500	49,500	51,500
CSM1: Between Barrington St & Curletts I/C	46,250	51,000	55,750	43,500	47,250	49,250
CSM1: Between Curletts I/C & Halswell Jn Rd	39,250	47,750	54,750	33,000	37,250	40,750
CSM2: Between Halswell Jn Rd & Shands I/C	19,750	27,250	32,750	N/A*	N/A*	N/A*
CSM2: Between Shands I/C & MSR	16,000	21,750	27,000	N/A*	N/A*	N/A*
[Halswell Jn Rd: West of Springs Rd]	20,750	24,250	28,000	29,750	34,250	37,750
[MSR: South of Halswell Jn Rd]	16,250	20,000	23,250	30,250	35,750	40,500
[MSR: South of Marshs Rd/ Barters Rd]	17,000	20,750	24,000	28,000	33,250	37,750
MSR: South of Robinsons Rd/ Curraghs Rd	26,750	36,250	45,750	25,000	31,000	36,750
MSR: South of Weedons Rd/ Weedons Ross Rd	27,000	34,000	40,750	24,750	30,500	35,250

Road locations enclosed in [] are bypassed by the Project.
* CSM2 between Halswell Junction Road and Main South Road does not exist in the Baseline model.

Points to note include:

- Growth in traffic volumes once CSM1 becomes operational in 2016
- Significant increases in traffic projections independent of whether CSM2 proceeds.

3.2.10 South-West Area Plan

The South West Area Plan was made operative in 2009. The Plan Overview states:

> *"The South-West Christchurch Area Plan (the Area Plan) provides the framework for land use planning and public expenditure, reflects how the local community want the area to develop, and ensures that growth is integrated, collaborative and maintains intrinsic values. The Area Plan establishes a vision for the area, goals to achieve the vision, and objectives to meet the vision and goals."*

One of the conditions of the consent was for Council to make application for a Network Discharge Consent. This application was made, hearings held in March 2012, no appeals were received and the Discharge Consent CRC120223 is now operative.

[4] Project and Baseline ADT Volumes – RoNS Southern Corridor, as derived from the traffic model

As the SWAP relates to the Project, a range of works have been identified in the Plan. These include the Halswell Junction Road Pond, Owaka Basin and enhancements to Upper Knights Stream. A copy of the "Plan 1" is attached as Figure 17 to **Appendix A.**

3.2.11 River Breakout Scenarios

The CSM Stormwater System Management Review (TNZ / Opus, 2008) highlight that the Waimakariri River flood inundation scenario has a particularly low risk with regards to the CSM2 Project:

> *Environment Canterbury has applied for [and now secured] resource consent to upgrade the Waimakariri River flood protection Project. When this is complete the system will be capable of containing 6,500 cumecs (an estimated 1 in 10,000 year return flood event). This level of service will provide sufficient protection that the effects of flooding at Christchurch from the Waimakariri River can be ignored. Because the topography along the CSM route is so flat, secondary flow generally passes informally overland as sheet flow (i.e. shallow depth and low velocity).*

Given ECan is currently constructing the secondary stop bank system, the Waimakariri River breakout scenario has not been considered further in this design.

Topography dictates that there is not a risk of flooding for the Project from the Selwyn River.

3.3 MSRFL

The four laning of SH1 from near Robinsons Road (south of Templeton) to Rolleston, will extend over a distance of approximately 4.5 km. Typically the existing provision of road drainage is variable with sheet flow off the road discharging into adjacent swales or into adjacent properties. Formal soak pits are regularly used on the rural roads in the area and along the existing Main South Road as a method to dispose of surface water to ground.

On the recent widening and passing lane section on the MSRFL where the existing passing lane has been recently added between the Weedons Road intersection and Rolleston, a swale and soakage system has been designed and constructed, with gravel soak pits at approximately 200 m centres.

The 4.5 km stretch of existing SH1 forms an impediment to the overland flows generated in the catchment between the road and the railway. There was only one observed crossing (excluding stockwater races) beneath the railway between Rolleston and Templeton, which is a 300 mm diameter pipe adjacent to the Weedons Ross Road stockwater race. This pipe allows some passage of surface water from the upstream catchment beneath the rail corridor and towards SH1. In general the limited number of crossings beneath the rail embankment protects properties between the railway and SH1 from flooding.

The intention is to protect the Project drainage system from overland flows. Widening of the road corridor will displace some overland flow and paths, however, this will be offset by the collection of runoff from the Project which will be disposed to land via the existing road drainage system supplemented with additional soakage pits. The mitigation of this effect is discussed in Sections 6.2 and 9.

The existing environment for the MSRFL section of proposed work will now be discussed in more detail from a stormwater and erosion control perspective.

3.3.1 Adjacent Park Lane to Weedons Ross Road (Chainage 1350 m – 3100 m)

Refer to drawings 62236-A-C401 to C403, C407 and C408 in the Plan set contained in Volume 5.

3.3.1 A Description

From the traffic lights on SH1 in Rolleston at chainage 1000 m (approx.), the SH1 alignment climbs to the commencement of the Project at chainage 1350 m.

From chainage 1350 m where the Project starts, the vertical alignment rises to a point at chainage 1550 m adjacent to Park Lane. Park Lane heads east from SH1 at chainage 1550 m, 200 m north of where the SH1 four laning commences. From the high point, the carriageway alignment slopes gently down to the north to a crossroad intersection with Weedons Ross Road, located at chainage 3025 m.

There is limited stormwater infrastructure on the existing SH1 alignment. The exception to this is the passing lane between the Weedons Ross Road intersection and Rolleston where a swale and soakage system has been constructed (with gravel soak pits at approximate 200 m centres). At one of the soak pits on the north eastern side of the MSRFL, it appears that surface flow path is used to drain the adjacent field (between the railway and the road) to the road stormwater disposal system. A series of shallow swales is observed on both sides of the carriageway draining to land.

An existing stockwater race runs along the south side of Weedons Ross Road. This is controlled by a series of gates and grade control weirs upstream of the intersection with Jones Road. One leg of the race continues along Weedons Ross Road, under South Island Main Trunk (SIMT) rail along Weedons Road, under SH1 and continues down Weedons Road. The second leg deviates 150 m north of Weedons Ross Road, turns and passes under Jones Road, SIMT rail, through the Digga-Link site to SH1 where it turns and traverses east along SH 1.

There are several existing private properties on both sides of the carriageway. There is a large substation located at the south west corner of the Weedons Ross Road / Jones Road intersection.

The existing SH1 road horizontal and vertical alignment will be generally maintained, generally falling gently from a high point in the carriageway just north of Park Lane. A shallow under vertical occurs just south of Weedons Road intersection.

3.3.1 B Catchment

At the southeast extent of the Project, the MSRFL alignment rises from chainage 1350 m to the high point at chainage 1750 m. In event of failure of the swales and soakage system the surplus overland flow would travel south east and into Rolleston.

An industrial estate locally known as the 'I Zone' is located to the south west of the Project. The I Zone is at a similar elevation to the Project commencement but does not contribute to the MSRFL catchment. The area of land between the rail and SH 1 is small and again would not contribute to overland flow. Thus for the earlier part of the Project we do not anticipate any overland flow contributing to the Project from upstream catchments.

The railway embankment forms an obstruction to overland flows from the south west and an absence of culverts of reasonable diameter suggests that little, if any, stormwater surface runoff from west of the railway will reach SH1

The catchment of this stretch of SH1 is gently undulating farmland sloping from south west to north east. There are minimal impervious surfaces in the catchment area and the small portion of surface water runoff will be captured by the existing stockwater race at Weedons Ross Road.

Flooding of the catchment upstream of the rail embankment i.e. above Jones Road, is unlikely to overtop the rail embankment in the design event. Any overland flow is likely to be captured by the stockwater race and fed under the rail embankment either at Weedons Ross Road or through the culvert under SIMT rail to the Digga-Link site.

Further discussion of this residual risk can be found in Section 8.

3.3.1 C Stormwater Design Constraints

There is an existing embankment on the rail side of the MSRFL, towards the Park Lane end of the site, which runs parallel with SH1 before tying into higher ground prior to the Weedons Ross Road interchange. Overland flow in the land between the railway and SH1 concentrates to a dip in the embankment where it discharges to the highway drainage system. In events exceeding the capacity of the soak pit, flooding of the current stormwater infrastructure would potentially occur. Flows would be transferred to the low point in the existing alignment near Weedons Ross Road where it would continue to flow over Weedons Ross Road into the adjacent stockwater race and beneath SH1 to the south.

Should the stockwater race be at capacity or blocked there is potential for further overtopping onto SH 1 carriageway at Weedons Intersection.

3.3.1 D Groundwater

The historical groundwater level at the Weedons Ross Road intersection is approximately RL 36 m, approximately 15 m below existing ground level. An allowance for historical maxima (approximately 7 m) and CPW (approximately 4 m) established the design groundwater level of approximately RL 46 m or 4 m below ground.

3.3.2 Weedons Ross Road to CSM2 (Chainage 3100 m – 5875 m)

Refer to drawings 62236-A-C403 to C406 in the Plan set contained in Volume 5.

3.3.2 A Description

This portion of SH1 is from Weedons Ross Road to the CSM2 extent of physical works 400 m before Robinsons Road. There are two intersections: Larcombs Road and Berketts Road. The surrounding land is generally farmland with associated dwellings and structures.

The MSRFL alignment will maintain the existing grade sloping gently from south west to north east.

There are two existing stockwater races in the vicinity of Weedons Ross Road:

- One race adjacent to Weedons Ross Road: this race continues to the south east, crosses SH1 and continues flowing parallel to Weedons Ross Road.
- A second race that arrives to the north west of SH1 chainage 3175 m: this race turns east and conveys parallel to SH1 to chainage 3475 m (on the north side of the carriageway) where it crosses below the existing carriageway heading south into farmland.
- A branch of the second race heads east parallel to the edge of SH1 until chainage 5150 m, before turning south. Further branches also head south from this line at a point adjacent to Larcombs Road and at chainage 4750 m.

A single stormwater culvert under SH1 has been identified along the MSRFL route at the Digga-Link site adjacent to Weedons Ross Road on the north west corner of the existing Weedons Ross Road intersection. The culvert is above a natural low point in the topography between two adjacent stockwater races to the immediate north and south. The culvert is approximately 450 mm diameter and drains to land on the south side of SH1. The culvert will require extension and/or replacement depending on the depth of the final pavement design.

Six potential overland flow paths have been identified from the west. These are located in low points in the existing topography and have the potential to convey overland flow in extreme storm events.

To the west of the rail is Jones Road. At the intersection of Weedons Ross and Jones Roads, the existing road corridor is of insufficient width to accommodate the proposed upgrade works. The Project works will require the relocation of services including a power pole line and the existing stockwater race elements.

3.3.2 B Rear Access Roads

To the immediate east of the rail, is a strip of land which has been identified as the Western Rear Access Road route between Weedons and Robinsons. The land is largely flat yet includes mild undulations that reflect the original topography prior to construction of the rail and roads in the area. There are a number of low points along this route that have been inferred as ancient stream channels that once flowed over the plains.

To the immediate east of the SH1, is a strip of land which has been identified as the Eastern Rear Access Road route between Larcombs and Robinsons. The land is largely flat yet includes mild undulations that reflect the original topography prior to construction of the roads in the area.

3.3.2 C Catchment

The catchment of this stretch of SH1 is gently undulating farmland sloping from south west to north east. There are minimal impervious surfaces in the catchment area and a portion of surface water runoff will be captured by the network of existing stockwater races.

The distance between the MSRFL and the railway has increased, compared to the section of SH1 between Park Lane and Weedons Ross Road discussed in Section 3.2.1. There is an existing 750 mm diameter culvert beneath the railway north of this section of SH1. This allows stormwater flow to pass into the catchment of SH1, however, Jones Road will be an impediment to overland flows reaching the railway, therefore the culvert is likely to only pass high flows (in extreme rainfall events) beneath the railway towards SH1.

The existing 450 mm diameter culvert at the Digga-Link site services a small catchment of low topography between two existing stockwater races.

3.3.2 D Stormwater Design Constraints

The existing SH1 road alignment will be maintained. The existing road centreline is currently an impediment to overland flow paths and will remain so post construction.

There is a super elevation in the road carriageway adjacent to and just past the Weedons Ross Road interchange. Surface water runoff from the existing road surface will flow to the north only.

3.3.2 E Groundwater

The historical groundwater level at the Weedons Ross Road intersection has been measured at RL 36 m (approximately 15 m below existing ground level of RL = 51 m). An allowance for historical maxima (approximately 7 m) and CPW (approximately 3 - 4 m) established the design groundwater level of approximately RL 47 m or 4 m below ground.

The groundwater level at the CSM2 connection / Robinsons Road has been measured at RL 32 m (approximately 13 m below existing ground level of RL = 45.2 m). An allowance for historical maxima (approximately 6 m) and CPW (approximately 3 m) established the design groundwater level of approximately RL 41 m or 4 m below ground.

3.4 CSM2

The CSM2 alignment crosses a number of surface flow paths (e.g. old river braids), which have the potential to carry overland flows in extreme events. These have been identified using the Project survey and long section of the alignment and by using aerial photography.

The path of the old stream channels outside of the corridor has not yet able to be defined because the lack of field information, the absence of LiDAR or aerial photogrammetry. As a result we have not been able at this stage to precisely define catchment areas that potentially contribute to the flow paths. The land for which the catchment information is required is privately owned. LiDAR has been recommended as the best way to achieve a good level of confidence as to the individual catchment extent and characteristics.

What are known are the locations of the culverts that pass beneath the railway embankment to the west. This embankment forms a natural barrier to overland flow from upstream of that embankment. There are only a very limited number of culverts constructed beneath the rail. These culverts are generally small in diameter and as such do not have the capacity to convey large flows under the rail embankment. Overland flow is predicted to pond upstream of the rail embankment and the amount of water able to pass under the embankment is predicted to be relatively small and thus unable to have a significant impact upon the proposed CSM2 Project. The Project will still have to accept overland flow from upstream of the Project and pass this flow beneath the Project.

Site inspections revealed three crossings beneath the railway embankment as listed below in Table 2 (in addition to the crossing at Weedons Ross Road described above and excluding stockwater races). The existing railway culvert at 1096 Main South Road (of approximately 750 mm diameter) is between Waterholes Road and Robinsons Road. The catchment of this culvert is small with Jones

Road providing a further barrier between the paddocks to the north west and the railway culvert. There is no corresponding culvert beneath SH1 downstream of the railway culvert.

The two northern most crossings are small diameter and elevated above the adjacent ground level or isolated from nearby overland flow channels by topography or Jones Road. As such, the capacity of these culverts is limited and has not been considered further in the design given that the runoff from between SH1 and CSM2 is considerably more dominant in the sizing of the overland flow siphons.

Table 2 Railway Crossings

Location (approximate MSRFL/CSM2 chainage m)	Diameter (mm)	Comment
1096 Main South Road (CSM2 900)	750	Limited catchment area as described in text above
Kissel Street (CSM2 3500)	300	Crossing beneath railway in Templeton with significantly less capacity than required for catchment between SH1 and CSM2.
784 Main South Road (CSM2 3700)	225	Crossing beneath railway in behind layby with significantly less capacity than required for catchment between SH1 and CSM2.

The CCC has proposed new stormwater infrastructure in the SWAP Stormwater Management Plan (SMP) (CCC, 2011) in the vicinity of Sections 1 and 2 of CSM2 between Halswell Junction Road and Marshs Road. The SMP includes existing and proposed infrastructure potentially affected by the proposed alignment including:

- Halswell Junction Road Detention Basin
- Owaka Basin
- Montgomery's Drain
- Wilmers Road Quarry disposal area.

In general the stormwater proposals in the SWAP SMP (CCC, 2011) will remain unaffected by CSM2. Siphoning of Montgomery's Drain beneath CSM2 and Halswell Junction Road will be required to maintain the function of the SWAP proposed stormwater infrastructure.

Infrastructure built as part of CSM1 will be impacted by CSM2. Most notably being:

- The Mushroom Pond and its overflow to the culvert beneath CSM1
- Lengthening of the culvert beneath CSM1 and

- The Lee Basin.

Allowances for modifications to this infrastructure have been made in the design of CSM2.

The alignment will intersect an open drain along the eastern edge of Springs Road. As Springs Road is to be elevated over the Project alignment, the open drain will need to be realigned around the extent of the underpass embankments and culverted under the motorway.

3.4.1 MSRFL/CSM2 to Blakes Road (Chainage 0 m – 3800 m)

Refer to drawings 62236-B-C401 to C407 in the Plan set contained in Volume 5.

3.4.1 A Description

The CSM2 extent of works commences approximately 400 m west of the existing Robinsons Road interchange. From this intersection, the proposed alignment of CSM2 diverts south into greenfield land. The alignment will cross existing farmland which is comparable to land surrounding the CSM2 works that is gently undulating, sloping generally from west to east.

There are existing dwellings and structures associated with the farmland on both the north and south sides of the proposed alignment.

Between chainage 0 m and 3800 m, the proposed alignment will cross Robinsons Road, Waterholes Road and Trents Road. These crossing points will not become at grade intersections.

The distance between the CSM2 Project alignment and the railway increases as the chainage increases – from less than 200m at Robinsons Road to approximately 3 km at Halswell Junction Road.

On the north west corner of the intersection with Robinsons Road, there is an existing business where farming machinery and machinery parts are traded and stored. There is a risk of contaminated site runoff from this site and this is discussed further in Section 6.3.1.

The existing railway, Jones Road and SH1 alignment north and west of the proposed CSM2 intersection forms an embankment to overland flow. A 600 mm diameter culvert at the railway feeds to this area and is one of only a few areas where overland flow is conveyed from the west side of the railway to the east.

Some flooding within existing depressions in the topography has been observed on the west side of the railway, Jones Road and SH1. The NZTA report that there are no historical overflows over SH1. Given no SH1 stormwater infrastructure has been identified in this length, it has been assumed that overtopping will not occur and the catchments upstream of CSM2 are limited by SH1. Should overland flow overtop SH1, the Railway embankment would form a barrier to upstream overland flow and the increment in catchment area (between SH1 and the railway) would be small.

At the locations where historical surface ponding has occurred, the groundwater is several metres below surface level. The surface ponding (flooding) occurs where the rainfall rates exceeds the infiltration rate, ponding occurs and where gradient is available, then overland flow occurs. Once the effects of CPW are felt, the groundwater level will rise but will still be several metres below ground level upstream of the Project. As such we do not anticipate any increase the frequency or extent of overland flow from the effects of CPW.

As described in 3.3.1 above, the Western Rear Access Road route is proposed between Weedons and Robinsons and a further 200 m extension past Robinsons Road. The land is similar to the previous section and is largely flat yet includes mild undulations that reflect the original topography prior to construction of the rail and roads in the area. There are a number of low points along this route that have been inferred as old stream channels that once flowed over the plains.

3.4.1 B Stockwater Races

Various stockwater races will be encountered with the new alignment and these will be incorporated into the stormwater drainage design to ensure that their function and performance will not be adversely affected.

- An existing stockwater race flowing south runs along the south west side of Robinsons Road, crossing below SH1 at approximately chainage 350 m.
- An existing stockwater race flowing south runs along Waterholes Road. At Waterholes Road and SH1 intersection the stockwater race crosses SH1 to the north east side and continues down Waterholes Road where it also crosses CSM2 alignment at chainage 2000 m.
- An existing stockwater race flowing south runs along the north east side of Trents Road, crossing below CSM2 at approximately chainage 3500 m.
- An existing stockwater race flowing south runs along the south west side of Blakes Road, crossing below SH1 at approximately chainage 3800 m.

3.4.1 C Catchment

As described above, there are few culverts through the rail embankment above this section of CSM2. There is significant storage for overland flows that could arrive above the rail embankment. Any flows that come through the embankment are of small consequence to the CSM2 alignment.

The distance between the proposed CSM2 road and the existing SH1 alignment and in turn the existing railway increases as the chainage increases. This means the potential catchment areas upstream of CSM2 between the SIMT rail and the proposed CSM2 alignment get progressively larger.

3.4.1 D Stormwater Design Constraints

The existing Robinsons Road intersection is a cross road with SH1 and will be upgraded as part of the Project scope of works by means of a proposed overpass (Robinsons under CSM2 and under SH1). The required excavation depths for the Robinsons Road excavation are significant with the new Robinson Road carriageway at its deepest part approximately 6.5 m below its current location. The depth of the excavation forms a design constraint, with regards to stormwater disposal and compliance with the NRRP (1 m clearance between disposal depth at the highest inferred groundwater depth). Further details are discussed in Section 6.

Runoff from the site on the north west corner of the Robinsons Road intersection may be contaminated and therefore should not be allowed to reach any proposed stormwater treatment or conveyance areas within the proposed CSM2 drainage layout. As discussed above, the CSM2 alignment will form a barrier to overland flows. The road drainage system will need to connect beneath the Project alignment, to the natural channel downstream, so that overland flows are not impeded.

Existing stockwater races will require diversion or need to be piped below the CSM2 alignment.

The vertical alignment of Blakes Road may form a low point where it terminates on the north and south side of CSM2. Soak pits draining to land will be required to dispose of any ponding of runoff or overland flow at these locations.

3.4.1 E Groundwater

The following groundwater levels were measured at local road crossings of CSM2:

- Robinsons Road intersection – The groundwater levels were measured at between RL 31 – 32 m at the proposed location of the Robinsons Road structure (approximately 13 to 14 m below existing ground level) during 2010 and 2011
- Waterholes and Hamptons Road Intersection – The groundwater levels were measured at between RL 26 – 27 m (approximately 13 to 14 m below existing ground level) at the proposed location of the Waterholes Road structure during 2010 and 2011 and
- Trents and Blakes Road Intersection – The groundwater levels were measured at between RL 23 – 24 m during 2010 and 2011 (approximately 12 to 13 m below existing ground level) at the proposed location of the Trents Road underpass.

Table 3 summarises the measured groundwater levels, design adjustments and Scheme Assessment Report (SAR) design groundwater levels and depths. For example, the measured Robinsons Road depth of 13 to 14 m below ground has been increased by 9 m (6 m for the historical maxima and 3 m for CPW) to give a design groundwater depth of 5 m below ground (or a level of about RL 40 m.

Since the SAR phase further detailed groundwater assessment and modelling has been undertaken by the Project team and is included in **Appendix C**. The groundwater work now predicts groundwater high including for the effects of CPW to be at RL = 39.6m.

Table 3 Design Groundwater Levels between Robinsons Road and Trents Road

Location	Ground Level (m)	Measured Groundwater Level / Depth (m)	Historical Maxima Allowance (m)	CPW Allowance (m)	Design Groundwater Level (incl. historical and CPW) / depth (m)
Robinsons Road	45	31-32 / 13-14	+6	+3	40 / 5
Waterholes Road / Hamptons	40	26 / 14	+6	+2	34 / 6
Trents	36	24 / 12	+6	+2	31 / 5

Based upon the initial findings associated with the SAR and the uncertainties raised a further work package was commissioned. This work was completed by Beca (August 2012) and included as **Appendix C**. Table 4 is a summary of that report and shows the frequency and period of time upon which the groundwater level is likely to exceed a certain level. Thus once the effects of CPW are present then for 5% of time the expected groundwater level will exceed RL = 37.4 m. (i.e. 5% of time or 18 days on average). Without the effects of CPW, groundwater levels would be well below the road level.

Table 4 Revised Groundwater Levels at Robinsons Road

Frequency	GWL Exceedance as a %age of time	Design Groundwater Level (including Historical and CPW) / Depth (m)
Peak GWL		39.6
	5%	37.4
	7.5%	36.6
	10%	36.3

3.4.2 Blakes Road to before Springs Road (Chainage 3800 m – 7100 m)

Refer to drawings 62236-B-C407 to C412 in the Plan set contained in Volume 5.

3.4.2 A Description

The portion of CSM2 from Blakes Road to before Shands Road continues through farmland, gently undulating and sloping from south west to north east. The CSM2 alignment requires crossing Shands Road at chainage 5350 m, Marshs Road at chainage 5950 m, and the existing disused Little River railway line at chainage 6600 m. The Marshs Road stockwater race currently passes beneath the disused railway line.

Part of the land to the north of CSM2, west of Shands Road, is zoned as industrial. The Marshs Road stockwater race currently intercepts two potential overland flow paths originating from this land.

Further stockwater races will be encountered with the new alignment over this section which are:

- An existing stockwater race flowing south east runs along the north east side of Marshs Road, crossing CSM2 at approximately chainage 6000 m.
- An existing stockwater race flowing north east runs along the north east side of Springs Road, crossing CSM2 at approximately chainage 7250 m.

3.4.2 B Catchment

The catchment area is farmland between the CSM2 and existing SH1 alignments. Runoff from catchments upstream of motorway flows to the Project area. This occurs now and is independent of the Project. The Project proposes to capture these flows in a swale and divert them to a storage and/or soakage system, or divert them to a siphon for passage under the Project alignment.

3.4.2 C Stormwater Design Constraints

The proposed alignment requires crossing of Shands Road and Marshs Road. The existing cycle / walking trail on the disused Little River railway line will form an embankment and a potential overland flow collection point at the cross over with the CSM2 alignment.

Existing stockwater races will require diversion or need to be piped below the CSM2 alignment.

3.4.2 D Groundwater

At the Marshs Road and Shands Road Intersections the historic groundwater levels were measured. The ground water depth range during 2010 and 2011 was between RL 17 m and RL 20 m. The ground level is at RL = 26.9 and 28.2 m at the respective intersections with the CSM2 alignment. Historic groundwater is approximately 7 to 10 m below existing ground level at the proposed location of the Marshs Road Structure, and 8 to 11 m below existing ground level at the proposed location of the Shands Road. An allowance for historical maxima (approximately 3.5 m) and CPW (approximately 1.5 m) established the design groundwater level high of RL – 23 m. At a groundwater high level of 23 m, there is more than 3 m clearance to groundwater.

3.4.3 Before Springs Road to CSM1 (Chainage 7100 m – 8400 m)

Refer to drawings 62236-B-C412 to C414 in the Plan set contained in Volume 5.

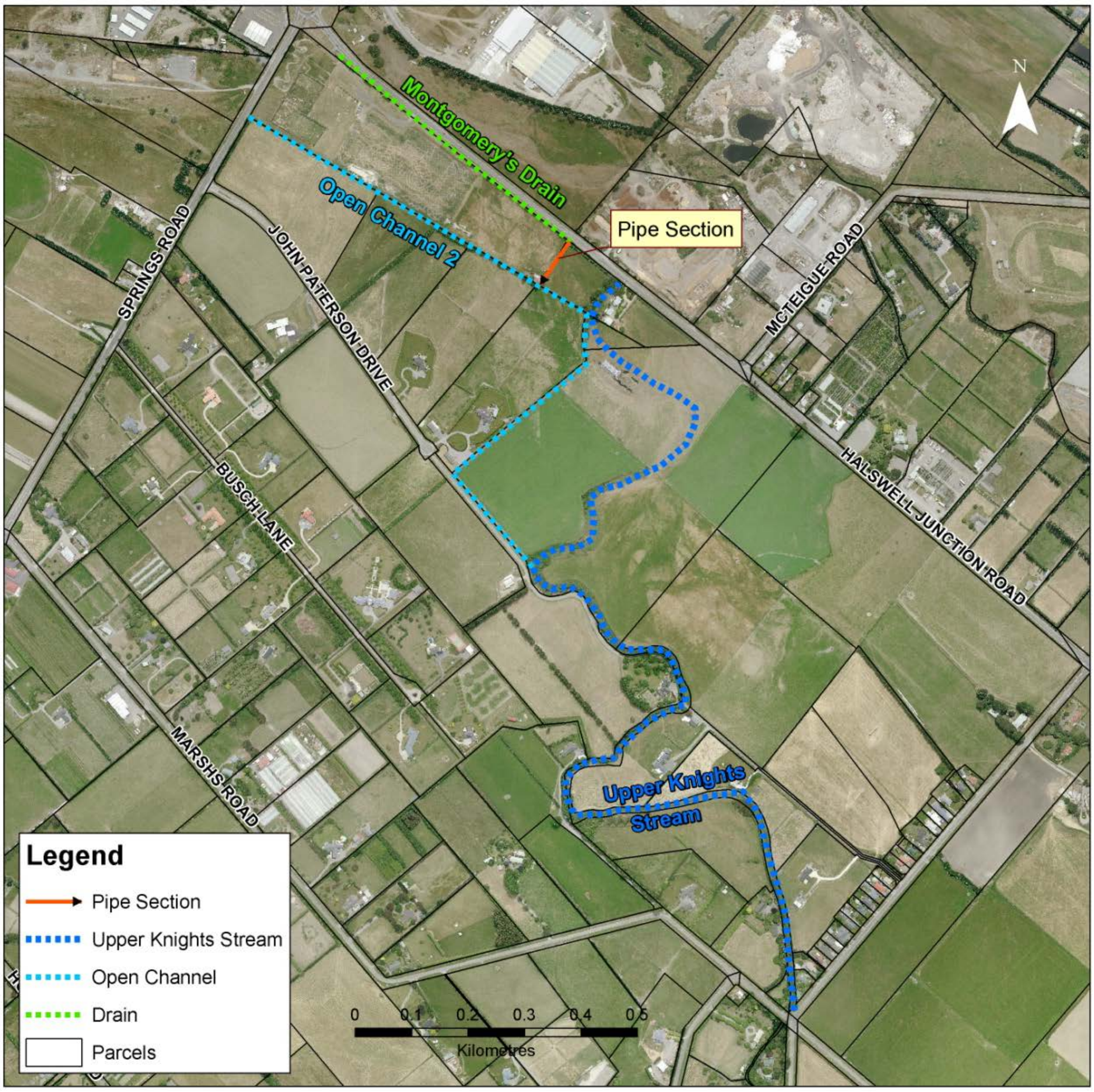
3.4.3 A Description

The proposed alignment from chainage 7100 m before Springs Road to chainage 8400 m at CSM1, passes to the south east of an existing industrial area. CSM2 crosses Springs Road at chainage 7200 m and Halswell Junction Road at chainage 7600 m. Springs Road and Halswell Junction Road intersect at a roundabout to the west of CSM2 which also services Wilmers Road. The roundabout is currently being reconstructed as part of the CSM1 works.

On the north west side of the CSM1 alignment and Halswell Junction Road, there are the Mushroom Ponds that were constructed as part of the CSM1 works.

Montgomery's Drain (occasionally referred to as Upper Knights Drain) runs parallel with Halswell Junction Road starting near the Halswell Junction Road Roundabout and heading south east for approximately 550 m before entering a piped system. The Drain is fed from a 675 mm diameter pipe originating from the Halswell Junction Road stormwater pond overflow. The Drains ends in a pipe. The 750 mm diameter pipe heads away from Halswell Junction Road to the south where it discharges to an open channel which continues to the south before heading south east near the end of John Paterson Drive. This open channel discharges to the Upper Knights Stream. The layout of these drains is shown in Figure 8.

Figure 8 Existing Stormwater Infrastructure Downstream of Montgomery's Drain



The SWAP includes a stormwater treatment pond on the east side of the proposed CSM1 alignment known as the Owaka Basin. Independent of the CSM1 Project, the CCC undertook (under contract) bulk excavation to form the Owaka Basin shape (as shown in an intermediate state in Figure 9 (Photo early 2012)). This bulk excavation work is now substantially complete. This treatment facility has been designed to capture overflows from the Halswell Junction Road Pond (via Montgomery's Drain) and provide additional stormwater treatment. The normal discharge from the Owaka Basin is by soakage with the primary overflow to the north (beneath CSM1) to the Wilmers Quarry site. Once the capacity is exceeded, the system will overflow south under Halswell Junction Road into Montgomery's Drain and to Upper Knights Stream.

Figure 9 Excavations at the Owaka Pond Site

There is an existing field drain on the west side of Springs Road that turns east approximately 250 m before the Halswell Junction Road roundabout.

3.4.3 B Catchment

The most northern section of CSM2 is part of the Halswell River Catchment. This area drains to the Halswell River via Montgomery's Drain and Upper Knights Stream. ECan has stated that the Halswell River is sensitive to any increases in peak discharge rate or volume, as there is a history of flooding and the community is actively engaged in its management.

ECan maps indicate two streams in the fields to the north of Springs Road between Halswell Junction Road and Marshs Road. Field inspections failed to confirm the location of these 'streams', however, it is likely that these 'streams' are ephemeral and would discharge to drainage ditches along Springs Road which both head towards and away from the City along the northern side of the road.

3.4.3 C Stormwater Design Constraints

A small portion of the proposed CSM2, up to Marshs Road (sections 1 and 2), is within the CCC SWAP surface water management scheme area. The CCC has indicated that it may be possible to connect a limited extent of the Project to the existing stormwater drainage network. This would include discharging flows to either the existing or proposed stormwater retention basins in the area. The potential for this is somewhat limited by the proposed vertical alignment of the CSM2 and limited fall between the edge of seal and the base of the proposed CCC pond system and, as such, has not been included in the design.

The Project will cross Montgomery's Drain which runs parallel to Halswell Junction Road. The drain eventually discharges into Upper Knight's Stream (via a pipe and open channel system) and eventually discharges into the Halswell River. Siphoning of this drain beneath the Project alignment will be required.

The existing Halswell Junction Road pond has been recently enlarged as part of the CSM1 project. This pond discharges to Montgomery's Drain downstream of the Springs/Halswell Junction Road roundabout.

Elevated groundwater levels in the vicinity form a constraint on the invert of the proposed disposal systems and are discussed below.

Any realignment of Montgomery's Drain will require adequate erosion and sediment control measures during construction to ensure protection of the downstream watercourses.

3.4.3 D Groundwater

At Springs Road, the historical groundwater depths were measured at 5 m to 7 m below ground during 2010 and 2011.

During the SAR phase, an allowance for historical maxima at Springs Road (approximately 2 m) and CPW (approximately 1 m) established the preliminary design groundwater level of approximately RL 20 m or 3 m below ground. This compares to what is shown in the ECan well records (M36 – 4018) of between 3 – 6 m deep near Wigram with a historical maximum of RL 17 m.

Beca investigated the frequency of extreme groundwater events and the likelihood on coincidence with extreme rainfall as part of the CSM1 work (Beca 2011). It concluded that a design groundwater level about Halswell Junction Road was RL 18.3 m for the coincidence of a 50 year ARI groundwater event to be used in conjunction with a three month rainfall event (with consideration be given to the three year event). This figure has been used as the design groundwater level around Halswell Junction Road. Note: these predictions were made before the outcome of the CPW project appeal hearings and as such do not account for the predicted future effects of the CPW project.

The most recent ground water investigation (Beca, August 2012) and attached as **Appendix C** sets out the latest predictions of future groundwater levels at Halswell Junction Road. The predictions include the effects of CPW which is now part of the existing planning environment. The groundwater assessment work has included a 1.3 m increase in groundwater levels at Halswell Junction Road from the effects of CPW.

Table 5 shows the frequency and period of time upon which the **Appendix C** groundwater level predictions are likely to exceed a certain level. Thus once the effects of CPW are present then for 5% of time the expected groundwater level will exceed RL = 18.8 m (i.e. 5% of time or 18 days on average per year).

Table 5 Revised Groundwater Levels at Halswell Junction Road at the Maize Maze Pond

Frequency	GWL Exceedance as a %age of time	Design Groundwater Level (including Historical and CPW) / Depth (m)
Peak GWL		19.4
	5%	18.8
	7.5%	18.4
	10%	18.2

The effects of the groundwater level rise on the design including the effects on the ponds from the future groundwater rise are set out in Chapter 6.

4 Design Philosophy / Design Treatment

This chapter summarises the stormwater design philosophy adopted for the CSM2 and MSRFL Projects. The key elements include:

- Separation of the Project drainage system from the surrounding surface water and stormwater systems, and from stockwater races
- Stopping overland flows from entering the proposed Project drainage system and flooding high speed carriageway
- Designing the components of the stormwater and drainage system for the 100 year ARI event
- Designing to allow for an increase in rainfall intensity of 16% due to climate change as per the Waterways, Wetlands and Drainage Guide ((WWDG (CCC, 2003))
- Designing with the knowledge that the vertical alignment has only two sag or low points with considerable contributing area. These low points are located at:
  - → Weedons Ross Road
  - → Halswell Junction Road
- Treatment of stormwater will be achieved primarily by sheet flow over the grassed verge and flow in treatment swales. In addition first flush basins have been included where required in the NRRP.
- Typical details for the proposed stormwater infrastructure can be found in **Appendix B**.

4.1 Design Rainfall

Rainfall figures from the WWDG (CCC, 2003) including the updated table addressing climate change have been used in the design SAR report phase of the CSM2 Project. Climate change effects were not part of the 2003 version although the effects of climate change were allowed for in the SAR phase of the project.

CCC has released an update to WWDG chapter 21 Rainfall & Runoff (Nov 2011) and this edition includes for the effects of climate change. The latest CCC update rainfall tables were very similar to the 2003 WWDG tabulated data when the effects of climate change as recommended by MfE were included.

SDC recently commissioned work on local rainfall, the outcomes have been used in the design for MSRFL (Development of Design Rainfall for Selwyn District, (Opus, 2009)). A full discussion of the methods and rainfall values adopted for the design are set out in the Stormwater Management and

Disposal Options Report (GHD/Beca, 2011), including a comparison with the NIWA High Intensity Rainfall Data System (HIRDS) v3 data.

4.1.1 Climate Change

Both the WWDG (Nov 2011 update) and SDC documents accommodate for the MFE (2008) predicted effects of climate change.

For the CSM2 SAR, the predicted mid-range effects of climate change were added to the 100 year ARI rainfall event to ensure that the assessment of effects would be appropriate for the foreseeable life of the asset being constructed. The effects of climate change have been adopted from Preparing for Climate Change, A Guide for Local Government, (MfE; July 2008) using the mid range temperature rise of 2.1℃ temperature rise to 2090. These climate change effects are now incorporated in the revised WWDG and SDC guides and should be used for the balance of the project design.

For the determination of flows within and across the Project, the 24 hour rainfall depth has been used to determine average flow rates. In order to determine peak flow rates the WWDG has specific guidance which has been adopted for this Project. In summary, the revised climate change adjusted 100 year, 24 hour rainfall depth of 158.4 mm has been applied to all of CSM2. The Rolleston adjusted Christchurch Aero data (as per the SDC design standard) has been used for MSRFL. A figure of 140.7 mm was used for the same event as above.

4.2 Design Runoff Rate

For the SAR phase of the project the design rainfall figures set out above have been used in conjunction with the United States Soil Conservation Service (SCS) method to estimate the peak discharge rate of runoff from the Project and from the adjacent rural catchment. Curve numbers (CN) of 74 and 98 have been applied to pervious and impervious areas respectively. The CN of 74 is considered conservative at this phase of the design and consenting process. Mathematically converting this CN to a volumetric runoff coefficient using the 100 year ARI 24 hour design storm gives a value of 0.58. For comparison the WWDG recommends a volumetric runoff coefficient of 0.35 for rural areas (CCC 2003).

In adopting the above design runoff characteristics there is a potential that the peak runoff rate and consequential stormwater element sizes will be larger than required for the design rain event. In order to establish an improved estimate of overland flow rates, more site specific (infiltration) field testing can be used to calibrate the runoff model and potentially reduce the stormwater infrastructure element size.

These hydraulic elements set out in the SAR have been replicated within this report.

The Project also requires volumetric storage and these are used to determine the storage component elements of the design. There are two key areas where storage volumes are critical and influence the design, namely:

- The peak volume in the Project ponds and other CCC ponds adjacent to Halswell Junction Road.
- The volumetric capacity of the swales to determine the peak stored volume. The equation 'peak stored volume' is equal to the runoff volume less the volume out (volume to soakage) for critical time steps through the storm event.

In order to establish the effects on the Halswell River catchment where the critical storm has a time of up to 60 hours in length, we have adopted a total storm runoff approach. This has been to capture 100% of the 24 hour storm and to store this volume for release once there is surplus capacity in the Halswell River following the storm event. In addition to the 100% capture of the 24 hour storm, further modelling runs have been undertaken to determine the effects of longer period storm events (48 hour and 60 hour). The results of this modelling work are presented in Appendix D, Stormwater Modelling Report.

For the SAR phase of the project the calculation method adopted the unit hydrograph method using the SCS method to determine the volumetric runoff. The soakage to ground rates were assumed to ensure that the swales would not overtop in the critical event. Following swale sizing, the indicative design for the soakage was carried out to meet the assumed values. Based upon this, typical swale lengths of up to 300 m in length were deemed suitable for the CSM2 section and up to 200 m long for the MSRFL section where there was reduced designation width available. The same method was used for the indicative sizing of the cross drainage elements including culverts and siphons.

In order to confirm the stormwater design elements could fit within the designation and have no off-site discharge to land for up to the 100 year ARI event, further work has been done as part of this current report. The CCC WWDG method of using Hortons and multiple nested storm profiles was used to confirm the swale volumetric capacity. Based upon this work, swale lengths of up to 300m have been confirmed for the CSM2 section, and maximum swale lengths of 200 m are confirmed for the MSRFL section together with intermediate bunds.

For water quality treatment and other volume related hydraulic elements the WWDG triangular hyetograph has been used for the volumetric calculations used in the sizing of water quality treatment device elements.

In order to establish a design runoff rate suitable for final design, a larger scale peer reviewed field infiltration testing is recommended to establish a less conservative runoff coefficient that is also cognisant of the risks together with the predictions involved.

4.3 Water Quantity

The various components of the stormwater and drainage system have been designed for the 100 year ARI event. This includes the conveyance capacity of swales and pipes and the required storage within the disposal system. This standard is required as the vast majority of the stormwater collection and treatment system will be constructed below the existing ground level, limiting the ability to 'spill' out of the system in large events.

The 100 year ARI standard required by the NZTA exceeds the requirements of the WWDG (CCC, 2003), the SDC Code of Practice and the NZ Building Code, all of which specify the 50 year ARI event, but it is consistent with the NZTA stormwater treatment standard. Events in excess of this ARI event have the potential to cause flooding upstream of the Project and of the Project itself. These over-design events (over and above the 100 year ARI event) are discussed further in Section 6.

The amount of storage required in the system is a function of runoff (i.e. inflow) and the disposal rate (i.e. outflow), as defined in the hydrological equation (total inflow – total outflow = storage). The maximum amount of storage is typically set by the geometry of the swale or the pond. In order to contain the discharge disposal within the Project footprint for the 100 year ARI event, the design has a number of key elements namely:

- Swales: Runoff will occur from pavement to swale, flow occurs along the swale with loss to ground occurs during this process. At the lower end of the swale, storage will occur behind the bund constructed to induce storage. A mechanism has been designed to collected treated flow and discharge this to ground through an infiltration trench constructed downstream of this bund.
- For swales in the MSRFL section a higher soakage to ground rate is required and this can be achieved at detailed design. For the CSM2 section a lower design discharge rate is required. In event of overtopping of intermediate bunds, flow will end up at the Shands Road interchange where large storage areas are available within the interchange area.
- The storage available within the ponds within the collective Halswell Junction Road ponds has been addressed in the Stormwater Modelling report attached as Appendix D.

Given that the Project runoff is being disposed exclusively to land (with the exception at Halswell Junction Road), the effects from the following matters are not considered as significant, as subsequently outlined:

- Intermediate design storm events, e.g. 2, 5, 10 and 50 year ARI events
- Downstream effects
- Receiving waterway sedimentation/erosion
- Attenuation of flows / hydraulic neutrality.

- Downstream erosion

These considerations are only relevant to surface water discharges where erosion of the stream banks is expected in lesser magnitude events (2 – 5 year average recurrence intervals (ARI)) or where flood control and downstream capacities are of concern (across a range of ARI and duration events). In this Project any surface water discharges will be limited to exceedance rainfall events (greater than 100 year ARI) or extreme groundwater events. For smaller events there will be no surface water discharge as the runoff is totally contained within the project. As such no consideration has been given to erosion control or flood control in intermediate events. For larger extreme events, there will be already base flow in the receiving stream. Any project discharge rate is likely to be very small in relation to overall receiving environment flow and as the overall gradient is relatively flat (i.e approximately 1:250 which is the average background gradient of the Canterbury Plains) then erosion potential at these flat gradients is minimal.

Consideration has been given to erosion and sediment control during the construction of the Project as discussed in Section 9.3.1.

4.3.1 Halswell Junction Road Water Quality and Pond Capacity

The proposed water quality treatment design is significantly influenced by the ground water prediction advice received for this portion of the project. Recent groundwater advice (Beca, August 2012) predicts groundwater highs as being elevated above the previous predictions. There is also a frequency analysis that gives a period for which elevated levels can be expected. This has resulted in a specific approach to determining water quality treatment at the Ramp Ponds and the Maize Maze Ponds.

Initial predictions had determined an adequate level of redundancy above the ground water highs to the pond base. The groundwater assessment predictions have the future groundwater highs potentially causing significant inundation of the ponds that has the highest predicted level at 0.75 m above pond base level. Without intervention the groundwater would flow into the pond and reduce available storage in the pond prior to the storm event.

As a result the strategy and design philosophy for the pond operation is as follows:

- As a result of the predictions, an intervention strategy is proposed to intercept rising groundwater by means of a sub drainage system is proposed that will lower the effects of the predicted future ground water table rise. At the beginning of any major storm event, the ground water table would be artificially lowered to pond base level (or lower). This system would then discharge by gravity to Upper Knights Stream, downstream of John Paterson Drive. During the period of the storm event there is a potential for groundwater level to rise above pond base level. The groundwater assessment confirms that the groundwater level rise will be less than the pond level rise. As such the groundwater will not negatively affect the potential storage calculated for storm storage. The groundwater drainage system will ensure an accelerated draw down following any significant rain event.

- This involves an under drainage system:
  - → Commencing at Halswell Junction Road / Springs Road Roundabout and continuing under CSM2 and Maize Maze Ponds
  - → Draining below the Ramp ponds
  - → As the CCC Owaka Basin capacity would also be compromised by the predicted ground water highs, a further under drainage system is expected to be constructed under the CCC facility (but this is outside of the scope of this Project).

On-going consultation with CCC catchment/network managers of the Halswell catchment is proposed, to ensure there is an integrated approach to future groundwater highs taken at all stormwater facilities in the Halswell Junction Road area.

The Stormwater Water Modelling Report is attached as **Appendix D.** This work describes the performance of the all the ponds (Maize Maze, Ramp, Owaka, HJR Pond, Mushroom Pond and Wilmers Quarry Basin) through a range of storm events.

4.3.2 Risk and Larger Storm Events

The Project drainage systems and the receiving and passage of cross drainage have been designed for the 100 year ARI rainfall event. In the event of an over-design rainfall event across the entire alignment, the storage in the system will be filled. Stormwater will fill the intermediate storage and overflow to the next storage basin or swale downstream. This overflow regime has potential for large stormwater volumes to accumulate at the lower or sag points in the Project, namely the Maize Maze Pond. Water will flow to the low points in the system, most notably the sag points adjacent to Halswell Junction Road and Weedons Ross Road. There may be potential to spill out of the system to existing overland flow path downstream of the alignment. This is discussed in more detail in Section 8.1.1.

The existing MSRFL and the existing (greenfield) alignment has the potential for overland flow from upstream to pass over the alignment once the capacity of the existing system capacity is exceeded. The existing systems include overland flow paths and a series of culverts. Blockage or partial restriction would exacerbate the problem. The design assesses the risks with each existing or new crossing and makes an allowance for restricted capacity.

4.4 Overland Flows

Site design will aim to reduce the effect of the Project on overland flow and runoff conditions. The natural and existing drainage network will be utilised as much as possible and only diverted or re-formed should it be absolutely necessary. Potential overland flow paths and stockwater races have been identified and shown on the Drainage series drawings in Volume 5 of the application documents (the Plan Set). The contributing catchments to these overland flows have yet to be confirmed because of a lack of available detailed survey and/or LiDAR generated contour data on the private land uphill of the Project. This additional survey will be carried out at the detailed design phase. Irrespective of this, the combined effect will not alter.

The effect of the Project on flooding upstream and downstream will be minor in events less than or equal to the 100 year ARI event through the mitigation incorporated into the design of the Project (i.e. the crossing capacity conduits and of the design at the inlet and outlet structures of the system).

The area around the inlets to the overland flow siphons will be lowered to construct a settlement area (to reduce the volume of silt entering the system) and ponding area, and to limit the elevation of the inlet. This will reduce the upstream ponding depth and any increases in flood level (degree of afflux) on the adjacent property will be limited to 250 mm. A further requirement is recommended that there shall be no increase in flood levels where the effects of the project that might impinge on habitable floor levels.

Events exceeding the design storm event could potentially lead to increased flooding upstream or overtop into the Project drainage system. Should the latter be the case, redundancy in the Project drainage design will limit the potential for flooding the carriageway. The potential effect upstream of the Project can be managed by restricting the increase in upstream flood level by the following rationale, which needs to be included into a consent condition. A maximum increase in upstream flood level (or afflux) of 250 mm is recommended for rural land provided no habitable floor levels are affected. A zero increase in upstream flood level (or zero afflux) is recommended at existing dwellings for the 50 year ARI design storm. (The use of the 50 year event relates to the Building Act where a 50 year ARI event is specified).

In addition to the overland flow siphons, cross drains will be provided within the Project drainage system, between the lateral swales. It is proposed to link swales on both sides of the main alignment using these cross drains. Entry sumps would have a lip level at just below the height of the top of the bund in the swale. This allows additional redundancy should the upstream swale receive flows from outside of the project area during extreme rain events. They will also provide the opportunity to pass flows under the carriageway that would otherwise build up and potentially flood the carriageway.

The transfer of water beneath the motorway via the cross drains will also facilitate pumping down of the system after extreme events using temporary pumps (potentially discharging to downstream overland flow paths).

The design to date has identified nine stockwater races that cross the Project alignment. In addition another eight overland flow path locations were identified and reported in the Stormwater Options and Disposal Options report (GHD, 2011). A further two major overland flow paths have also been identified in Table 6 below.

Table 6 Design Surface Water Siphon Crossings

Project	Siphon Location	Chainage (m)	Contributing area (ha)	Comments
CSM2	SH1 interchange & Ramps	320 SH1 / 700 Ramp	3	Including on and off ramps
CSM2	Waterholes Road west 2	1600	8	Existing drainage ditch
CSM2	Waterholes Road west 1	1750	8	Overland flow
CSM2	Waterholes Road east	2450	20	Overland flow
CSM2	Blakes Road	3780	5	Overland flow
CSM2	Trents Road west 2	2850	39	Overland flow – Conservative estimate as a lot of contributing subdivision will be to soakage
CSM2	Marshs Road west	5300	31	Overland flow – assume contributing to existing race
CSM2	Railway Corridor	6450	31	Overland flow
CSM2	Springs Road west	7050	8	Overland flow
CSM2	Halswell Junction Road	7700	3	Montgomery's Drain (including allowance for discharges from existing Halswell Junction Road Basin)

Before or during the detailed design process, further topographical survey and /or LiDAR survey will be required to ascertain catchments and overland flow paths with a higher degree of accuracy. An analysis of this information may identify other overland flow paths, and will assist in confirming the assumptions on which the areas in Table 2 are based.

The base case for the analysis of effects is to allow for each overland flow to be picked up on the upstream side of the Project and passed beneath in a siphon and discharged into the existing overland path on the downstream side of the Project.

The existing survey and existing LiDAR coverage is insufficient to identify the contributing catchment to each of the proposed siphons and upstream diversion bunds. Further LiDAR is being organised and is expected to be available before Feb 2013. Thus once this information is available, then further analysis of catchments and identification of potential flow paths upstream of the motorway can be undertaken. This work can be undertaken during detailed design or before.

However, it is expected that during the detailed design process and/or the construction phase that there may be opportunity to rationalise the number of newly identified and/or currently proposed crossing points and it is proposed that any modifications to the design adhere to the following criteria:

1. An investigation into the upstream effects is made in conjunction with the design of siphons under the Project alignment

2. A design process is undertaken to avoid any increase in upstream habitable floor level flooding in events up to the 50 year ARI event (i.e. zero afflux)

3. A design process is undertaken to avoid any increase more than 250 mm in flooding depth for events up to the 100 year ARI event (i.e. max afflux level of 250 mm) where habitable floors are not affected

4. An investigation of the downstream effects is made as a consequence of concentrating flow to a point discharge

5. A design process is undertaken to avoid any increase in downstream habitable floor level flooding in events up to the 50 year ARI event.

The total catchment area contributing to the downstream flow paths will be reduced by the footprint of the Project due to the road drainage system, because runoff from the Project will be disposed of to the ground rather than to the surface.

Following a large rain event the siphons will fill and convey overland flow. However once the event has passed they will be notionally full. It is prosed to incorporate a low flow drainage to allow the siphons to drain and hence for most of the time they will be notionally dry. Designing so that the siphons dry out after an event mitigates the risk of standing water causing odour issues or the promotion of nuisance insects. Some of these discharges occur in areas where groundwater is less than 6 metres below ground level (as indicated on the planning maps) and an organic filtration media cannot be maintained. The infrequent activation of the siphons will ensure that any effects on groundwater are less than minor.

At Shands and Weedons intersections, there is sufficient space to allow for overland flow to be channelled and captured in soakage basins within the designation area and within the Shands Road on and off ramps. Thus the primary method of dealing with overland flow will be to soakage basins. Siphons will be required to convey this flow under the on and off ramps.

There is a potential for these basins to fill in extreme rainfall and as such overflow weirs and passage from the weirs downstream will be required. These will consist of further siphons but with reduced capacity. It is intended to discharge to the road reserve in Shands Road and to the Hamptons Road stockwater race at the Waterholes underpass.

In parts of the Project, the swale to intercept the overland flow on the upstream side has little or no vertical gradient (i.e. at cloverleaf interchanges), and ponding will occur and may induce some flooding on the upstream property, due to the flat gradients of the existing ground. The level of flooding will be limited by use of multiple siphons under the off ramp as required.

In order to mitigate for these effects we recommend that the increases in flood level upstream of the Project alignment be limited to 250 mm (i.e. maximum afflux level of 250 mm). In the most critical location, Shands Road, additional pipes are proposed to alleviate any significant increases in flood level and pass flows around or under the interchange to the downstream surface flow path.

4.5 Stockwater Races

Nine stockwater races cross the current Project alignment. All of these races are piped under the existing SH1 and local road network. Some of the races are in pipes at grade, with the balance depressed under the carriageway in pipes but using the (inverted) siphon principle.

A series of (inverted) siphons will be used to convey stockwater races from one side of the MSRFL and CSM2 alignments to the other. Given that the Project drainage system will be below ground the only alternative would be to pump flows past the alignment. In order to prevent sedimentation of the siphon, a small diameter pipe with higher velocities is preferred. SDC provided siphon diameters for the crossings and these have been used for preliminary design purposes, however, the proposed pipe diameters will not have sufficient capacity to pass flood flows. A second parallel pipe has been proposed to maintain the land drainage function of the races and to prevent flooding immediately upstream of the crossing points. The upstream catchment draining to each stockwater race and the race capacity will be determined during detailed design along with confirmation of the need for a secondary culvert at every crossing location.

A shallow earth 'spillway' is proposed near the crest of the existing water race to allow the activation of the second, normally dry pipe. After a significant rainfall event has passed the secondary siphon pipe will then drain to a short soakage trench and drain away leaving a dry pipe.

Sizing of the secondary pipe has been based upon catchments delineated upstream of the proposed alignment and peak discharge calculations using the SCS method. The secondary pipe could also allow the diversion of normal flows to permit maintenance of the primary pipe. Both the primary and secondary pipes will require some screening to prevent blockage and would be laid at a grade to facilitate self-cleaning.

Closure of stockwater races is proposed in a limited number of locations. These are set out in the Detailed Project Description (Section 6). As noted in Section 3.2.4 this will require unanimous agreement from paying race users affected by the closures and approval from SDC.

Given the likelihood of penetrating the porous subsoil layers at the new race re-locations, the races are expected to be installed with a new liner system to prevent water loss. Protection of the base of the liner may be required using some larger gravels as SDC have indicated that cleaning of the races is done by heavy machinery which could easily penetrate the liner. Historically liners have not been installed in the races as the race walls and base will bind with silt over time. Given the lengths of proposed diversions, a more formalised liner will be required on the proposed stockwater race deviations.

During consultation with SDC, the Council has advised that stockwater races can be closed for up to 24 hours without notice and for longer periods with the prescribed notice.

4.6 Water Quality

The Project will have an effect on the daily traffic volumes along the existing road network. Traffic is predicted to shift off the portion of SH1 between HJR and CSM2 and onto CSM2 and increase along MSRFL as outlined below in Table 7. The Baseline assumes CSM2 is not built.

The ADT volume is an indicator of water quality, simply being: more traffic creates more contaminants. A full discussion on the effect of the Project on traffic can be found in Technical Report No. 2, Assessment of Traffic and Transportation Effects (GHD/Beca, 2012).

Table 7 Average Daily Traffic (ADT) Volumes – 2041 Prediction

Road and Location	Baseline (2016)	Project (2041)
Main South Road: South of Halswell Junction Road	30,250	23,250
Main South Road: South of Robinsons Rd/ Curraghs Rd	25,000	44,300
CSM2: East of Main South Road	n/a	27,000

The change in ADT volume as a result of the Project will alter the quality of the stormwater runoff being disposed to land. ADT volumes will reduce on the existing, untreated length of SH1 north of the CSM2 connection point. Typically, stormwater discharges to land along CSM2 will be via a treatment system which meets the requirements of the NRRP.

Treatment objectives will be met with a treatment train approach incorporating sheet flow across grass, water quality swales, first flush basins (where required) and controlled percolation rates (where required). The NRRP allows untreated road runoff to be disposed to land for much of the proposed alignment; however, almost the entirety of the Project will receive some treatment in the swale system prior to discharge to land (excluding some very limited kerb and channelled sections and the base of the Robinsons Road Overpass). The main contaminants in run off from a State highway environment are:

- Hydrocarbons – from vehicle emissions, oil leaks and vehicle accidents
- Metals – from brake, tyre wear and spillage
- Sediments – from vehicles and surface wear and
- Litter – general rubbish.

Contaminant generation modelling was not considered necessary for this Project as compliance with the NRRP prescriptive treatment standard (where treatment is required) provides evidence of mitigation of effects. The distance between the level of disposal and the typical groundwater level provides treatment of any residual contaminants which may not be captured in the treatment system.

4.6.1 First Flush Treatment

The principle of first flush capture and treatment is that many of the contaminants accumulate on surfaces such as roads and roofs during dry periods. These contaminants are removed by small storms or during the first part of longer duration, larger storms. In minor events it is accepted that the infiltration rate through the base of the swale may exceed the runoff generated on the surface. Contaminants in these small events will be collected in the topsoil of the swale. In events which exceed the swale percolation rate contaminants could pass down the swale to the storage and/or disposal points. Storage of the first flush allows the contaminants to settle out of the stormwater and to collect in the base of the stormwater system.

There are a number of different methods for calculating the volume required for the treatment of the first flush, otherwise known as the water quality volume. These are discussed in the NZTA Stormwater Treatment Standard for State Highway Infrastructure (NZTA, 2010) and the Stormwater Management and Disposal Options Report (GHD / Beca, 2011).

A conservative first flush treatment depth of 25 mm has been chosen to ensure compliance with local design guidance (WWDG (CCC, 2008)). First flush treatment has been considered in areas where stormwater treatment is required by the NRRP. The NRRP prescribes areas where treatment is required prior to disposal from a road to land (as the sole source). The NRRP maps the south eastern extent being roughly the Christchurch side of the Marshs Road/Shands Road interchange.

4.6.2 Design Criteria for Swales

The following table outlines the design criteria used for swales design to improve water quality. The proposed swale longitudinal slope is less than specified in the NZTA Stormwater Treatment

Standard for State Highway Infrastructure (NZTA, May 2010) due to the flat nature of the topography.

Table 8 Design Criteria for Swales

Parameter	Criteria	Comment / Source
Longitudinal slope	Typically 0.5% to 1% Minimum 0.3%	Flatter than standard, but acceptable given permeable subsoil and considered to be Best Practicable Option (BPO) to minimise road corridor
Maximum velocity	0.8 m/s	NZTA Standard[5]
Design vegetation height	100 – 150 mm	NZTA Standard
Typical water depth above vegetation	Should not exceed design vegetation height under the treatment design storm	NZTA Standard
Bottom width	0.6 to 2 m	NZTA Standard
Hydraulic residence time	9 minutes (minimum)	NZTA Standard
Maximum catchment area served	4 ha	NZTA Standard
Minimum length	30 m	Typical spacing is 300 m
Side slope	1 V : 4 H on road side. 1 V : 4 H target on back of MSRFL swales, however localised steeper sections at transitions to culvert entrances and at pinch points	Steepened rear faces to MSRFL swales to minimise road width and impacts of land purchase on adjacent property owners

4.6.3 Organic Filter Layers

The NRRP specifies compliant treatment/disposal area, filter media characteristics (i.e. organic soil layer depths and percolation rates) in areas mapped with less than 6 m to groundwater zone. The specified permissible disposal rates range between 20 mm/hr and 50 mm/hr for systems where

[5] The NZTA Stormwater Treatment Standard for State Highway Infrastructure, May 2010

infiltration is the design treatment. The proposed treatment solution in the less than 6 m to groundwater zone area for the Project includes swale treatment and first flush capture.

The first flush flows will be disposed through an organic filter media with a specification for the soil properties (material size and organic content) rather than percolation rate. The same specification for laying the filter material has been approved by ECan for the CSM1 Project in accordance with the *Stormwater Biofiltration Systems, Adoption Guidelines: Planning, Design and Practical Implementation*, Version 1, (Facility for Advanced Water Biofiltration, Monash University, June 2009).

The benefit of the specification is to avoid the final percolation rate as the defining compliance measure, thus allowing a greater variation in construction tolerances while still maintaining the environmental objectives.

4.7 Design Criteria for Disposal Methods

There are three types of disposal methods proposed in the Project:

- Soak pits
- Treatment areas
- Dry ponds.

The application of these solutions is dependent on the ECan planning maps and depth to design groundwater level.

4.7.1 Soak Pits

Soak pits are proposed at the ends of swales where the mapped depth to groundwater level is greater than 6 m indicating in the NRRP that treatment of stormwater prior to discharge to land is not required. This applies for the majority of the proposed route, from Rolleston to the city end of the Shands Road interchange. A map showing the ECan groundwater zoning is provided in Figure 13 of **Appendix A**.

In these areas the swale will provide primary treatment. Once the soakage ability of the swale invert is exceeded then flow will occur in the swale and flow to 1050 mm diameter manholes with scruffy dome inlets of invert 300 mm above the invert of the swale. The area immediately surrounding the scruffy dome will be constructed of coarse free draining material (with a null or low organic content). Settlement of sediment will take place in this area, with overflow dropping into the scruffy dome manhole. The perforated manhole base will provide further capture of gross pollutants prior to disposal to land. An outlet pipe from the scruffy dome manhole will convey flow to a soakage field which extends beneath the beginning of the downstream swale (and includes a flushing pit for ease of maintenance) and this pipe will be perforated to ensure spread disposal of runoff to land.

The swales (upstream of the soak pits) have generally been designed to the methodology outlined in the NZTA Stormwater Treatment Standard for State Highway Infrastructure (NZTA, May 2010). In some locations due to the lack of longitudinal gradient it is not possible to conform to the requirements set out in the NZTA Standards.

Throughout the Project kerbing has been kept to a minimum and is mainly located at underpasses/interchanges such as at the CSM1 connection. Channels will capture the runoff at interchanges and feed flow to specific stormwater treatment devices. Following treatment discharge will be to soak pits (except adjacent to Halswell Junction Road).

As per WWDG (CCC 2003), Christchurch's free-draining alluvial soils, to the south and west, provide the opportunity for surface water management by soakage. As required in the WWDG, the soakage system proposed will incorporate treatment prior to discharge that will reduce the flow of harmful contaminants that may enter the groundwater. The WWDG requires that the design must ensure that the risk of contaminating the underlying groundwater is minimised.

4.7.2 Treatment Areas

Where the NRRP indicates an area less than 6 m depth to groundwater water quality treatment is required prior to discharge to land as described above in Section 4.6.3. In these areas first flush basins are proposed with organic filter layers prior to disposal of the first flush. These basins will provide water quality treatment to the first flush with larger flows spilling to a higher level gross pollutant trap and soak pit arrangement.

4.7.3 Dry Ponds

A series of dry ponds are proposed along the Project alignment as follows:

- At Weedons Road, two ponds are proposed on the inside of the cloverleaf interchange. These ponds are to treat runoff prior to discharge to ground.
- At Robinsons Road for the discharge of pumped surface and groundwater
- At Shands Road underpass for the treatment of stormwater runoff prior to discharge to ground, and
- Adjacent to Halswell Junction Road for the treatment of runoff into the Ramp and Maize Maze Ponds.

The groundwater study has predicted high groundwater levels following the effects of the consented CPW project. This has an effect on pond discharges at Robinsons and at the Halswell Junction Road ponds.

The Halswell ponds are within the less than 6 m to groundwater zone. As such the ponds have been designed to comply with the NRRP requirements for discharge in this zone. These ponds will include a first flush and organic filter media prior to discharge to ground. In addition the storage area will be divided by bunds, to ensure that any spill from the pond is water which has been retained for the greatest duration. This is discussed in greater detail in Section 8.2.

The preliminary SAR design percolation rate applied for the dry Halswell Junction Road ponds is 12 mm/hr. This rate was derived through field testing and applying an increased factor of safety. Subsequent groundwater investigations have predicted high groundwater following the effects of CPW. During these future events the discharge to ground is likely to reduce to very low rates later in the storm event. As such a philosophy of under drainage and groundwater lowering and having a discretionary ability to draw down the pond levels (following the storm event) has been adopted. Other tests closer to the Marshs/Shands Road interchange have a higher tested percolation rate.

A dry pond is proposed near Berketts Road to collect and dispose of runoff generated on acceleration and deceleration lanes about Berketts Road. These lanes do not allow the inclusion of swales within the proposed designation width. Kerb and channelling will collect stormwater to a piped system. This flow will discharge to the dry pond proposed on the railway side of the Project. This pond will be designed to provide a level of water quality treatment similar to the swale system prior to disposal.

4.8 Underpasses

Note: Underpass by definition is defined in relation to the main alignment. For example where the motorway passes under a minor road, the bridge over the motorway is defined as an Underpass.

There are 7 new underpasses required to allow local roads to pass over the Project alignment. These are located at:

- Weedons Ross Road
- SH1 south bound on-ramp
- Waterholes Road
- Trents Road
- Shands Road
- Springs Road and
- Halswell Junction Road.

Typically run-off from the underpasses will be captured by kerb and channel, sumps and pipe work. The water will be discharged to swales at the base of the embankment or directly to disposal areas at existing ground level. In isolated areas where there is insufficient space to fit a swale at the base of the embankment and a disposal trench/toe drain will be required to dispose of any increased runoff. As runoff from the road will be collected by kerb and channel, the runoff will be solely from the grassed embankment. The areas where restricted width occurs are at Marshs, Trents and Waterholes Road embankments.

4.9 Overpass

There is one new overpass required to allow local roads to pass under the Project alignment. This is located at Robinsons Road. Robinsons Road will be excavated to approximately 6.5 m below existing ground level and will be potentially submerged by groundwater should the effects of Central Plains Water (CPW) be as significant as predicted (Weir, 2009).

The run-off from the realigned Robinson Road will fall to a new low point at the base of the under vertical curve. Flow will be captured by kerb and channel, sumps and pipe work that will flow to a horizontal storage pipe. Gross pollutant traps are to be incorporated to include pre-treatment of stormwater runoff.

During periods of low groundwater (prior to implementation of CPW), there will be sufficient hydraulic head to allow gravity flow from the storage pipe to a dedicated in ground infiltration gallery.

CPW is now part of the planning environment and as such the effects must be accounted for. Thus once sufficient CPW infrastructure has been constructed and CPW irrigation initiated there is a predicted rise in groundwater levels. Post CPW construction, and over time, and following periods of rain events there are predicted to be periods of elevated groundwater, where there will be insufficient head to dispose of water by gravity and a pumped solution is required.

Preliminary design of the pump has been carried out to pump surface water. Pumping rates would not match peak surface inflow rates so the inflows will be buffered in the surface water runoff storage tank.

In addition to pumping of surface water there is a potential need to pump groundwater. The ground water pumping system has been designed to dispose of groundwater where this has risen to within 0.6 m of the low point in the carriageway to protect the subgrade. The pump would be capable of pumping groundwater for potentially some weeks or months at a time during the period when the groundwater table is elevated. Thus once the effects of CPW elevated groundwater are felt, the pump would be installed. The pump operation is not expected to be continuous for significant periods throughout the year.

Should the pumping system fail the road is likely to flood. A potential consequence of this would be to close Robinsons Road where it passes under the CSM2 alignment. The effect of this would be to divert traffic to other crossing motorway points. The traffic report (Technical Report 2) reports existing traffic movements as being up to 600 vehicle movements per day.

It is proposed the pump(s) would be installed and maintained by the NZTA or their nominated subcontractor or out sourced to SDC. This installation would only be undertaken when it became evident that predicted groundwater rises were actually occurring, likely to be some years into the future. However, the wet well for any future pump would be incorporated into the drainage system during motorway construction.

The carriageway runoff will be collected in cesspits and piped to a storage tank where primary treatment will take place with screening and sedimentation facilities.

During initial phases of the overpass operation, the surface water will pass through the primary treatment and tank and discharge to ground via soakage.

Where elevated ground water levels begin to affect the performance of the discharge to ground a pumped solution will be required. A pump system has been designed to take accumulated water in the tank and to discharge this flow to an adjacent soakage field for disposal to land. As the discharge area is outside the "greater than 6 m to groundwater [from existing ground level] zone" direct discharge to land is permitted without treatment. Primary treatment of the discharge to land will still be as set out above. The low traffic volumes and limited length of road discharging to land at this location will generate only a small amount of contaminants with a minor localised effect.

A pumped surface water disposal field has been identified 400 m in a south easterly direction from the Robinsons Road Overpass. The Robinson Road under drainage system, pump well and disposal pipe are proposed to be constructed as part of the initial construction phase. However, the mechanical/electrical system would be employed some years later when the groundwater highs get to within 1.5 m of pavement level. The surface water pumping system is planned to operate at an expected design pump rate of 5-10 L/s.

In addition to the surface water pumping (primary) system and as groundwater levels rise a proposed (secondary) groundwater pumping well and system would be required to maintain ground water levels below pavement levels for as long as is practical. It is proposed to extract water from a well or wells drilled into the shallow aquifer below the pavement. The pumping rate within the extraction well would control the ground water level. The initial extraction rate is expected to increase over time. The latest study (September 2012) suggests a rate in excess of 100 L/s extraction rate may be required to control groundwater to an adequate level below pavement.

It is proposed to pump the extracted groundwater into the stockwater race adjacent to Robinsons Road. The expected capacity of the secondary pumping system is not expected to exceed 100 L/s.

5 Options Considered

As with all projects of this size a range of design options have been considered for the Project. Some of these issues were debated early in the design sequence and in general affect the whole Project or significant elements others are more of an individual discipline only option.

This chapter discusses options considered and the reasons why those options were accepted and/or rejected.

5.1 Project Vertical Alignment

5.1.1 MSRFL

In order to maintain connectivity to existing infrastructure, stockwater races, side roads and other local access it was chosen early in the process to maintain the existing vertical profile of this section of carriageway.

5.1.2 CSM2

Based upon early considerations of landscape, geometrics, adjacent community aspirations and the preliminary work carried out by Opus Consultants, there was a desire to lower the first section of the alignment into a trench from the southern end of the CSM1 extent. The advantages included reduced noise and visual effects and lowered height of underpass and approach embankments.

Following groundwater analysis and considerations regarding the discharge into Montgomery's Drain and Upper Knights Stream, the option of placing the whole Project alignment into a trench had a series of issues, potential conflicts and a lack of ability to discharge Project runoff to groundwater.

More particularly, the existing record of groundwater historical highs had predicted a groundwater table at around 4 m below existing surface level at Halswell Junction Road. Further to this CPW is projected to raise the groundwater in the area. The certainty of the prediction is low thus a conservative figure of 1 m has been applied in the analysis.

Thus CSM2 geometry, cross fall, side drainage and gradient to convey water, store surplus water and still maintain a freeboard above long term highs in the water table forced the decision that CSM2 trenched construction was not possible without pumping either of groundwater or of stormwater runoff.

As such a decision was made relative early in the design process to raise the centreline vertical alignment to at grade and to raise the associated ramps to pass over the CSM2 alignment using elevated ramps. Thus the stormwater system would be below notional ground level to accommodate CSM2 cross fall and the balance of the drainage mechanisms.

Overall the proposed alignment is typically at or near grade to allow the disposal of stormwater above design groundwater levels, minimise the depth of stormwater crossings and stockwater race siphons, and minimise the risk of road closure due to flooding. The elevation of the design groundwater level limits the depth to which the vertical alignment can be placed into a trench. Allowing for cross fall across the CSM2, a verge, swale and soak pit, the design disposal layer is typically and at least 1-2 m above the design groundwater level.

A range of geometric options was considered for the Main South Road, CSM2 and Robinsons Road intersections. From a geometric view, the chosen option was to place Robinsons under CSM2 (overpass). However from a stormwater and groundwater perspective, the Robinsons Road was at risk of inundation in extreme events from a combination of effects of CPW and from rain events. The road has a low traffic count (less than 600 vpd) and SDC advised short periods of closure was acceptable. The preliminary design has been carried out to limit the effects of closure by a range of pumping (of groundwater) options. Tanking of the underpass was considered but rejected based upon high physical cost.

5.2 Discharge of Surface Water Runoff

Options considered for conveyance and discharge of runoff included the following:

- Kerb and channel and piped system. In general a kerb and channel collection system was rejected for a variety of reasons, including: (a) cost; (b) difficulty in having a dedicated downstream discharge mechanism with the capacity to accept the additional runoff; and (c) it is not in keeping with the rural character of roads in the area. However, kerb and channelling has been adopted at low points and at interchanges to minimise the depth of the drainage system and to accommodate narrower bridge spans than would otherwise be required. Kerb and channelling is also recommended in these areas due to other design constraints outside of the drainage design (e.g. traffic considerations).

- Kerb and channel will be adopted on ramps and on the Project infrastructure around Halswell Junction Road. The length has been minimised to allow the greatest length of sheet flow over the grassed verge (which forms the first component of the stormwater treatment system).

- Discharge of surface water. The option of constructing a piped discharge network to discharge to the Selwyn River and/or directly to the Halswell River was discounted due to the significant costs for such a system.

- Global pumping options were rejected due to cost and the ability for a pumped system to operate on a near continuous duty during rain events given the variance in inflows (due to the range in actual rainfall events). Difficulty with testing of stormwater pumps on a regular basis, due to the relatively dry climate, was also a consideration. Pumping options were only considered in very rare instances where other solutions were not possible, such as Robinsons Road

- Conveying runoff to ponds and dedicated larger disposal fields was considered but rejected due to having to designate and purchase larger blocks of land to accommodate these features and
- Collection by swale and discharge to land has been adopted for the majority of the Project alignment. This has resulted in a wider footprint compared with a kerb and channel and piped system.

5.3 Stormwater Treatment

The treatment of stormwater is required by NRRP in areas mapped as less than 6 m to groundwater. Collection and treatment in swales was considered an acceptable methodology as this is used throughout the region. The methods set out in the NRRP are prescriptive to achieve a permitted activity status. Most of the Project is compliant with the NRRP rules regarding stormwater treatment and disposal.

Proprietary stormwater treatment devices have not been considered due to high cost and high maintenance requirements.

5.4 Stockwater Race crossings

Nine existing stockwater races cross the proposed alignment. The vertical grade of the alignment was determined early in the design sequence and shifting of the alignment to accommodate open channel flow of the stockwater races was not practical or cost effective.

A range of considerations were examined including:

- Closing all stockwater races was not an option that would be acceptable to the users of those races or to SDC. The stockwater races also have a dual function of providing land drainage during heavy rain and providing environmental flows to the Halswell River.
- Closing sections of some individual races and/or rerouting of races has been considered where it can minimise the number of stockwater siphons, an alternative supply is possible to service those customers affected and if the race can be reconnected downstream.
- Pumping was discounted on grounds of cost and reliability.
- With respect to fish passage, the design will accommodate the passage of fish, where deemed necessary through the ecology assessment.
- A range of single-conduit and duplicate conduit siphons was considered. Duplicate siphons allowed for a low flow pipe to operate maintaining velocity to avoid sedimentation whilst the

duplicate larger diameter was to be used for the passage of storm flows with minimum head loss.

The stockwater race network is owned and maintained by Selwyn District Council. There are a number of Selwyn owned races within Christchurch City jurisdiction.

The stockwater race along Marshs Road has no individual property takes after the race passes under Shands Road. SDC advised[6] that the status of the race can change from a stockwater race to a land drainage race downstream of Shands Road. This race continues down Marshs and Springs Roads, flows overland then joins the Upper Knights Stream as a feed for the Halswell River during periods of low flow.

5.5 Overland Flow Paths

The Project crosses approximately 10 overland flow paths in addition to the stockwater race flow paths set out above. Options to deal with these included:

- Ignoring the overland flow paths and making the assumption that soakage would be effective. If soakage was not effective the overland flow generated in the upstream catchment would discharge to the highway drainage network. Should the highway drainage network be overwhelmed it would spill over the Project alignment centreline (if the cross-connection pipe capacity was exceeded) and fill the downhill swale before continuing as overland flow downstream of the Project alignment but not necessarily in the same location. This option was rejected as it would require detailed engineering to fix the alignment and a full topographical survey to ascertain the extent of flooding and the effects to the same or different flow paths downstream of the Project alignment.
- Allowing overland flow to enter the Project drainage swale. This option was rejected because of potential overloading of the highway drainage system.
- Pumping, and storage options were ruled out because of cost and maintenance and the increase in designation area and its associated land take required to store the necessary volumes.
- Collection of overland flows upstream of the Project alignment and passing the flow under the Project alignment in an inverted siphon and discharge of that flow on the downstream side of the alignment. This option was adopted.

5.6 Designation Width

A range of options was considered for the ultimate configuration of the Project alignment and especially outside the interchanges. Narrow corridor options were rejected as stormwater treatment and conveyance could not be achieved with the rural flat landscape.

[6] Vicki Rollinson, SDC Pers Comms August 2012

The chosen option considered a range of stormwater treatment capacity, disposal areas and maintaining hydraulic separation of potential overland flows from outside of the catchment interfering with the conveyance and treatment of Project carriageway collection, treatment and runoff. The designation was enlarged to also accommodate collection of overland flows from upstream of the Project designation and the passage of these flows beneath the Project and discharge downstream.

Similar considerations were given to determining the Project extents around interchange embankments. In addition there are additional criteria and these interchange and over/underpasses is to have sufficient room to construct each embankment, maintain a temporary flow of traffic, manage erosion and sediment control works then have sufficient room for the construction of the stockwater race network.

This Project relies on the capacity of the soakage system (swales) to store the difference between the run off rate and the soakage to ground discharge rate. This requirement is in addition to the NZTA Stormwater Treatment Standard and is particular to this Project. The reason for this is to minimise the designation width and to avoid discharge to land outside of the Project boundary.

6 Detailed Project Description

This chapter details the design of stormwater management systems including conveyance and treatment devices for the MSRFL and CSM2. It also details proposed stockwater race diversions, crossings and in some cases infilling. Specific mitigation measures regarding disposal and water quality are also discussed. This section uses the same division of the proposed works as utilised in Section 3 of this report. A description of the wider design philosophy is provided above in Section 4.

6.1 Traffic Impacts

Table 9 Reproduction of Table 6-1 from Traffic Report (#2)

CSM2 and MSRFL and Baseline Average Daily Traffic Volumes – CSM1 Corridor

Road and Location	CSM2 & MSRFL			Baseline		
	2016	2026	2041	2016	2026	2041
Brougham St: West of Selwyn St	59,250	62,300	65,950	57,850	61,050	63,050
CSM1: Between Barrington St & Curletts I/C	64,050	68,950	73,550	61,400	65,000	67,100
CSM1: Between Curletts I/C & Halswell Jn Rd	39,200	47,650	54,750	32,950	37,250	40,850
CSM2: Between Halswell Jn Rd & Shands I/C	19,800	27,150	32,850	N/A	N/A	N/A
CSM2: Between Shands I/C & Main South Rd	15,950	21,700	27,100	N/A	N/A	N/A
[Halswell Jn Rd: North of Springs Rd]	18,800	22,350	25,950	27,850	32,350	35,900
[MSR: West of Halswell Jn Rd]	10,100	13,750	17,150	24,000	29,700	34,350
[MSR: West of Marshs Rd/ Barters Rd]	15,400	19,050	22,450	26,400	31,600	36,200
MSR: West of Robinsons Rd/ Curraghs Rd	25,250	34,550	44,300	23,550	29,500	35,100
MSR: West of Weedons Rd/ Weedons Ross Rd	24,150	31,350	38,050	21,950	27,600	32,400

Road locations enclosed in [ ] are bypassed by the CSM2 and MSRFL Project

Table 7 shows:

- As a result of the attractiveness of the CSM2 route there is a further traffic increase over and above the baseline case (i.e. without the Project); and
- Reduction in traffic volumes on MSR between Halswell Junction Road and CSM2 junction once CSM2 is in operation.

From a stormwater conveyance perspective the stormwater infrastructure proposed as part of the Project is not affected by traffic volumes.

However from a water quality perspective the traffic volumes do influence the contaminant load. The design of the Project has been cognisant of this and has developed a series of water quality treatment facilities along the Project which meet the rules set out in NRRP.

6.2 MSRFL

This section outlines the design of stormwater management systems including conveyance and treatment devices for the MSRFL.

6.2.1 Park Lane to Weedons Ross Road (Chainage 1350 m – 3100 m)

Refer to drawings 62236-A-C401 to C403, C407 and C408 in the Plan set contained in Volume.

6.2.1 A Project description

As described in Section 3.2.1, the carriageway alignment slopes gently down to the east from the high point adjacent to Park Lane (Ch 1783) to the first low point at ch 2860 just before the crossroad intersection with Weedons Ross Road (3025 m). The alignment climbs approximately 1 m in elevation at chainage 3425 m before falling to the CSM2 connection.

The intersection with Weedons Ross Road is to be upgraded significantly with the addition of on and off ramps to replace the existing cross road arrangement. Weedons Road will be realigned to pass over SH1, and two roundabouts will provide the intersections from the on and off ramps with Weedons Road. As part of the works, the Jones Road / Weedons Ross Road intersection to the north of SH1 will also be replaced with a new roundabout.

The MSRFL widening to four lanes is proposed to incorporate swales that will provide slow and shallow depth flow for stormwater runoff treatment. The swales will drain to a series of soak pits constructed along the MSRFL alignment on both sides. The spacing of the soak pits has been used to ensure that storage is activated within the swale and to ensure that the water quality depth for swale flow is not exceeded in the small design events. Inclusion of storage in the design reduces the need for high disposal rates directly to ground, which would otherwise be required without swale storage.

There is super elevation reaching 3.5% over the horizontal curve at Weedons intersection. Central median drainage will be required utilising sumps and pipe work. The discharge point of the pipework will be to the head of the adjacent swale to ensure that the maximum possible water quality treatment is achieved prior to discharge.

6.2.1 B Stockwater Races

An existing stockwater race is located on the east side of Weedons Ross Road and flows north to south. There is a race junction just north of Jones Road intersection. The race continues under South Island Main Trunk (SIMT) rail along Weedons Road, under MSRFL and continues down Weedons Road to the south. This race will continue to operate in a similar fashion at the completion of works. However the section between Jones Road and MSRFL will be upgraded to account for the reduction in conveyance by the partial closure of the race that passes through the Digga-Link site.

A series of simple sluice gates on the race network controls flow at each of the junctions in the network. These are often locked and under control of the SDC race network operator.

Proposals for modification of the stockwater races in the area comply with the design philosophy set out in Section 4.

For this section of works, the above race will require to be piped and or the conveyance pipework upgraded as follows;

- Under Jones Road roundabout the existing race and pipework will be upgraded to take additional flow, to handle the land drainage function and to take the flow that otherwise was diverted through the Digga-link site.

- Under SIMT, an upgraded pipe is required. This pipe will extend approximately 100 m towards SH1 to manage the traffic widening as part of the partial clover leaf construction.

- Under SH1. The existing section of the super elevated SH1 carriageway will be widened to 4 lanes as part of the MSRFL works. The full super elevation will have no alternative overland flow path and as such an upgraded pipe will be required to pass the land drainage function of the race network without inundation of SH1 carriageway.

- At the southern end of the above culvert a new race junction structure will be constructed to control flows down Weedons road and to feed the race relocation along beside the new MSRFL carriageway to the east. This will be discussed in more detail in section 6.2.2 below.

6.2.1 C Conveyance

An overland flow path has been identified that conveys flow to a dip in the existing embankment on the north side of SH1, at chainage 2500 m. It is proposed that a soak pit be constructed here to reduce the risk of flooding the highway. Protection of the highway drainage system will be maintained by extending the existing embankment between the swale and the proposed soak pit. Further improvements will be made to the existing Jones Road and railway culverts to further reduce the potential of overland flow reaching SH1 from the west. The catchment to be serviced by this soak pit is small.

Soak pits will be strategically placed around the Weedons Ross Road / Jones Road roundabout, and the railway level crossing. The land encircled by the proposed on and off ramps will be re-contoured to drain to proposed soak pits at the north east corners. This will reduce the contributing area to any existing flooding locations, giving a benefit to adjacent landowners.

In the event of an over-design rainfall event across the entire alignment the storage in the system will be filled and exceeded. Water will flow to the low points in the system, one of which is a sag point adjacent to Weedons Ross Road. Additional disposal footprint was included to further reduce the chance of spill from the system in events exceeding the design event. A range of site specific soakage testing results has given rise to a lowered design disposal rate used at the interchange. Further site specific soakage testing is required during the detailed design phase to confirm or update the design disposal rate. There is potential for water to spill out of the system to the existing overland flow path downstream of the alignment in very rare events. Given the infrequent occurrence of this spill the effects on the downstream race are considered minor.

There are two emergency spill points proposed to the stockwater race system. These will be from the inside of the clover leaf interchange where extended soakage systems are proposed. Spill would be limited to exceedingly large and/or long duration rain events.

6.2.1 D Stormwater Treatment

Treatment will be provided by the water quality swales and soak pits proposed for this length of MSRFL by discharge from swale directly to soak pits.

The Project's effects on stormwater conveyance and water quality for the MSRFL section from Park Lane to Weedons Ross Road are expected to be less than minor. The existing arrangements have been considered and modified or improved. Within this location, the NRRP does not require treatment of discharges prior to disposal to land as this area is outside of the less than 6 m to groundwater zone. Notwithstanding this, the proposed highway drainage system will provide treatment through sheet flow over the grassed verge and through the grassed swale prior to discharge to land (as outlined in the design philosophy in Section 4).

6.2.2 Weedons Ross Road to CSM2 (Chainage 3100 m – 5875 m)

Refer to drawings 62236-A-C403 to C406 in the Plan set contained in Volume 5.

6.2.2 A Project description

As described in section 3.2.2, the carriageway alignment slopes gently down to the east from Weedons Ross Road to the extent of works for the CSM2 located at chainage 5875 m.

The existing culvert at the Digga-Link site will be modified or replaced depending on its existing condition. This will drain to the existing surface water path on the south side of the carriageway and will have to pass beneath the proposed stockwater race pipe. Proposals for this culvert are discussed in greater detail below.

As described in Section 6.2.1 super elevation is proposed on the bend in the highway adjacent to Weedons Road. Central median drainage will be required at the super elevated section utilising sumps and pipe work. The discharge point of the pipework will be to the head of the adjacent swale to ensure that the maximum possible water quality treatment is achieved prior to discharge. Sheet flow over the Project may occur during over-design events where the pipe work has insufficient capacity to pass flows to the disposal system. As described above events below the design storm event should be completely contained within the Project corridor, reducing potential flooding effects downstream.

Proposed swales will provide slow and shallow depth flow for stormwater runoff treatment. The swales will drain to a series of soak pits constructed along the MSRFL alignment on both sides of the carriageway. The spacing of the soak pits has been used to ensure that storage is activated within the swale and to ensure that the water quality depth for swale flow is not exceeded in the water quality design events. Inclusion of storage in the design reduces the need for high disposal rate, consequentially minimising the potential effects of mounding of groundwater.

6.2.2 B Berketts Road Intersection

Widening of MSRFL about Berketts Road to allow for access will prevent the adoption of the preferred swale drainage solution due to the limited width of the proposed designation. A kerb and channel collection system draining to a pipe network will be required to meet the 100 year design standard. It is proposed that the pipe network pass beneath the Project to a new pond. The depth of water in the pond will be approximately 1 m. The pond will provide water quality treatment prior to disposal to land.

6.2.2 C Rear Access Roads

Within the Project there is a proposed duplication of Jones Road to the east of the rail line. This new road is to be called the Western Rear Access Road and is to provide rear access to properties currently fronting onto the MSR between Weedons Road and Robinson Road. The road is proposed to be constructed at grade and will rise over any culverts currently under the rail that will need to be extended to the downhill side of the Western Rear Access Road. Downstream of the Western Rear Access Road the water will fall to a swale to provide treatment from road runoff and the swale will be designed for a capacity of up to the 50 year ARI event as per the SDC Engineering Code of Practice (SDC February 2012). Surface water will be disposed of to land via soak pits along the swale at regular intervals.

Within the Project and to the south of MSR is a proposed new road that is to be called the Eastern Rear Access Road and is to provide rear access to properties currently fronting onto the MSR between Berketts Road and Robinson Road. The road is proposed to be constructed at grade and will rise over any culverts and stockwater races. An existing stub road called Paige Place has been constructed to the south of Larcombs Road and a further stub road will be provided from Larcombs Road north to service further properties. Surface water will be disposed of to land via soak pits along the swale at regular intervals.

6.2.2 D Stockwater Races

There is an existing stockwater race that traverses down Weedons Ross Road. This pipe passes under Jones Road, SIMT rail and SH1. These crossings are proposed to be upgraded to handle the land drainage function of the race and to prevent potential ponding of flood water on the super elevated section where the old Weedons Road crosses the MSRFL. Works are described in Section 6.2.1 above.

Subject to agreement with the property owner, it is proposed to terminate the stockwater race that flows through the current Digga-Link site within that site at a new soak pit. The Digger Link site is adjacent to Weedons Road and shown on drawing 6223-A-C403 in the Plan set contained in Volume 5. This would eliminate the need to reinstate the stockwater race crossing MSRFL at chainage 3475 m. Minimising the number of pipe crossings beneath the Project reduces the risk of flooding resulting from blockage of the structures. It will also reduce the on-going maintenance burden of the system.

There is an existing stockwater race on the south side of SH1 that flows east from chainage 3075 m. It is proposed to close the race through the Digga-link site and to feed this race from the Weedons Road race from the junction on the south side of SH1.

The existing race that flows along the south side of SH1 is currently an open drain. As part of the Project it is proposed to pipe this race for a distance of some 2100 m. The proposed size is 600 mm in diameter which is larger than its minimum capacity and has been sized to allow for some redundancy and to allow for a limited quantity of sedimentation within the pipe. As discussed in section 3.1, SDC has little information on the dry and wet weather flows in the system therefore design assumptions have been made at this stage.

Further work is required at detailed design phase to confirm the design assumptions to ensure that the pipe achieves SDC objectives. The piped section of stockwater race flow will cross Larcombs Road and Berketts Road, and design of the pipe and backfill etc. will ensure protection of the pipe should limited depth of cover be available.

Piping the stockwater race has been proposed to minimise the required corridor width, thus reducing the effect of land purchase on adjacent property owners. Piping the race also increases the safety of the MSRFL section of road with no ditch within the 9 m clear zone. Piping the stockwater race will mean the conveyance of flow can be kept within the NZTA boundary, even though the MSRFL will not contribute to flows in the pipe. Outlets from the pipe will be provided by means of a weir structure at chainage 4750 m and 5150 m to ensure the existing races and appropriate flow split heading south can remain serviced. In addition new manholes at approximately 100 m centres will be located along the 2 km section of pipeline to allow for future maintenance access. These manholes will have scruffy dome lids to allow light into the pipe to facilitate fish passage should they enter the pipe network.

6.2.2 E Siphons

A siphon will convey the stockwater race on the east side of Weedons Road under SH1. The downstream siphon structure will have an outlet into the existing south bound stockwater race. It will also incorporate a weir structure to control the flow of water into both stockwater races including the proposed along the southern side of SH1.

6.2.2 F Conveyance

A series of overland flow paths have been identified with a potential to convey overland flows to SH1. Typically these potential overland flows would occur in a depression or old stream/river channel. It is predicted that the small catchment areas for some of these paths will generate insufficient volume to fill and overtop the Project alignment. Protection of the proposed highway drainage system will be required to ensure that the Project disposal system is not overwhelmed and cause a decrease in levels of service.

Protection against overland flows is proposed by way of shallow bunds or gradual re-contouring of the ground surface to the west of the Project but just within the designation. The stormwater prevented from entering the highway drainage system will soak to ground as per the existing situation. The highway drainage soak pits have been located away from overland flow paths where possible to minimise the width of the drainage systems at the overland flow path locations. In some locations construction of low timber flood walls may be required at the top of the bunds to minimise the height and footprint of the earth bund, reduce land requirements and consequential impacts on adjacent property owners. The timber flood walls are discussed in the Urban and Landscape Design Framework (Technical Report No. 6). The existing railway and Jones Road continue to form an obstruction to overland flow reaching SH1 from the north. Alternatives such as re-contouring within private property were considered but dismissed as land owner consent cannot be assured.

There is an existing low area within the Digga-link site. This area currently drains into the adjacent stockwater race. As the proposed stockwater race at this location is proposed to be in filled, an alternative arrangement will have to be made for the draining of this site. It is expected that a new culvert under MSRFL will be required to allow the passage of flood waters under SH1 to the south.

6.2.2 G Stormwater Treatment

As part of the widening of this section of SH1, a new swale is proposed to be constructed on the north side of the carriageway to collect runoff from the carriageway, treat within the newly formed swale and include discharge to ground soak areas at approximately 200 m intervals.

A similar method of collection and treatment is proposed on the southern side of SH1. However this side will need to accommodate the shallow piped stockwater race feed pipe as well.

The NRRP does not require treatment of discharges prior to disposal to land as this area is outside of the less than 6 m to groundwater zone. Notwithstanding this, the proposed highway drainage system will provide treatment through sheet flow over the grassed verge and through the grassed swale prior to discharge to land (as outlined in the design philosophy in Section 4).

Along the Western Rear Access Road a swale would collect runoff from the road and soak pits provided at regular intervals.

6.3 CSM2

This section outlines the design of stormwater management systems including conveyance and treatment devices for the CSM2 works. It also outlines proposed stockwater race diversions, crossings and in some cases infilling. Specific mitigation measures regarding disposal and water quality are also discussed.

6.3.1 MSRFL/CSM2 to Blakes Road (Chainage 0 m – 3800 m)

Refer to drawing 62236-B-C401 to C407 in the Plan set contained in Volume 5.

6.3.1 A Project description

CSM2 commences approximately 400 m south of the existing Robinsons Road intersection. From this intersection, the alignment of CSM2 diverts north east into farmland. It will cross Robinsons Road by means of an overpass[7], and Waterholes Road and Trents Road by means of an underpass[8].

Overland flow path flows arriving at the alignment will be passed below CSM2 or disposed to land via soak pits.

A stormwater bund is proposed (within the Project footprint) to contain site runoff within the property on the north west corner of the Robinsons Road intersection that is potentially contaminated. Excavation here will be minimised and it is the intention that the proposed stormwater treatment and conveyance devices in this location will receive no runoff from this site. The natural fall of the ground on this property is to the south east, so the affected property will continue to drain adequately.

As part of the design for the Robinson Road overpass (i.e. Robinsons under CSM2 alignment) will be a sump and facility for a future pumping system. This is covered in more detail below.

A new roundabout is proposed on Main South Road and Waterholes Road. This intersection will be upgraded to a roundabout. Traffic from the south will flow north at grade along the existing alignment. Flows to the south (from Waterholes roundabout) will be grade separated and pass over the new CSM2 alignment.

Trents Road will cross CSM2 by means of an underpass (i.e. Trents over CSM2 alignment). As described in section 6.1, run-off from the local road carriageway will typically be captured by kerb and channel, sumps and pipe work which will be piped to stormwater treatment swales and disposal adjacent to the toe of the embankment.

Blakes Road will terminate on the north and south side of CSM2. The existing Blakes Road is a collector for overland flow and an overland flow siphon will be required to maintain the land drainage function of the existing stockwater race.

6.3.1 B Stockwater Races

An existing stockwater race flowing south runs along the west side of Robinsons Road, crossing below SH1 at approximately chainage 350 m. As part of the Project the existing stockwater race will be re-aligned to the south and within a piped system to avoid the excavation for the overpass. The pipe will be passed below SH1 and continue to the east of SH1 until clear of the excavation line.

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[7] Overpass by convention is in relation to the main alignment. In this case the major CSM2 alignment passes over minor Robinson Road.

[8] Underpass by convention is in relation to the main alignment. In this case the major CSM2 alignment passes under the minor Trent Road.

Manholes will be placed at strategic intervals to allow maintenance of the pipe and siphon structure. The function of the race will be maintained without significant effect.

The existing stockwater race at the intersection to be formed between SH1 and CSM2 at chainage 800 m will also be passed below the proposed CSM2 alignment. The function of the race will be maintained without significant effect. Again manhole access between the existing SH1 and the new southbound onramp will be included to provide maintenance access.

There is an existing stockwater race along Waterholes Road that crosses SH1. This section of pipe and open watercourse will be piped and diverted around the intersection with access points to allow for maintenance access.

The existing stockwater race at the proposed Waterholes Road CSM2 underpass at chainage 1950 m will be passed below the proposed alignment, realigned to suit the extents of the proposed embankments for the underpass. The function of the race will be maintained without significant effect.

There is an existing stockwater race flowing south on the west side of Trents Road and on the east side of Blakes Road.

Trents Road will cross CSM2 by means of an underpass and modifications to the Trents Road stockwater race will be required as part of these works. It is proposed to realign the race along the bottom of the western embankment that will form the underpass. The stockwater race will pass below CSM2 and continue on in the new alignment until the proposed embankment ends, allowing the race to recommence its original alignment. On the south west corner of the crossover point of CSM2 and Trents Road, a branch departs the main stockwater race and heads west before turning south, crossing the proposed CSM2 alignment at chainage 3100 m. This branch connection point will be removed as part of the underpass works, and it is proposed that this branch be in-filled subject to property owner consent or a soak pit be placed downstream of the final race user. This will reduce the requirement for a stockwater race crossing at chainage 3100 m. The branch will recommence on the south side of the CSM2 alignment, picking up a new branch that will come from the realigned stockwater race on the western side of the Trents Road underpass.

It is proposed that the stockwater race on the south side of Blakes Road will be maintained but with reduced stockwater conveyance. This would terminate adjacent to the upstream side of the motorway in a soak pit. The overland flow component of the race would remain and a new siphon to be constructed under the CSM2 alignment. The race inlet at the Trents Road intersection will be closed. The overland flows will be conveyed within this existing race channel and this will pass below CSM2 alignment by means of a siphon.

6.3.1 C Siphons

A series of siphons and associated structures will be constructed to pass stockwater race flows below the CSM2 alignment at Robinsons Road overpass, SH1 divergence intersection, chainage

1300 m, 1550 m and 1750 m, Waterholes Road underpass, Trents Road underpass, Marshs Road underpass, Marshs Road, Springs Road underpass and the Halswell Road underpass.

A siphon will also pass overland flow below CSM2 at chainage 1175 m, 1750 m, 2500 m, 2800 m, 3300 m, 3800 m, 4350 m, 6450 m and 7000 m. Typically collected flows have been redirected to the upstream siphon structure by means of a stormwater bund on the western side of CSM2 alignment.

6.3.1 D Conveyance

Overland flow paths have been identified based on the topography of the land adjacent to the CSM2 alignment. Generally these overland flows arriving at the upstream side of the CSM2 will be detained by a stormwater bund, collected at an upstream siphon structure and conveyed to the opposite side of the carriageway via a siphon arrangement at the locations identified above. As discussed above in Section 4 the inclusion of overland flow siphons reduces the effect of flooding from the Project on adjacent property owners.

6.3.1 E Robinsons Pumped System

The Robinson Road overpass has been designed to allow traffic to pass under the CSM2 alignment that is to be constructed at grade. The existing groundwater profile is some metres below the proposed Robinsons Road carriageway. The CPW project has been consented and is now part of the planning landscape.

Once the effects of CPW as felt on the groundwater system there will be a predicted increase in groundwater levels of up to 4 m at this location. As a result the future groundwater highs are predicted to peak above carriageway level.

There are a number of ways to deal with an inundation of the carriageway as follows.

- Let the groundwater pond on the carriageway and close the road. Although this is acceptable to SDC for short periods, the community has not yet had input on this aspect.

- Construction of a wide footpath on one side of the carriageway that has the capacity to allow the passage of light vehicles at an elevation above the main carriageway but with reduced headroom clearance.

- The carriageway falls to a low point where stormwater can be treated and disposed to ground where the groundwater is some meters below carriageway level. However in future years a pumped system is required to intercept disposal to ground and convey this flow some 300 m to the south and dispose to ground in an open basin. The maximum disposal rate for the disposal basin is up to 10 L/s. A pump out rate of 10 L/s would be sufficient to keep the carriageway dry for most rain events as long as the groundwater level was below pavement level. However as groundwater level rises this pump rate would not be sufficient.

- In order to maintain the road open, a much higher pump out rate is required. One or more ground water extraction wells will be required to maintain a groundwater level at or below subgrade level. In order to dispose of up to 100 L/s a different disposal system will be required and this will discharge to the adjacent stockwater race. A meeting with SDC in August 2012 advised that approval in principle was given on the proviso that the NZTA would be responsible for any associated downstream capacity maintenance / improvements.

The extent or period of these future predicted ground water highs remains uncertain but expected to be relatively short. I.e. pumping may be required for a number of days (or just weeks) every few years. The level at which the groundwater will be below 95% of time is approximately 2 m below the low point in Robinsons Road.

6.3.1 F Stormwater Treatment

Stormwater treatment for this section of CSM2 will consist of a similar treatment set out to that described for the MSRFL section above. The depth to groundwater for this section is outside the 'less than 6 m to ground' and as such does not require treatment prior to discharge to ground, under the NRRP. Nevertheless a system of swales either side of the carriageway will collect sheet flow from carriageway runoff and provide treatment to a high standard (refer to **Appendix F** Aqua Terra Contaminant Load Assessment). This is important as the actual depth to groundwater following the effects of CPW is expected to be less than 6 m in some locations outside of the NRRP 'less than 6 m to ground' for periods in the future. Soakage to ground will occur and at 300 metre centres or less, soakage to ground pits will be excavated and backfilled with appropriate rock media to allow accelerated disposal to ground.

The Rear Access Road set out in section 6.3.2, will receive similar treatment to that above. I.e. collection in swales and discharge to ground at regular intervals by use of soak holes.

There are embankments proposed at Main South Road south bound onramp, Waterholes, and Trents underpasses. Stormwater runoff from the carriageway will be collected by kerb and channel and fed to stormwater treatment devices. Water from the embankments will flow down the embankment and will soak to ground at the base of that embankment.

The existing Robinsons Road intersection will be renewed as part of the CSM2 scope of works by means of an overpass (CSM2 over Robinson). Runoff from the carriageway either side of the low point will fall to the low point and be collected in sumps and piped to a holding tank. This tank will act as a gross pollutant trap and sedimentation chamber. As such maintenance will be required. In the initial years of operation flow will be allowed to discharge to ground. However once the effects of CPW are realised then discharge to ground will no longer function and pumping will be required.

Installation of the primary pumping system would occur in stages with all non-'mechanical & electrical' infrastructure installed during construction of the road (including: road sumps and pipe work, pump sump, disposal field and manholes). The installation of the 'mechanical & electrical' equipment would be installed after the effects of CPW could be measured and/or the 1 m clearance to groundwater highs cannot be confidently maintained. An additional stand by pump will also be allowed. It is anticipated that groundwater pumping will occur only when the groundwater is within 600 mm of the pavement level.

The detailed design will need to ensure that the catchment draining to the low point is minimised. Operation of the stormwater disposal system would be maintained until flooding occurs, at which time pumping of the stormwater would be required.

Given the seasonal variation in groundwater levels and the uncertainty of the potential effects of CPW it will not be possible to guarantee the 1 m clearance to groundwater recommended in the NRRP for disposal of the small contributing area either side of the overpass. The environmental effects of the 1 m buffer not being constantly maintained will be less than minor for the following reasons:

- For the majority of the time the groundwater buffer will be maintained
- The future traffic loading predictions are low (approximately 600 vehicle movements per day)
- In the highest groundwater conditions, runoff will be pumped to the higher-level disposal area which will have more than 1 m clearance
- The small contributing area which is exposed to rainfall (totalling approximately 100 m of local road).

A disposal field is required adjacent to Robinsons Road to dispose of the pumped stormwater and groundwater from the overpass. The disposal area will only be utilised in elevated groundwater events. The final disposal area will be in the order of 0.3 ha with the storage required being dependent on the pump rate selected in detailed design. Given that the groundwater inflows will occur over extended durations (effectively 'infinite' volume) the design disposal rate will match groundwater percolation rates at the base of the overpass and any storage will only be to buffer pump operation cycles.

In addition to the primary system above a secondary system of potentially large capacity will need to be designed to reduce groundwater levels. Initial indications are that flows up to 100 L/s may be required to be disposed of to keep groundwater below base of pavement level.

6.3.2 Blakes Road to Before Springs Road (Chainage 3800 m – 7100 m)

Refer to drawing 62236-B-C407 to C411 in the Plan set contained in Volume 5.

6.3.2 A Project description

As discussed in section 3.3.2, the portion of CSM2 from Blakes Road to before Shands Road continues through farmland crossing Shands Road at chainage 5350 m and Marshs Road at chainage 5950 m, and also the existing disused Little River railway line at chainage 6600 m.

Shands Road will become an interchange with north and southbound on and off ramps. The main Shands Road alignment will cross CSM2 by means of an underpass. Land encircled by the on and off ramps and CSM2 will be re-contoured to suit proposed runoff disposal systems that are discussed below.

The Shands Road and Marshs Road intersections will be upgraded as part of the works.

Marshs Road will cross over the CSM2 alignment by means of a new underpass.

The existing disused railway line at chainage 6600 m will be crossed by the CSM2 alignment.

6.3.2 B Stockwater Races

Modifications to the stockwater race network will be required as part of the works for this section.

An existing stockwater race flows down the north side of Marshs Road and within the current road reserve. The Project embankment extent required for the Marshs Road underpass is approximately 800 m in length. The embankment width extends beyond the existing road reserve, and the toe of the embankment is approximately 12m from the old road reserve boundary. The designation boundary extends further out with sufficient room for construction and the relocation of the old stockwater race that is to become a land drainage race. The race will cross under the Project alignment at chainage 8100 m and continue in a new alignment at the toe of the embankment until meeting the old race some 200 m further to the south east.

6.3.2 C Siphons

Consistent with the Project thus far, siphons and associated structures will be constructed to pass stockwater race flows below the CSM2 alignment at the Marshs Road and Shands Road intersection and at chainage 5800 m.

The realigned stockwater race inside the Marshs Road intersection is protected against overland flow therefore a second siphon will be required at a similar diameter to the dry weather flow siphon to allow maintenance and to minimise the risk of blockage resulting in flooding of the State highway.

Overland flow paths have been identified based on the topography of the land adjacent to the CSM2 alignment. Generally these flows will be detained by a stormwater bund, collected at an upstream siphon structure and conveyed to the opposite side of the carriageway via a siphon arrangement. A siphon below CSM2 at chainage 7000 m will also pass overland flow that has been concentrated from the wider low area to the upstream siphon structure by means of a stormwater bund on the north side of CSM2 alignment.

6.3.2 D Conveyance

Marshs Road intersection with Shands Road, and the Shands Road underpass runoff will be captured by kerb and channel, sumps and pipe work. Treatment of these flows is discussed below.

6.3.2 E Stormwater Treatment

Stormwater collected via kerb and channel will be discharged to swales at the base of the embankment to facilitate stormwater treatment via the swale prior to disposal. In some isolated instances it will not be possible to discharge from the kerb and channel at the upstream end of the swale and so discharge from the kerb and channel will be part way along the swale. The effect of this reduced treatment will be less than minor on the groundwater receiving environment given that:

- Capture of first flush and percolation via an organic filter layer will still occur at the treatment design soakage areas within the ‘less than 6 m to groundwater’ zone
- Depth to groundwater outside of the ‘less than 6 m groundwater zone’ will permit treatment in ground prior to reaching groundwater, noting that the NRRP does not require water quality treatment in these areas.

Drainage of the CSM2 on and off ramps and the main CSM2 alignment will be drained to land via roadside swales and soak pits.

Soak pit water quality dry ponds are proposed between chainage 6100 m and CSM1. These dry ponds will provide a storage area for stormwater, where flow can pass through a filter bed to remove contaminants. The ponds will be located at the downstream end of the roadside swales. These swales will drain to soak pits, however, the water quality dry ponds will provide further stormwater storage and treatment in periods of high flow. The bed materials proposed for the water quality dry ponds is set out in the proposed mitigation section (Section 9). The goal of the mitigation is to achieve the desired water quality treatment through the specification of the filter material rather than the percolation rates specified in the NRRP (WQL6). This approach for specifying basin filtration media has recently been accepted by ECan for the CSM1 Project.

6.3.2 F Specific Mitigation

As mentioned above, the runoff from the industrial zone to the south west of the Shands Road interchange is uncontrolled with uncertainty regarding runoff generation. To ensure this runoff does not cause flooding to the proposed CSM2 road and northbound off ramp to Shands Road, a stormwater bund extending from chainage 4050 m to Shands Road is proposed. This will divert runoff to a surface water soakage area. A cross drain will be installed below Shands Road to safeguard against overland flow flooding. The cross drain will drain into an area of land re-contoured to drain to a surface water soakage area. A soakage area is proposed for all areas of land surrounded by the Shands Road on and off ramps, and CSM2. Disposal levels for these soakage areas will be below adjacent swale inverts. The effect of this disposal of surface water on the groundwater receiving environment is very similar to the highway drainage system and is permitted under the NRRP (WQL6) subject to conditions, as discussed below in Section 7.4.

6.3.3 Before Springs Road to CSM1 (Chainage 7100 m – 8400 m)

Refer to drawings 62236-B-C412 to C414 in the Plan set contained in Volume 5.

6.3.3 A Project description

The final section of the CSM2 scope of works is at the north end of the site where the proposed alignment meets CSM1. An underpass will be constructed at chainage 7200 m to allow Springs Road to cross over the CSM2 alignment and a second underpass at chainage 7700 m will allow Halswell Junction Road to also cross over the Project. The existing Halswell Junction Road / Springs Road roundabout will be upgraded and the existing Halswell Junction Road/CSM1 roundabout will be removed. As well the intersection of Springs Road with John Paterson Drive will be realigned. A new south bound off ramp from CSM1 will be constructed at chainage 8300 m and will join Halswell Junction Road at a new roundabout. It is further proposed to join John Paterson Drive to the same intersection. The John Paterson Drive extension will pass over Montgomery's Drain, and pass beside the Upper Knights Drain. The construction adjacent to the drain will need to account for maintaining potential flow paths down the drain as the drain is notionally dry.

A north bound on ramp to CSM1 will be constructed at about the same chainage to collect traffic from the existing Halswell Junction Road roundabout. As part of these works the north bound on-ramp to CSM1 fill embankment will partially infill the Mushroom Pond and eastern first flush basin.

The CSM2 motorway will cross the existing Montgomery's Drain, which runs parallel to Halswell Junction Road. The drain over this section will be piped north and discharge into the Owaka basin. This is to be undertaken by this CSM2 project at the request of CCC to comply with the intent of the SWAP.

The proposed Owaka Basin and the Wilmers Road Quarry Disposal Area are stormwater assets that are included in the CCC's proposed SWAP scheme. The SWAP is discussed in detail in Section 3. Effects to this existing and proposed infrastructure by the proposed alignment have been minimised through the design process.

In general the SWAP proposals will remain unaffected by CSM2. The SWAP intends discharging the overflow from the Halswell Junction Road Pond into the Owaka Basin. This will require a new siphon beneath CSM2 motorway. Once the Owaka Basin is partially full, flow will then be diverted to the Wilmers Quarry disposal area. Once both Wilmers and Owaka are full then an overflow weir will allow flow to spill, pass under Halswell Junction Road and discharge back into the Montgomery's Drain adjacent to the new off ramp location.

The CSM2 proposals also include a high level spill to Montgomery's Drain from the CSM2 Maize Maze and Ramp Pond detention basins. The proposed vertical alignment of the outside lane of the CSM2 is elevated above the ground adjacent to the Owaka detention basin, so that the Project carriageway is unlikely to flood. This overflow would also have to protect the pond against backflow from the drain. A joint overflow with the Owaka Pond is proposed and we understand that this is acceptable to the CCC, however, level constraints (imposed by geometric design and road safety considerations) may lead to the partial inundation of the off-ramp carriageway.

Consideration was given to lifting the whole alignment but this had a significant cost, safety (as it would require steep vertical grades about the Halswell Junction Road Roundabout) and a visual penalty. On balance the short term potential flooding of the outer lane of the off ramp in extreme events was considered acceptable from a transportation perspective and that the effects on the environment were less than minor.

Based upon advice set out in **Appendix D**, the maximum future groundwater level predictions place the level some 750 mm above the pond inverts of Maize Maze, Ramp and CCC Owaka basin. As such and without intervention there is a real possibility that the high groundwater will flow into these ponds, reduce the pond capacity prior to large rain events and decrease the ability of the pond to drain to ground.

In order to mitigate for the effects of high groundwater in combination with the effects of the CPW, an artificial lowering of the groundwater beneath the ponds is proposed. A system of interconnected drainage wells and/or in combination with a pipe drainage network beneath the pond will act to artificially limit ground water level rise to a target level of RL = 18.5 m. The outlet of the under drainage system would be laid at depth and with flat grades. The outlet is proposed to continue along the realigned portion John Paterson Drive then deviate southeast and discharge to the Upper Knights Stream. This would pass through land under current ownership of Fulton Hogan Land Development Ltd, however we understand this land will pass to CCC and become an active playing field reserve.

In this way the pond capacity at the commencement of a large storm event is expected to be empty or near empty. The fill rate of the pond from surface water runoff is expected to be faster than the rising of groundwater from the same rain event. Thus the pond capacity would not be compromised from high groundwater.

The proposed vertical alignment at the connection with CSM1 poses the greatest difficulty with designing a functional drainage system. Elevated groundwater levels in the vicinity form a constraint on the invert of soakage disposal systems. This is accentuated by a number of issues:

- The sag or low point in the alignment being approximately 250 m from the nearest available land for treatment and disposal
- The proximity to the existing Halswell Junction Road roundabout limiting the elevation of the Halswell Junction Road underpass, thereby limiting the opportunity to increase the elevation of the Project to minimise the effects of carriageway inundation
- The location of the Owaka Basin
- The contributing area of the detention basin includes the northbound on ramp
- Construction of the northbound on-ramp will require backfill of approximately 5% of the CSM1 'Mushroom' Pond and its associated first flush basin. There is a reduction in the catchment currently draining to the Pond (by one third) and this flow will be re-directed south to the Maize Maze Pond. The remaining pond capacity will be sufficient to service the Halswell Junction Road roundabout and its associated impervious areas. The on-ramp and CSM1 contributing areas will be diverted to the Maize Maze Pond, thus ensuring that the CSM1 system still operates as intended.

The proposed vertical alignment and subsequent drainage design balances the required functionality of the system with the constraints listed above and the visual and noise effects.

The alignment will also intersect an open drain along the eastern edge of Springs Road. As Springs Road is to be elevated over the Project, the open drain will need to be realigned around the extent of the underpass embankments and culverted under the Project.

During construction of Springs Road it will be necessary to close the John Paterson junction with Springs whilst the Springs embankment is under construction. An extension of John Paterson to the CSM1 and Halswell Junction Road roundabout will be constructed early in the construction sequence. This extended John Paterson Drive will remain and provide access to a new Council park area.

6.3.3 B Field Drains

It is proposed to realign the field drain / race network away from the Springs Road embankment to eliminate the requirement for two siphon crossings beneath the Project. The functionality of the Springs Road land drainage race will be maintained.

6.3.3 C Siphons and other drainage infrastructure

Siphons and associated structures will be constructed to:

- Pass Montgomery's Drain below the CSM2 alignment at chainage 7600 m
- Connect Montgomery's Drain downstream of the above siphon to the Owaka Basin through the embankment for the Halswell Junction Road Underpass
- Connect the overflows from the Owaka Basin and the Ramp Ponds to Upper Knights Stream beneath Halswell Junction Road
- Provide for a discretionary draw down facility consisting of a pipe at a low level (near the base of the pond) connecting the proposed Maize Maze Pond directly to Montgomery's Drain to facilitate drawdown of the pond if the pond remains full for more than a few days after extreme rainfall and high groundwater events.

As discussed below the low level pipe will have to be controlled with a sluice gate so that the pipe is normally closed. The control of this sluice gate is discussed again in Section 8.3.1.

6.3.3 D Conveyance

Overland flow paths have been identified based on the topography of the land adjacent to the CSM2 alignment. Generally these flows will be detained by a stormwater bund, collected at an upstream siphon structure and conveyed to the opposite side of the carriageway via a siphon arrangement.

Runoff from the area enclosed by CSM2, Halswell Junction Road and Springs Road is drained to the north east corner where together with Montgomery's Drain flow will pass below the CSM2 alignment by means of a siphon. A soak pit water quality dry pond is to be located at the south corner of this area.

Drainage of CSM2, Springs Road underpass, and Halswell Junction Road underpass will be to land via kerb and channel systems discharging to roadside swales and soak pits.

It is proposed to spill into Montgomery's Drain in events exceeding the 100 year ARI design storm event. These flows will eventually reach the Halswell River but adoption of the high design standard should mitigate any flooding effects given that:

- The stormwater pond is sized to store the entire runoff in the 100 year 24 hour event
- The resulting spill in over-design events will be flows only in excess of the 100 year and may be less than the existing catchment runoff.

A second scenario may require discharge from the ponds. Should the groundwater level beneath the site become elevated (following the implementation of CPW in conjunction with a prolonged period of wet weather), the expected disposal rate to ground would reduce accordingly. This could lead to prolonged storage times and risk of overtopping from subsequent rainfall events. It is proposed to directly connect a low level pipe from the last pond cell to discharge to Montgomery's Drain adjacent to John Paterson Drive. A sluice gate on the pipe would normally be closed, but would be opened following the completion of the storm to facilitate drainage of the pond to allow for storage volumes in subsequent storm events. In order to draw down the pond volume over a period of 2 days, a discharge rate of up to 60 L/s will be required. The effects of this discharge on Upper Knights Stream have been discussed with the CCC planning and engineering officers (G.Harrington & K.Couling, pers comm., Feb 20112) and are expected to be less than minor because:

- Of the infrequent nature of the discharge;
- The discharge would occur well after the peak of the rainfall event, mitigating potential flooding effects;
- The quality of the water is expected to be high given the discharge from the most downstream 'bunded area' of the storage pond.

The potential effect on flooding will be further reduced by seeking dialogue and verbal agreement from ECan and the CCC Halswell River Managers prior to each time the low level drain from the pond is to be opened to allow accelerated draining of the pond after the peak of the flood event in the Halswell River. This will ensure that the pond is operated in a manner that would avoid making worse the effects of flooding on the River.

The base of the ponds are proposed to have staggered or undulating base (as opposed to flat). The purpose is to ensure the vegetation on the higher pond base level elevation would have a higher chance of survival during prolonged inundation (i.e. 60 hours, plus a 48 hour drainage time is longer than the typical 72 hours inundation guideline for vegetation survival). Therefore, grass cover may have to be reinstated after prolonged retention.

6.3.3 E Stormwater Treatment

The large stormwater treatment and disposal area at chainage 7500 m – 7600 m between Springs Road and Halswell Junction Road will be designed to contain all stormwater flows from the contributory catchment for rainfall events up to a 100 year ARI design storm event. It is proposed to spill into Montgomery's Drain in events exceeding the 100 year ARI design storm event. Figure 10 shows an example of a stormwater basin (the Awatea Basin) in the vicinity of CSM1 which was constructed as part of the SWAP.

Figure 10 Example of Engineered Stormwater Detention Basin Adjacent to Awatea Road

Detention basins are required due to low percolation rates and sometimes high groundwater levels. The location of these three detention basins coincides with the groundwater protection area so water quality is a primary consideration as set out in the NRRP. The detention basins required here include: one in the SW corner of the Halswell Junction Road / CSM2 intersection (main detention basin), and one adjacent to the proposed truck only off ramp connecting CSM2 to Halswell Junction Road (ramp detention basin) formed of two parts. This detention basin been shaped to avoid the proposed CCC Owaka Pond and the proposed cycle / walk way through this area (discussed in section 3.3.2).

It is not possible to meet all the NRRP requirements for this pond and thus obtain a permitted activity status hence consent has been sought for an alternative BPO approach which includes the following features:

- All stormwater runoff to be fed to a forebay/ first flush basin with lined base (to limit discharge to ground from the base of the forebay, until water has received treatment within the main body of the pond).
- Stormwater then flows to a second (storage and attenuation) basin with an organic filter media on the outlet of the basin (as described in Section 4.6.3). From the first flush basin and second basin, outflow feeds to a third basin where soakage to land will occur through the base of the pond and though an exfiltration gallery excavated to the south east of the last Maize Maze Pond as shown on drawing 62236-B-C425, (refer to Volume 5 for drainage drawings).
- Emergency spillway to Montgomery's Drain with a controlled discharge feature to allow draining of pond following large events to be operated with liaison from the CCC catchment/network and ECan River Managers.

The maximum water levels for these detention basins are driven by sag points in the proposed alignment, cross fall and longitudinal drainage to convey stormwater to the detention basin locations. Kerbing will be used on the perimeters of both detention basins to limit the depth below ground as much as possible. The invert levels/depths of the detention basins are limited by the design groundwater levels.

The Maize Maze detention basin is sized to cater for a contributing area including the balance of the north bound on ramp and most of the Halswell Junction Road underpass. The existing Mushroom Ponds will have to be partially in-filled to allow for the north bound on ramp. The remaining portion of the Mushroom Pond will continue to operate and collect flows from a much reduced catchment, being the approaches to the Halswell Junction Road roundabout. The design overflow from the Mushroom Basin is via a swale heading east from the pond discharging flow to the Wilmers Quarry. This function will be maintained with a swale or pipe from the pond to the culvert along the base of the new on ramp embankment.

A disposal field of underground pipes is proposed to fit within the same parcel of land as the pond and to allow the total storm volume to be disposed of within 24 hours. The detention basin depths have been minimised to allow the greatest depth between the disposal system and the groundwater level. It may be possible to raise the pond invert level in the next design stage by considering flooding of the piped drainage system or the edge of seal. The disposal system will be located adjacent to the detention basin, rather than beneath the detention basin, to allow cover on the pipe and raise the disposal system invert. A lowered disposal rate should account for the risk of elevated groundwater levels filling the pore space beneath the pond and disposal field. The disposal will rely on lateral groundwater movement which is slower given flatter hydraulic gradients during periods of high groundwater. Drawdown times will be determined by the lateral permeability of the ground.

Soak pits are proposed to be constructed at the downstream end of swales or at centres not exceeding 200 m. On Springs Road the stormwater runoff will be collected to the Maize Maze Pond where possible and where not through road side swales.

On John Paterson Drive treatment of carriageway runoff is proposed in a series of roadside swales as there is insufficient elevation to obtain fall from that road to any of the project ponds.

6.3.3 F John Paterson Drive

The existing John Paterson Drive is to be closed at its intersection with the new Springs Road underpass.

Currently, John Paterson Drive ends in a cul-de-sac. As part of the Project, the cul-de-sac will be modified and the road extended through to the proposed roundabout on Halswell Junction Road. This new road alignment and its associated earthwork batters cross what appears to be an historic overland path upstream of Upper Knights Stream. There is no evidence of overland flow in this location at present. As such the overland flow path will need to be realigned around the base of the road batter slope. The Upper Knights Stream is fed from Montgomery's Drain and generally only operates to convey stormwater when the Halswell Junction Road Pond overtops and spills.

The Upper Knights Stream is also fed from a SDC stockwater race that traverses from Marshs Road, along Springs Road before being diverted south to the south east of the Maize Maze ponds. The combined flow of the stockwater race / land drainage race plus the peak spill capacity from the Montgomery's Drain will need to be piped under the John Paterson Drive extension, and discharged into the Upper Knights Stream.

7 Statutory Analysis

7.1 CSM1

There is a series of consents that allow the CSM1 works. This Project involves the modification of a range of elements at the CSM1 and CSM2 interface including the Lee Basin, Mushroom Ponds and other infrastructure already constructed.

The works proposed as part of the CSM2 will modify a range of CSM1 structures and there is a potential to affect the operation phase requirements of those consents.

7.2 Christchurch City: South-West Area Plan

The CCC has proposed the development of a large area of land adjacent to the current south-western limit of Christchurch City. The area includes from Halswell Junction Road and its surrounds (extending north almost to SH1, and to the south to the City administrative boundary), and the upper extent of the Halswell River Catchment (Montgomery's Drain and Upper Knights Stream). The South-West Area Plan (SWAP) (CCC April 2009) highlights restoration and naturalisation of watercourses as the first goal to achieving the vision for development of land in the area. The plan was adopted by the CCC in 2009 and provides a description of the receiving environment for the discharges from the proposed stormwater ponds at the CSM1 connection:

> *"South-West Christchurch is characterised by an extensive network of waterways and floodplains. The water environment is highly sensitive to the effects of land-use activities. Without good management, urbanisation can lead to an increased risk of sedimentation and pollution. Urbanisation also increases impervious (sealed) surfaces with more stormwater runoff, especially peak flows, and the associated problems of flooding. A further consequence of urban development is a reduction in surface water filtering down into groundwater, increasing the possibility of aquifer and spring depletion.*
>
> *A well designed, maintained and naturalised stormwater network will replicate the natural environment, protect and improve water quality and quantity, manage flood risk, and maintain and improve natural habitats. A naturalised stormwater network includes use of a variety of stormwater mitigation facilities, including soil adsorption, sedimentation and detention basins, wet ponds, swales and wetlands, connected across the catchments and incorporating esplanade margins. This approach is distinct from the more traditional utility approach of pipes, concrete channels, boxed drains and pumping stations."*

One of the goals to achieve the vision of the SWAP is to *provide a high-quality naturalised water environment, connected across the South-West.* In order to achieve this goal a number of objectives have been set out, including:

> *Objective 1.3: Site stormwater mitigation facilities to avoid interference with public water supply wells and unmanaged or contaminated fill sites.*
>
> *Objective 1.4: Use detention basins and soakage to ground to reduce flood risk and manage downstream flows during flood events.*
>
> *Objective 1.7: Maximise soakage to ground opportunities and pervious surfaces in new urban developments, including the road network, to increase groundwater recharge.*
>
> *Objective 1.8: Develop the naturalised stormwater network using large consolidated stormwater mitigation facilities, rather than a proliferation of smaller facilities.*

Further to the objectives listed above the "water environment" and "ecology" are set out as key issues within the Plan. Extracted from the Plan are the relevant sections with regards to these topics:

> ***Water Environment***
>
> *Improving water quality and managing flooding are central to the sustainable management of the Heathcote and Halswell River catchments. Development and intensification of land use from rural to urban affects surface water quality and quantity. Impervious (sealed) surfaces and channelling of water that would otherwise pond increase the rate of stormwater runoff into drains and rivers. Contaminants in the water change from those produced by rural activities (such as nitrates) to those from urban activities (such as heavy metals). Development disturbs the soil and increases soil erosion into waterways. These effects can increase the risk of flooding, reduce the natural values of waterways and pose threats to human health, for example from swimming.*
>
> *Much of the aquifer providing Christchurch's untreated drinking supply flows under the South-West. For the most part, a confining layer of soils and natural upward pressure prevents contaminants from leaching into the groundwater. Generally, the intensification of urban development is not considered a significant risk to groundwater quality. Some areas are more vulnerable to contamination, where the groundwater is near the surface and not as confined. Restricting high-risk activities, such as industrial development, and ensuring the on-going close management of land- use activities are necessary in vulnerable areas.*
>
> ***Ecology***

> *The majority of habitats in the South-West are highly modified and show evidence of degradation by existing land uses. Indigenous vegetation is fragmented and reduced to remnant patches. Preserving the remaining indigenous areas is necessary to initiate the process of habitat restoration. Maintaining and enhancing habitats along migration routes (including between Lake Ellesmere/ Te Waihora and the Estuary/Ihutai, from the Port Hills to the Canterbury Plains, Heathcote River/Ōpawaho and Halswell River/Hurutini) is critical to improving diversity and populations of indigenous species. Maintaining waterways in a natural state and controlling the discharge of stormwater run-off is also fundamental to sustaining and restoring aquatic biodiversity. Habitats need to be adequately sized, spaced and connected and, ideally, built on existing natural features and remnant patches of indigenous vegetation. Habitat restoration is a long-term goal, requiring on-going management, including weed management, and the participation and encouragement of developers and residents.*

SWAP Phase 1 report, Assessment of Natural Values(CCC, February 2008) supporting the SWAP describes the potential groundwater effects from the SWAP. Extracts from that report are included below.

> *[There are] a number of potential groundwater issues relating to urbanisation in the South-West area. Generally the conversion of land from agricultural to urban uses, and the resulting increase in impervious surfaces, would reduce aquifer recharge from rainfall. However, urbanisation could theoretically cause some positive changes – locally derived stormwater could infiltrate fully into the groundwater system and with reduced evapotranspiration the amount of aquifer recharge could increase. Furthermore, the amount of nitrogen from rural land-uses seeping into the groundwater could reduce, and higher levels of infiltration could increase the dilution of nitrogen concentrations in groundwater received from up-gradient.*
>
> *There is increasing pressure on groundwater resources throughout the Canterbury Plains. It is important that the City Council monitors the effects of those activities within the Plains which use groundwater. Such activities have the potential to impact on springs (by altering groundwater flows) and ultimately natural values of waterways within the study area.*

Overall, the vision of the SWAP has been adopted in the design of CSM2 and this has been facilitated through discussion with planning officers and engineers of the CCC. The layout of proposed stormwater infrastructure in the SWAP has been considered in developing the preliminary design of the CSM2 drainage system.

7.3 Resource Management Act 1991

Section 15(1)(b) of the RMA states that for the discharge of contaminants into the environment:

"No person may discharge any

(a) Contaminant or water into water

(b) Contaminant onto or into land in circumstances that may result in that contaminant (or any other contaminant emanating as a result of natural process from that contaminant) entering water

...unless the discharge is expressly allowed by a national environmental standard or other regulations, a rule in a regional plan as well as a rule in a proposed regional plan for the same region (if there is one) or a resource consent."

The Project involves discharges of stormwater into land and therefore must be authorised by a regional plan or resource consent.

7.3.1 SWAP – Network Discharge Consent

Following the Council adoption of the SWAP in April 2009, a network Discharge Consent was applied for to allow the elements of that plan to take effect.

The consent application was notified and a Resource Consent hearing held in March 2012. Following the release of the decision, no appeals were received and the consent has taken effect.

7.4 Natural Resources Regional Plan (NRRP)

The NRRP became fully operative on 11 July 2011.

The MSRFL and CSM2 corridor is located in an area identified within ECan planning maps as containing unconfined and/or semi-confined groundwater. The Project from Halswell Junction Road to Marshs Road is in the Christchurch Groundwater Protection Zone and an area identified with a groundwater depth of less than 6 metres.

7.4.1 Rule WQL6

The primary Rule relevant to stormwater discharges in the Water Quality (WQL) Chapter of the NRRP is Rule WQL6 as follows – *"Discharge of stormwater onto or into land where contaminants may enter groundwater."*

Rule WQL6 applies to both stormwater discharge during construction and stormwater discharge for the operation of the completed road. There are a number of locations where dispersed discharge to land will occur, being:

- Infiltration through the base of the swales

- The highway drainage soak pits
- The highway drainage treatment devices (used in areas where groundwater is less than 6 metres depth (as indicated on the planning maps))
- The ponds located at Robinsons Road and Halswell Junction Road
- The overland flow soak pits intended to discharge flows which cannot be passed beneath the Project (e.g. areas within interchanges or overland flows upstream of the Shands Road interchange) both within and outside of the area mapped with groundwater depths greater than 6 m
- The base of the overland flow siphons
- The base of the secondary siphons at stockwater race crossings.

In addition there are a number of specific discharge points, which includes the discharge to surface water in limited situations. These are shown on plan Figure 18 in **Appendix A** and described as follows:

- Weedons Road cloverleaf interchange. Two soakage basins will be located within the middle of the clover leaf. There will be a piped discharge into each soakage basin from Weedons Road to take embankment runoff. However in the very unlikely event that the soakage becomes full these pipes could act in reverse and discharge or spill to the Weedons stockwater race.

- Robinsons Road. Two point discharges are proposed. Discharge 1 is from the primary pump system to the surface water discharge point on Main South Road and shown on drawing 6223-B-C401. The second discharge (to ground) is located 300m further down Robinsons Road. This will occur to a trench to be excavated within the Robinsons Road reserve from 300 m to 500 m away from Main South Road.

- On Marshs Road. The new embankment is to be constructed where Marshs crosses the CSM2 alignment. There is an existing stockwater race which is to be relocated adjacent to the toe of the new embankment. However the designation of the stockwater race will change to land drainage race. The runoff from the new embankment will discharge to the land drainage race.

- At Halswell Junction Road. The Maize Maze and Ramp Ponds and CCC's Owaka Basin all will have an overflow discharge to Montgomery's Drain. There will also be the ability to turn on a valve and provide an ability to drain the pond (once full and following rain) in order to lower the pond level and make room available for a subsequent rain event.

Rule WQL6 permits the discharge of stormwater from a road or other source onto or into land where contaminants may enter groundwater provided that certain conditions are met as detailed below:

Conditions

1. Discharge from any source

- *(a) The discharge shall not cause stormwater from up to and including a 24 hour duration 2% exceedance probability rainfall event to enter any other property beyond the boundary of the property or area in which the discharge occurs, unless written authorisation from the affected landowner is obtained;*
- *(b) The discharge shall not result in the ponding of stormwater on the ground for more than 48 hours;*
- *(c) The discharge shall not cause erosion of soil;*
- *(d) The discharge system shall be located at least one metre above the highest groundwater level that can be reasonably inferred for the site at or about the time the system is constructed; and*
- *(e) The discharge shall not be onto or from a property that has been registered by the Environment Canterbury on its Listed Land Use Register as a site that is; 'not investigated', 'below guideline values for', 'managed for', 'partially investigated', 'significant adverse environmental effects' or 'contaminated for'.*

2. Discharge solely from a roof

(not applicable)

3. Discharge from any source other than a roof

- *(a) The discharge shall not be within a Community Drinking Water Supply Protection Zone for a well listed in Schedule WQL2 if:*
  - *(i) the discharge was not lawfully established before the date this rule became operative; and*
  - *(ii) the discharge is from that part of a road, including a State Highway that has four lanes for motor vehicles.*
- *(b) The discharge shall not be from a property where:*
  - *(i) an activity or industry specified in Schedule WQL9 is occurring; or*
  - *(ii) the quantity of hazardous substances stored or handled exceeds the thresholds in Schedule WQL9; and the hazardous substances may become entrained in stormwater.*
- *(c) A discharge that is:*
  - *(i) solely from a sealed road; or*
  - *(ii) from a combination of sources; and is located in an area where the depth to unconfined or semi-confined groundwater is less than six metres as indicated in Map Volume - Part 2 Indicative Maps, shall either be via a fully vegetated soil treatment system with the following characteristics:*

(1) a minimum depth of 200 millimetres of soil, and

(2) an infiltration rate between 20 and 50 millimetres per hour, and

(3) at least 5 per cent clay content in the soil, and

(4) be designed to capture and infiltrate all contributing stormwater for rainfall events up to and including a 24 hour duration ten per cent annual exceedance probability; or via an alternative stormwater treatment system that is certified in writing by a suitably qualified and competent person as providing at least equivalent stormwater treatment. A copy of that certification, design plans for the system and appropriate technical documentation that demonstrates the technical basis for the certification shall be provided to the Environment.

(d) Unless the discharge from a combination of sources was lawfully established before the date this rule became operative, or the discharge is into a stormwater collection system for an authorised stormwater discharge, the discharge shall not be from an area of disturbed land of greater than:

(i) 1000 square metres within Zone BP in Map Volume – Part 1 Planning Maps, or

(ii) two hectares in any other location.

7.4.2 Compliance with Rule WQL6

In general, the Project will conform to the requirements of Rule WQL6. Compliance with each condition is discussed in **Appendix H**.

7.4.3 Rule WQL2, 7, 31, 36, 37 and 48

A tabulation of all the relevant NRRP rules is provided below with a brief commentary of the Project's adherence to each rule is set out in **Appendix I**.

The stormwater design, location and management measures outlined in the Stormwater Management and Disposal Options Report (GHD, May 2012) have been developed in consultation with ECan and are considered appropriate in terms of the RMA and NRRP requirements.

7.5 Proposed Land and Water Regional Plan (PLWRP)

The Proposed Land and Water Regional Plan (PLWRP) was publically notified 11 August 2012. Submissions close on 5 October 2012. This proposed plan will eventually replace Chapters 4 to 8 of the NRRP relating to land and water resources, and embeds throughout the Plan the provisions currently found in Chapter 2 relating to Ngai Tahu and the management of natural resources. At this stage, while the rules have effect from the notification date, the PLWRP can only be afforded limited weight as it has not progressed through the public submission process.

The stormwater discharge rules in the PLWRP differ from the NRRP provisions in that they are not prescriptive. The discharge of stormwater to land (as per Rule WQL6 of the NRRP) is addressed via Rule 5.71 of the PLWRP:

> *Rule 5.71 The discharge of stormwater from a community or network utility operator stormwater system onto or into land or into or onto land where a contaminant may enter water or into groundwater or a surface water body is a restricted discretionary activity.*

Under the PLWRP the activity will be restricted discretionary under Rule 5.71.

A tabulation of all the relevant PLWRP rules is provided in Section 6 of the AEE (Volume 2 of the consent documents).

7.6 Summary of Resource Consent Requirements

7.6.1 A Land Use Consent

Land use consent will be required for a non-complying activity under Rule WQL36 for the excavation over an unconfined or semi-confined aquifer (for the construction of the Robinson Road Underpass and the Maize Maze Pond). This is considered to be a discretionary activity under Rule 5.156 of the PLWRP.

Land use consent will be required for a discretionary activity under Rule WLQ37 for any deposition of fill associated with the above activities. This is considered to be a discretionary activity in accordance with Rule 5.161 of the PLWRP.

Land use consent will be required for a restricted discretionary activity under Rule WQL31 for the construction of the bore / infiltration facility related to Robinsons Rd underpass and for the relocation of domestic and stockwater bores. This is considered to be a permitted activity under Rules 5.78 and 5.79 of the PLWRP.

7.6.1 B Discharge Permit

Discharge permit will be required for a discretionary activity under Rule WQL6 for the discharge of stormwater to land during construction and operation of the Project. This is considered to be a restricted discretionary activity under Rule 5.71 of the PLWRP.

Discharge permit will be required for a discretionary activity under Rule WQL48 for the discharge of site de-watering where this takes place during construction and operation and is discharged to water (Montgomery's Drain). This is considered to be a permitted activity under Rule 5.92 and 5.77 of the PLWRP.

Discharge permit will be required for a discretionary activity under Rule WQL48 for the discharges to surface water. This is considered to be a restricted discretionary activity under Rule 5.71 of the PLWRP.

7.6.1 C Variation to CMS1 consents

It is noted that there may be a requirement to apply for a change of consent conditions applicable to the consents issued by ECan for the CSM1 project. This will be reviewed and appropriate applications submitted to ECan, if required. The CSM1 stormwater requirements have been accommodated in the design process for this Project.

8 Assessment of Effects

8.1 Uncertainties in Design

8.1.1 Uncertainties in rainfall and rainfall patterns

There is a degree of variability in the rainfall and rainfall patterns expected during the design life of the Project. In order to have some degree of certainty to establish a design value, the Project has adopted rainfall rates used by Christchurch City and Selwyn District Councils.

The return period of 100 year ARI event has been adopted for the Project as specified by the NZTA (NZTA, 2010). Other guidance comes from the Building Act where a 50 year level of protection for habitable floor levels is used. The Project adoption of a 100 year design life on this Project exceeds this value.

In a Project such as this, there will be events of greater magnitude and within the next 100 years the chance of an event greater than the 100 year ARI design event is approximately 63%.

8.1.2 Climate Change

In order to take account of the predicted effects from climate change the Project has used the values of increased rainfall rates as recommended by the MfE, 2008 document "Preparing for Climate Change, A guide for local government".

By adopting this advice, the effect of increased rainfall peaks will be mitigated for and are inherent in the design of the stormwater infrastructure for the Project.

It is noted that in the adopted rainfall patterns as recently updated by CCC (December 2011) that the revised Climate Change predictions have been incorporated into the latest revision.

8.1.3 Run-off rates

The Project has adopted the SCS method to determine the peak runoff rates as set out in Chapter 4 above. The project has adopted the CCC – WWDG procedure for determining the volumetric runoff values used in calculation for water quality devices.

Field testing has been undertaken to test the percolation rates at design disposal depths. This is not directly comparable to infiltration through surface soils. A discussion on these figures is provided below in Section 8.6.1. There have been no surface infiltration field tests undertaken for the Project. A range of surface soil infiltration rates can be expected along the Project. Where the design rainfall rates exceed the infiltration rates, higher runoff rates could be experienced.

However, where surface percolation rates are high and exceed the rainfall rate, there would be no runoff from the design storm.

The geometric spread and extent of low percolation areas is unknown but is expected to occur at limited sections of the catchment adjacent to the motorway/highway. This will become critical for the Project where low percolation rates could cause larger overland flows to the Project corridor for which the assumed design flow rates are exceeded.

If the design rainfall were to exceed the percolation rate then large quantities of runoff could be expected from catchment areas upstream of the Project. These flows would arrive upstream of the Project and have the potential to overtop the bunding and fill the upstream swales. Conveyance mechanisms have been designed to convey this flow and discharge it downstream of the Project.

In order to mitigate for rainfall and its associated runoff exceeding the adopted design values, two measures are recommended:

- Field trials and risk assessment are recommended along the route during the detailed design process to confirm the range for infiltration rates used in the preliminary design (a conservative runoff coefficient of 0.35 and CN74, where appropriate, have been used in the development of the preliminary design).
- The detailed design process is used to allow for collection of upstream flow, passage of that flow under the Project and an assessment of the effects of discharge of concentrated flow from the downstream end of the conduit. The design shall minimise the degree of upstream ponding and mitigate for potential effects from concentrated discharge downstream of the site. For this, the design should adhere to the criteria set out in Section 4.4, which are:
  - An investigation into the upstream effects is made in conjunction with the design of siphons under the Project alignment
  - A design process is undertaken to avoid any increase in upstream habitable floor level flooding in events up to the 50 year ARI event (i.e. zero afflux)
  - A design process is undertaken to avoid any increase more than 250 mm in flooding depth for events up to the 100 year ARI event (i.e. max afflux level of 250 mm)
  - An investigation of the downstream effects is made as a consequence of concentrating flow to a point discharge
  - A design process is undertaken to avoid any increase in downstream habitable floor level flooding in events up to the 50 year ARI event.

8.2 Effect of Groundwater on the Project

Groundwater levels have the potential to affect the Project at two locations. These are at Robinsons Road and to the stormwater disposal system adjacent to Halswell Junction Road. For the balance of the Project the depth to groundwater is generally sufficient to have no impact upon the construction or the operation of the Project once construction has been completed.

At the southern end of the project the depth to groundwater is generally between 12 and 20 m. however at the northern end of the project the depth from existing ground to groundwater is much less and historical highs come to within 3 m of ground surface.

In addition to local and seasonal variations, CPW has now been consented and is part of the planning landscape. As a result of CPW there is groundwater mounding predicted at 4 m at the southern end reducing to around 1 m at the northern end of the project adjacent to CSM1.

At the southern end of the Project the chance of encountering high groundwater is low. Adverse effects on the MSRFL groundwater disposal system due to changes in groundwater level resulting from CPW are avoided by the elevation of the disposal system, with the exception of the Robinsons Road Overpass.

Proposed mitigation measures and groundwater monitoring measures at the Robinsons Road site include pumping of surface water and of groundwater. Construction of some of the future ‘below ground infrastructure’ is proposed permitting the installation of the electrical and mechanical equipment at a later date when the effects of CPW are felt.

At the northern end of the Project the groundwater elevation is much closer to the Project alignment. The groundwater historic and predicted highs show that without CPW the groundwater will be within 3 m of current surface levels. With the CPW Project there is a conservative estimate of 1 m of predicted mounding. As such the predicted future groundwater level will be within 2 - 3 m of current surface levels and above the Project pond invert levels.

The effect of the future ground water highs will be that without intervention, the high groundwater would flow back into the pond and take up storage volume designed for water quality treatment. As such an intervention strategy is proposed to maintain future groundwater rises to at or below the pond base level.

In order to mitigate the potential for elevated groundwater, a number of actions have been considered. These include a further review of the groundwater monitoring and re-evaluation of the design high groundwater prediction.

A further area of mitigation is to consider the effects of high groundwater on the disposal rates and groundwater disposal mechanism at the north end of the Project. To this end the design has considered the following:

- The Maize Maze Pond, adjacent to Halswell Junction Road, has been designed to treat and dispose of stormwater and has an invert level of RL = 18.75 m. The Ramp Pond invert levels are proposed at RL = 18.5 m. Both pond invert levels are below the predicted groundwater high level (RL = 19.4 m).

- Both ponds have a proposed liner over the forebay floor to prevent / limit flows from the first flush/forebay to groundwater.
- The ponds have second and third bunded areas. Discharge from the ponds to land is limited to the latter bunded area and to the downstream disposal field.
- In order to mitigate for the potential of high groundwater in conjunction with heavy rainfall and associated runoff when the pond will fill (as designed), a programme of controlled emptying is to be employed. This system includes controlled discharges to the Upper Knights Stream. Discussions have been held with ECan and the CCC catchment/network managers of the Halswell catchment in relation to the proposed "emptying rate" of up to 60 L/s.
- Also and in order to prevent inundation of groundwater back into the empty pond from high groundwater levels, an under drainage system under the critical ponds is proposed to prevent uplift under the lined sections. The preliminary design of the under drainage system includes shallow drainage wells and/or infiltration trenches, gravity trunk drainage collection system and a discharge pipe downstream where the system would discharge to the Upper Knights Stream some 500 m downstream from the Project. This system is shown on drawing 62236-B-C425 (refer to Volume 5 for drainage drawings)
- Proposed consent conditions (see Table 10 of Section 9.1) include that when the pond owner/maintenance contractor wishes to draw down the pond level in order to make room for a subsequent rain/runoff event in the pond, the owners representative shall liaise with the ECan – 'Halswell River and Catchment Manager' and with the CCC, 'Surface Water Operations Manager' as to the proposed discharge rate and proposed timing of discharge
- Both the Maize Maze Pond and the Ramp Pond volumes have been designed to include total storm detention for a 24 hour design storm and for the pond level to be contained without effect on the associated CSM2 carriageway. In addition, these ponds accommodate the 60 hour event with low levels of discharge, late in the storm event. The cumulative effects of these discharges are less than minor, as set out in **Appendix D**.

8.3 Effects on Surface Water Bodies

8.3.1 Upper Knights Stream and Halswell River

The Halswell River has a history of flooding. The SWAP (CCC, 2009) and its associated SMP (CCC, 2011) have considered limiting the effects of flooding through a series of stormwater storage facilities. These include ponding and detention basins and a recommendation to encourage discharge to land.

There is a predicted effect of rising groundwater levels of the base flow in Upper Knights Drain and Halswell River from the effects of groundwater rise from the effects of CPW. Without intervention these increases will occur as part of the existing planning environment and not as a result of the Project.

However as a result of the proposed works to artificially limit the effects of ground water rise and in particular under the Project stormwater ponds, there will be groundwater draining into the Upper Knights Stream downstream of the Project.

With or without intervention a new equilibrium would be found. With the proposed pond under drainage system flow rates would be concentrated at the piped outlet, whereas without, the flow rates would be more spread along the Upper Knights Stream. If the flow rate out of the piped system can be controlled to an extent where there is no significant variation in flow following rain then the net effect increase in flow is likely to be small and the effect on the Upper Knights Stream similar.

The positive effect of having future high groundwater levels artificially controlled will be to have some surplus capacity in the soil matrix above the groundwater to temporarily hold water until the rain event has passed and the under drainage system can artificially lower the groundwater table again.

Once the significant rain event has passed and if the stormwater ponds remain full, it is recommended that the pond be artificially drawn down via a manually operated sluice gate to make storage available for a subsequent storm event.

In order to manage the effects on the drainage system, a period of monitoring of the discretionary discharge from the stormwater pond under controlled conditions is recommended. A process for the controlled release of water from the Maize Maze Pond to the Halswell River system is to include discussion with the ECan River Manager and the CCC Surface Water Operations Manager to consider the proposed release rate from the ponds, and to determine a rate that is unlikely to significantly affect the flood levels downstream in the Halswell River. This is included in Table 10 below.

Another effect of the Project on these water bodies is that the new highway impervious area will increase and as such there will be more runoff volume than would otherwise be soaking directly to land (ignoring the effects of evaporation and evapotranspiration). We consider there will be a slight increase in total volume to land but a negligible change during and immediately after a large storm event (by taking into consideration the time to soak away following that storm event).

8.3.2 Stockwater Races

Stockwater races form a dual function of providing water for irrigation and stock but also provide a secondary function of providing for land drainage.

From the Project perspective there will be a range of short and long term effects.

From a short term perspective there will be some stockwater race closures. It will be necessary during detailed design to ensure there is a management plan for each stockwater race to ensure all effects are investigated and where practical mitigation measures put in place to ensure the effects of any changes are managed in a measured way.

From a long term perspective the stockwater race will lose access to surface flow for the width of the Project and be transported in an inverted siphon.

The secondary function of the stockwater race is to provide land drainage. Overland flows in excess of the notional full capacity of the race have the potential to arrive upstream of the Project alignment. The extent of development immediately adjacent to the proposed alignment is currently limited therefore effects on flooding of habitable floors are likely to be less than minor. However, the current extent and frequency of inundation of pasture upstream of the alignment is not known.

On the downstream side of the Project alignment the siphons will discharge stormwater. This is also aided by distance between the Project alignment and the downstream properties. Natural dispersion of flows is likely to occur in the distance between the siphon outlets and the downstream properties.

There are design and operational constraints associated with the passage of overland flows in siphons and the following points set out the potential effects and the proposed mitigation.

Potential flooding effects from flows arriving at the upstream boundary of the Project and not all of this flow being able to be passed under the Project. This is managed by design and on-going maintenance of the inverted siphons.

Potential for blockage or partial blockage of the siphons can be managed by:

- Raising the upstream inlet above the immediate adjacent ground in order to allow settlement of solid particles and gravels from entering the siphon
- Installing scruffy dome type devices to limit larger floatables and branches from entering siphon
- Oversizing the capacity of the siphon in order to cater for limited over design events
- Attending to good engineering practice on the downstream end of the siphon to ensure effects of concentrated flow discharge are mitigated on a case by case basis
- Ensuring there is an adequate and functioning maintenance programme.

8.4 Effects on Fish Passage

The overland flow paths identified along the route are dry and as such do not have an aquatic environment.

The stockwater races are man-made and as such the races are not a natural waterway. Flow in the races is not permanent as the SDC will from time to time disconnect flow from the race for maintenance purposes. In this situation, it is expected that fish will remain in other sections and then later repopulate.

The proposed siphons do not have any physical barriers which would prevent fish passage along the water races. However, they will be typically over 100 m in length and will have velocities higher than the upstream and downstream races which may form an impediment to weaker species. It is expected that good swimming species will be able to traverse the inverted siphons from one side of the Project to the other as such the effects of the proposed Project should be minor. Further discussion on the effect of the siphons on fish passage is contained throughout Technical Report No. 17.

The one location where fish passage is unlikely to be maintained is the 2.1 km of piped stockwater race adjacent to MSRFL. An alternative route to the upstream network is available along Weedons Ross Road. No physical barrier is proposed to prevent fish accessing the race downstream of the piped race

8.5 Overland Flows and Flooding Effects

The effects of overland flows for stockwater races and overland flow paths have been addressed above.

The remaining issue is the overland flow and exceedance events at Halswell Junction Road.

8.5.1 Halswell Junction Road Pond

The Halswell Junction Road Pond has been recently upgraded and enlarged as part of the CSM1 works. Currently the pond overflows to Montgomery's Drain.

As part of the SWAP and its associated network Discharge Consent, the overflow from Halswell Junction Road Pond has been planned to divert this flow to the Owaka Basin. This diversion work has been requested by CCC for the NZTA to incorporate into the CSM2 Project.

8.5.2 Owaka Basin

The Owaka Basin has now been substantially constructed by Fulton Hogan under a contract with CCC.

The Owaka Basin has been designed to accommodate overflows from Halswell Road detention basin (which collects flows from the Hornby Industrial Area with an approximate catchment area of 100 ha). The CCC designed primary outlet from the Owaka Basin is to the old quarry pit on Wilmers Road. However, when this pit is full or there is insufficient hydraulic gradient, an overflow discharge from the Owaka Basin will discharge back to Montgomery's Drain and on to Upper Knights Stream. It will be necessary to maintain the connectivity and capacity of this overflow through the construction sequence of CSM2.

As such a specific condition requiring maintenance of flows in and out of the Owaka Basin through the CSM2 construction phase is recommended.

8.5.3 Mushroom Pond

The Mushroom Pond facility has been designed as part of CSM1 Project (without consideration of the effects of CPW). This pond is now constructed and is operational. As part of this Project, the eastern first flush basin will be in-filled as a result of the construction of the CSM2 north bound on ramp. The main soakage pond's volume will be lost by approximately 5% of its original storage volume.

The invert of the Mushroom Pond is higher than that proposed for the Maize Maze and Ramp Ponds. The Mushroom Pond will not be inundated by future groundwater level rises, however the discharge rate to ground is expected to reduce as groundwater level rises leading to longer periods before emptying of the ponds.

In parallel with this approximately 30% of the previous carriageway catchment area to the Mushroom Pond will be directed south east to the proposed Maize Maze Pond. The balance of the Mushroom Pond will be:

a) Appropriately sized to treat the reduced carriageway area

b) Cater for the attenuation and treatment of those flows.

Currently the Mushroom Pond has an overflow spill to Wilmers Quarry Pit which is also used for stormwater storage and disposal. The results of recent surface water modelling predict no spill in events up to the 100 year ARI event. However if the Halswell Junction Pond spillway were to operate then the overland flow would potentially overflow into the Mushroom Pond.

A specific condition requiring the maintenance of flows in and out of the Mushroom Pond is recommended.

8.5.4 Maize Maze Pond

The last issue is the discharge from the proposed Maize Maze Ponds. For events up to the 100 year ARI 24 hour event, the pond has been designed to capture and store such that there is no overflow. This includes existing discharges from impervious areas that currently discharge to the Mushroom Pond that are to be diverted south.

However for events exceeding the 100 year ARI event and for longer duration events (i.e. greater than 24 hour storms but potentially more frequent than the 100 year ARI event), there is a potential that the pond will fill and overflow to Montgomery's Drain. Once CPW is implemented and combined with extended periods of high groundwater there may be some water in the ponds (either from groundwater or from previous rain events). In this scenario there is potential for water in the pond prior to the storm to reduce storage capacity. As a consequence, spill to Montgomery's Drain would occur more frequently. The intervention works set out in Section 8.3.1 above that include a pond under drainage system, means that the risk of pond capacity being exceeded is appreciably mitigated.

Thus there is a potential effect on the downstream receiving environment from the Project, namely, occasional spills from the pond within the critical 60 hour storm in the Halswell River.

The effects of the Project on the flows to the Montgomery's Drain, Upper Knights Drain and the Halswell River are less than minor as the intervention measures, i.e. the ability to artificially lower future groundwater highs, will reduce the risk of larger flows in the Halswell River and enable these to be appropriately managed.

8.6 Disposal and Groundwater Effects

8.6.1 Design Disposal

Disposal to land has been assumed throughout the design for the Project.

There has been limited field work to test the soakage rates, however, this field work has not confirmed (with certainty) the rates necessary for disposal. The measured rates vary from 4 mm to 2000 mm/hr. The design soakage has been varied along the route as set out in the stormwater design report (GHD/Beca, 2011). The uncertainty with this design input will be managed through the balance of the design and construction process. The proposed mitigation recommended includes specific soakage field tests at the detailed design stage with allowance for modification of the design disposal locations, if required.

Given the dispersed nature of the disposal system, the effects arising from failure of any individual soakage device is considered less than minor, as discussed below in Section 9.2. The design permits cleaning, maintenance and easy extension of the soakage devices should it be found that any particular device is not performing either due to blockage/sedimentation of the disposal pipe work/trench or poor percolation rates in the receiving ground. Should multiple disposal locations fail, a cascade effect could occur with the collection of stormwater to the sag point in the alignment at the Maize Maze. Given the unlikely occurrence of the design storm event over the entire alignment, storage should be available in the intervening swales, therefore reducing the chance of a cascade failure resulting in spillage from the system.

The most critical location for the system is at the Maize Maze site. At this particular location a much reduced disposal rate of 12 mm/hr has been assumed in the sizing of the disposal field. This is to allow for elevated groundwater levels coinciding with a design storm event and the cascade scenario described above. The size of the pond has also been designed to help mitigate this scenario.

There is also potential for sedimentation/blockage of the filter media and/or soak pit trenches. Regular maintenance of the soak pits will mitigate the risk of sedimentation. Monitoring of the operation of the system will identify surface water ponding in the swales upstream of the soakage devices and will prompt if further, irregular maintenance is required.

8.6.2 Groundwater mounding

A groundwater modelling study has been carried out and is attached as **Appendix C.** The Project has a low risk of impact from high groundwater prior to the effects of CPW being felt. However once the impacts of CPW are felt then there are two locations where the Project is impacted. These occur at Robinsons Road and at the ponds adjacent to Halswell Junction Road.

At Robinsons Road the groundwater mounding together with the groundwater highs have the potential to rise above the carriageway level for short periods. Mitigation measures are proposed in order to deal with the treatment of surface water runoff by pumping to adjacent disposal fields. A second extraction system is proposed in the event that future groundwater levels rise above pavement levels, which will be discharged to the adjacent stockwater race.

At the Halswell Junction Road ponds the future groundwater mounding from the CPW project and in conjunction with groundwater highs is likely to have a prolonged effect on the ability of the ponds to discharge to ground. Further, the future groundwater highs have the potential to flow back into the CSM2 ponds (Maize Maze and Ramp Ponds) and CCC ponds (Halswell Junction Road, Owaka Basin and Wilmers Quarry Pit). As such mitigation measures have been designed to intercept the groundwater highs and to keep the level of rise below the pond base level. The groundwater interception system has been designed to operate by gravity and discharge down John Paterson Drive and onto the Upper Knights Stream some 500 m from the Halswell Junction Road Ramp intersection roundabout.

8.7 Water Quality Effects

The disposal points proposed for the Project can be divided into two types:

- Road Drainage Disposal, where the catchment is limited to the road corridor (typical contaminant sources include: vehicle emissions, pavement wear, tyre wear, litter, spills and break wear) (NZTA 2010) and where runoff will be treated within the system prior to discharge, and

- Overland Flow Disposal, where the catchments are much larger but mostly rural (typical contaminant sources include: agricultural chemicals and fertilisers, animal faeces and silage leachate) (NZTA 2010) and where runoff will be untreated prior to discharge but will likely occur only in large rainfall events.

Vehicle emissions include volatile solids and hydrocarbons and pollutants that are generated by the everyday passage of vehicles. Tyre wear and vehicle corrosion all contribute, together with substances released from the wear of the paved surface.

The NRRP rules are prescriptive with regards to water quality effects. As such compliance with the rules infers adequate treatment and effects being less than minor. Soakage design on this Project is generally above the water table as per NRRP conditions ensuring that water quality objectives will easily be met for much of the alignment. Where water quality treatment is required first flush basins will be constructed with organic filter media included in the road drainage system prior to disposal. The residual risks of this approach are:

- Inappropriate maintenance of the system leading to reduced percolation rates and flooding
- Contaminant loads being generated in excess of the ability of the organic filter layer to absorb contaminants, or
- Bypass of the organic filter layer by inappropriate maintenance or accident.

Overall the treatment proposed is beyond that sought in the NRRP and is considered best practice. Notwithstanding the residual risks outlined above, the effects of the quality of road runoff are considered to be minor.

8.8 Stormwater Treatment

Stormwater treatment is to be provided for all carriageway runoff by way of swale or pond treatment before soakage to land.

As a minimum NRRP rules require stormwater treatment where discharge to ground is within the 'less than 6 m to groundwater' zone and this has been allowed for in the design. Outside of this zone, swale treatment is generally provided prior to discharge to ground via soak pits.

There is still a remote possibility of contamination of groundwater from larger liquid spills but general wear and minor spills can be readily catered for by swale treatment. Within the 'less than 6 m to groundwater' zone there is a level of redundancy in the system with a stormwater treatment train approach (i.e. swale, first flush basin, filtration media then discharge to soakage) and as such risk of groundwater contamination is considered to be low.

The engineered swale is an improvement on the existing table drain ditches either side of the existing road network and as such there will be an expected improvement in the stormwater runoff quality.

8.9 Effects on Adjacent water takes

Based on information supplied by the ECan GIS and data team in October 2010 a list of affected water supply wells within 1 km of the Project alignment has been compiled which are either beneath the Project footprint or within close proximity to the outside edge of the stormwater disposal system.

An updated list of potentially affected wells (i.e. those wells within the project footprint or within 100 m horizontal separation and 10 m vertical separation) has been completed in August 2012. This list is attached as **Appendix E.**

Those wells within the Project footprint are to be closed and capped. However the consent attached to a number of these wells are to be transferred to new wells to be sunk outside the Project footprint.

It is proposed to analyse all such wells within a 100 m buffer of the Project footprint. Where the drawdown effect of any such well or whether the well has the longer term potential to be impacted by discharges from the Project, then it is proposed to close the potentially affected wells and arrange for new wells to be sunk, developed, tested and connections made to a new power source.

Closing of these wells is proposed to minimise any potential effects from a large oil/chemical spill entering the groundwater system via a disposal point. Moving the location of some stormwater disposal points may also be possible during detailed design. The precise location of new wells, if required, will need to be identified in the detailed design phase.

8.10 Effects of Road Runoff

Attached as **Appendix F** is a report outlining the Contaminant Load Assessment. This report addresses the contaminant load generated by vehicles based upon relevant literature and the likely and expected treatment within the surface water swale treatment systems.

Based upon the findings of this report, the conclusions state that the effect of discharges to groundwater can be very complex. However the swale is expected to provide a high level of treatment from Total Suspended Solids and a good level of treatment for both particulate and soluble fraction of metals to the environment from the expected contaminant load from the road. Overall it is concluded that the contamination issues will be less than minor from this project.

Attached as **Appendix G** is a report outlining the Assessment of Groundwater Quality Effects. This report addresses the modelling undertaken to determine the risk of potential contamination in relation to drinking water standards and whether any remedial actions are required. The estimated concentrations of copper and zinc in stormwater are less than their NZ Drinking Water Standard values. Therefore, copper and zinc in stormwater discharged from the proposed alignment pose low risk to groundwater used for potable supply.

Risk assessment of pyrene and fluoranthene has indicated that when dilution in groundwater beneath the alignment and attenuation along the groundwater flow path is considered, these contaminants pose low risk to groundwater used for potable supply. This is valid for wells that are located 30m or more from the designation boundary. According to information supplied by Environment Canterbury, there are 17 wells within 30m of the designation boundary that may be affected by stormwater discharge.

9 Mitigation

This section details the mitigation of adverse environmental effects already included in the design and measures to be undertaken in later stages of the design process. Overall there are a number of aspects of the design philosophy which have been implemented to mitigate environmental effects including: the design standard applied, the dispersed drainage and disposal system, overland flow siphons and stockwater race conveyance pipes. The overall design philosophy is described above in Section 4. Areas where mitigation may be appropriately confirmed in consent conditions are indicated below.

9.1 Mitigation through potential Resource Consent conditions

A number of issues have been identified in preparing this assessment. Areas where consent conditions may be appropriate to mitigate the potential effects are listed below in Table 10.

Table 10 Mitigation through Conditions

Aspect	Commentary	Recommended mitigation
Soak Pits	The soak pits form an essential element for the disposal of stormwater along the route. The on-going operation of the soakage pits is an essential element in the design as there is no alternative disposal mechanism. The design is to achieve an adequate level of redundancy to ensure that progressive failure of individual elements in the Project design do not affect the users of the road system or cause negative off corridor effects such as additional surface flooding in the Halswell catchment.	Development of field testing programme to confirm soakage rates of receiving ground Drafting an Operation and Maintenance Plan during detailed design for soakage devices
Stormwater Treatment	The first flush basins rely on organic filter media to achieve the water quality objectives. These devices have the potential to concentrate contaminants and sediments. In order to ensure that they perform adequately a monitoring program is proposed.	• Specific soil parameters for first flush filter media • Monitoring of soil contamination at disposal sites • Replacement of soakage filtration media when contaminated • Monitoring of percolation rates

Aspect	Commentary	Recommended mitigation
		through soil media to ensure these are similar to design rates
Stockwater Races	The stockwater races form two distinct functions. a) as a conveyance mechanism for stockwater and irrigation and b) as a land drainage function during extreme weather conditions. The on-going operation of the stockwater races is an essential element in the Project design. The design is to achieve an adequate level of redundancy to ensure that individual elements in the Project design does not affect the stockwater race function as set out above	• Ensuring the ongoing supply of water from stockwater races during and post construction (as far as practicable) • Allowing for the passage of flood and land drainage function of the races • That the construction of deviations to be completed off line before the new deviation is made live • Limiting the time and occurrence of over pumping to emergency and limited period occasions (e.g. tie ins)
Overland Flow Paths	The overland flow forms an essential element for the passage of stormwater across the route. The on-going operation of the overland design is an essential element in the design as there is no alternative. The design is to achieve an adequate level of redundancy to ensure that progressive failure of individual elements in the Project design does not affect the users of the road system or cause negative off corridor effects.	• Management of additional flow paths identified following detailed topographical survey and how additional crossing points identified during the detailed design will be accommodated • Adherence to the design criteria outlined in Section 4.4 for designing the overland flow crossing points under the Project alignment including a full assessment of the upstream and downstream flooding and ponding/effects of discharge of concentrated flow on property and habitable floor levels downstream of the Project area
Owaka Basin, Mushroom Pond and Maize Maze Pond	The Maize Maze Pond and its associated disposal to land system form an essential element for the disposal of stormwater adjacent to the CSM1 – CSM2 – Halswell Junction Area. The on-going operation of the soakage to land and protection of groundwater quality is an essential element in the design. The design is to achieve an adequate level of redundancy to ensure	• Inclusion of a liner system that prevents the direct connection of surface water to land in the forebay section of the pond • The design of the pond shall include a) an ability to receive and store the entire 24 hour 100 year storm runoff from the CSM2 Project, b) an ability to draw down the level of the pond level following a large rain event and

Aspect	Commentary	Recommended mitigation
	that progressive failure of individual elements in the soakage system does not affect the users of the Project system or cause negative off corridor effects such as additional surface flooding in the Halswell catchment during events of lesser magnitude than the critical 100 year storm event.	discharge this flow to the Upper Knights Drain or Montgomery's Drain A process for the controlled release of water from the Maize Maze and Ramp Ponds to the Halswell River system after the rain event has ceased (including discussion with the ECan and CCC appropriate River Managers) • Maintenance of the flows in and out of the Mushroom Pond and Owaka Basin during construction and operation of the Project • Development of an Operation and Maintenance Plan for the ponds and disposal system
Robinsons Road	The potential for Robinsons Road overpass to be inundated by groundwater has been identified with the predicted CPW scheme in place. Given the uncertainties with the CPW implementation and effects, proposed conditions are designed to allow the uncertainties to be mitigated with future action.	• On-going monitoring of groundwater levels at the site undertaken to establish the appropriate time for installation of the electrical and mechanical equipment • A continuous monitoring regime is recommended. Recordings should be downloaded at least quarterly and an annual report produced with recommendations as to the ongoing frequency of downloads and recommendations made of pump install date • A high level alarm should be included to allow sufficient time to notify ECan and SDC officers of operation of the pumping system due to high groundwater levels • Development of an Operation and Maintenance Plan for the pumping and disposal system
Erosion and Sediment Control	E&SC form an essential element for the protection of the environment along the route. The on-going operation of the soakage design is an essential	• Development of an Erosion and Sediment Control Plan for each work section along the Project covering a) clean and clear water diversions, b)

Aspect	Commentary	Recommended mitigation
	element in the design as there is no alternative. The design is to achieve an adequate level of redundancy to ensure that progressive failure of individual elements in the Project design does not affect the users of the road system or cause negative off corridor effects such as additional surface flooding in the Halswell catchment.	diversion drains for sediment laden runoff, c) use of permanent swales and the ability to rehabilitate the swale to its final purpose during the construction process, e) specific disposal to land soak pits which are not to form part of the final soak pit system, f) methods to prevent discharge of sediment laden water off site or to land, g) cover the issues addressed in other plans such as overland flow path construction, stockwater race construction, existing bores/wells and the works required at each intersection, h) on-going maintenance requirements, i) disestablishment criteria
Existing Bores and Wells	The Existing bores and Wells form an essential element in supplying water to adjacent properties. This Project has a potential to affect existing bores. These conditions have been inserted for the protection of the access to water from these existing bores and wells.	• A suitable process for drilling, developing and testing of new wells to replace any shallow or close proximity wells to be closed • A suitable process to identify potentially affected wells (i.e. those wells within 10 m vertical and 100 m plan distance of the Project disposal points) and those wells beneath the footprint of the project • A suitable process for capping and/or plugging existing wells to be abandoned

9.2 Mitigation in Current Design

9.2.1 Design standard

The design standard applied in sizing the stormwater infrastructure is a 100 year return period. This is the primary tool used to mitigate the effects of the increased runoff generated by the Project and reduce the residual risks of spilling from the highway drainage system or potential failure of the disposal system. Utilising this design standard ensures that the Project will remain functional in extreme events and that effects on the receiving environment will be limited to rare instances.

The selection of the 100 year design standard limits the ability to depress the Project alignment beneath the existing surface to a great extent. This is due to the increased height of the drainage and disposal system above the calculated design groundwater level. Balancing the noise and visual effects has required the use of kerb and channelling about the sag point adjacent to Halswell Junction Road (which permits a shallower drainage system).

9.2.2 Highway Drainage

The proposed disposal system is dispersed (regular soak pits as opposed to large disposal facilities) so failure of one component will not result in catastrophic failure of the whole system. This provides some inherent redundancy in the system and allows a more passive maintenance programme whereby localised flooding can be used to identify failure in soakage devices (rather than by regular testing). Overtopping over bunds to the downstream soakage device and transfer of flows via the cross drains will occur prior to flooding of the Project carriageway. This will not occur at the sag points, most critically the CSM1 connection (the Maize Maze Pond), as such a more conservative approach has been used, including:

- The application of total storm detention at the Maize Maze and Ramp Ponds in a 24 hour event (providing sufficient storage to cater for all runoff assuming that groundwater levels are inundating the disposal system)
- Lowered percolation rates at the Maize Maze and Ramp Ponds (i.e. a greater safety factor applied to the recorded percolation rates) to allow for the effects of high groundwater on the disposal system.

A number of other key components of the highway drainage design have been implemented to mitigate the effects on the receiving environment, including:

- Pumping has been kept to a minimum to ensure reliability of the system and lowering residual risk. The notable exception is at the Robinsons Road overpass
- The placement of the proposed soakage devices has been to maximise the distance between the devices and any stockwater races or overland flow siphons
- Additional soakage devices and larger soakage areas have been proposed on the upstream side of the Project to facilitate the disposal of any overland flows which may overtop the stormwater bund protecting the highway drainage system.

9.2.3 Stormwater Treatment

Stormwater runoff from the Project pavement is predominately to swales. This runoff will be treated in the soil matrix of the swale invert until the infiltration rate cannot cope with the runoff rate. From here ponding and flow will commence within the swale that has been designed for minimum residence time (in accordance with the NZTA Stormwater Treatment Manual). Further treatment will occur as water ponds in storage at the lower end of each swale section prior to discharge to ground. In the less than 6m to groundwater zone, there will be further treatment in the first flush basins designed for that section of the Project.

The design of the Maize Maze Pond at the CSM1/CSM2 connection is to include separate areas divided by pervious bunds to ensure that water cannot short circuit the pond and the highest quality water is discharged from the pond. Lining of the forebay is proposed to isolate the groundwater from the water detained in the first bunded area (after the first flush basin). This will further reduce effects of the discharge beyond typical treatment standards.

Also proposed is the method specification for laying of the organic filter media in the treatment devices, as discussed in Section 4.6.

9.2.4 Overland flows (MSRFL)

Flooding may occur upstream of the existing SH1 alignment. The existing highway drainage system has not been designed to dispose of the flows generated in the catchment between the State highway and the railway. In order to mitigate the effects of overland flows on the disposal system bunds have been proposed to separate the 'engineered' and 'natural' systems. The effect on the 'natural' system is that the 'engineered' system will occupy flood volume but the effect of this is partially mitigated by a reduction in runoff volume contributing to the 'natural' system (i.e. discharges from the existing highway will be diverted to the disposal system). The effect of the reduction in flood plain volume will be minor.

There are two locations where overland flows may exceed the runoff from the local catchment downstream of the large railway embankment culvert and the Digga-link site. In both these instances specific infrastructure is proposed to mitigate any potential flooding effects by providing conveyance beneath the Project. More specifically:

- A culvert with a high level entry at a level near the existing road crest is proposed downstream of large diameter railway crossing culvert
- Extension and/or replacement of the existing Digga-Link culvert are proposed.

9.2.5 Overland flows (CSM2)

There is significant uncertainty with the occurrence and size of the overland flows generated in the catchments upstream of the Project. In order to mitigate this uncertainty bunds have been included upstream of the Project drainage system. As CSM2 is a greenfield development without any existing restriction to overland flows, siphons have been included to pass flows beneath the Project. Key aspects of their design to mitigate environmental effects are listed below.

- The overland flow siphons have been included in locations where the natural overland flows occur
- Consideration has been given to all topographic data presently available to minimise the effects of any concentration of overland flows on downstream properties
- Increases in flood level upstream of the siphons is intended to be limited to 250 mm in events up to the 50 year ARI event and with no increases in habitable floor level flooding

- The land adjacent to the siphon is slightly dropped to minimise sedimentation of the siphon (reducing the chance of blockage and upstream flooding)
- Soakage at the base of the inlet and outlet manholes has been included to allow the siphon to drain and remain dry between events, thus easing maintenance and reducing flood volumes.

In addition to the siphons the overland flow paths have influenced the highway drainage disposal system. As described above, the disposal points in the highway drainage system have been located and sized with consideration given to overland flow path locations. Further to the additional soakage devices and their location, cross drains have been included in the design to permit two functions:

- Activation of the disposal systems on both sides of the Project
- Facilitate pumping down of the system (using temporary pumps) to downstream overland flow paths after exceedance events.

In locations where overland flow siphons will be impractical (given length or geometric constraints) surface water soakage areas have been proposed.

9.2.6 Stockwater Races

The design of the secondary pipe system at each of the stockwater race crossings will provide sufficient conveyance to pass flood flows. This will mitigate any potential upstream flooding effects. The primary pipe will remain wet at all times allowing fish passage.

9.3 Mitigation Measures during Construction

There are temporary and permanent control measures to minimise the effects of erosion and sediment generated by the Project throughout its lifecycle. Temporary measures are discussed in this section. Permanent measures are discussed throughout the balance of this report and address the long-term measures to control runoff and their associated contaminants.

Erosion and sediment control (E&SC) will be provided throughout the duration of the construction works and through the defects liability period, until site stabilisation has occurred, to ensure protection of the downstream receiving environment from the adverse effects of sediment from the work area.

An Erosion and Sediment Control Plan will be required of the contractor as part of their overall Contractor Social and Environmental Management Plan (CS&EMP). The principle behind a plan is to control erosion across the construction site, to manage any sediment-laden stormwater runoff and prevent unacceptable discharges of sediment into the receiving environment. As much of the receiving environment is to groundwater the receiving environment is also to include protection of the groundwater aquifer.

The Erosion and Sediment Control Guidelines prepared by ECan[9] are proposed to be used as the principal basis for the formation of an Erosion and Sediment Control Plan (E&SCP). The NZTA Erosion and Sediment Control Standard for State Highway Infrastructure (NZTA, 2012) will be used as a second order of preference, or in order to reinforce best industry practice.

All sediment and erosion control measures will be inspected on a regular basis and following any significant rainfall event. The E&SCP will be a "live" document, and will be reviewed and updated where necessary if any measure is not providing its intended purpose.

9.3.1 Description of Controls

The options for stormwater disposal from the Project are limited by the absence of surface water disposal points. Key issues which will require addressing in the E&SCP include:

- Control of stormwater and isolating runoff from the stockwater network
- Separating clean from dirty water
- Protecting adjacent landowners from surface flows
- Minimise sediment leaving the site
- Disposal to land
- Separation of temporary stormwater discharge locations from the permanent stormwater discharge locations to ensure that the permanent locations are not compromised by construction activity.

The minimum proposed E&SC measures are summarised below:

- The contractor prepares and submits an E&SCP as part of their CS&EMP for the Project
- In order to identify the proposed E&SC measures and their associated land requirements a draft E&SCP be prepared during the detailed design phase. This draft plan is to identify each proposed work area, the permanent soak hole locations, the temporary bunding and E&SC ponds and their associated temporary soakage pits. This draft plan should be submitted to the manager ECan for their comment and records
- As part of the construction sequence the contractor will need to submit a revised E&SCP to reflect their specific work methodology. This revised E&SCP is to be submitted to ECan for approval which shall not be unreasonably withheld. Prior to commencement of any earthmoving work on site the contractor must have an approved E&SC plan in place
- An E&SCP is intended to provide the basis for erosion and sediment control relating to the works, however, the contractor may wish to amend the E&SCP depending on their preferred work

[9] *Erosion and Sediment Control Guidelines, Environment Canterbury, 2007*

methodology. Any amendment will be submitted to the ECan manager for information and comment prior to proceeding with the works.

These measures are included in Table 10 of Section 9.1, which describes recommended mitigation through potential consent conditions.

10 Consultation

As noted throughout, personal and telephone discussions regarding stormwater quality and quantity have been held with officers of the two territorial authorities and regional council relevant to the Project and with the Project advisory group. These are listed below.

10.1.1 A ECan

- Richard Purdon – Overall consenting including discharge to land
- Helen Caley – Overall consenting including discharge to land
- Ross Vesey – Halswell river effects
- Tony Oliver – Halswell river effects
- Ian Heslop – Halswell river effects

10.1.1 B CCC

- Paul Dickson – SWAP and overflows to Montgomery's Drain
- Ken Couling – SWAP (particularly Owaka Basin) and overflows to Montgomery's Drain
- Land Purchase Officer – Owaka Basin
- Roy Eastman – SWAP and overflows to Montgomery's Drain
- Graham Harrington – SWAP and overflows to Montgomery's Drain

10.1.1 C SDC

- Vicki Rollenson – Stockwater race network and flooding
- Andrew Mazey – Robinsons Road overpass and flooding.

11 Conclusion

This report describes the existing environment, the stormwater infrastructure proposed for the Project, the effect that it has on the environment and mitigation proposed.

The local topography is gently undulating, with the surrounding land being predominately rural, with some rural-residential, commercial and industrial areas. Constraints affecting the stormwater design of the proposed works are associated with the existing South West Area Plan (SWAP) (CCC, April 2009) water environment, the existing groundwater levels and protections zones, existing wells and the stockwater race network.

The three key stormwater issues which required addressing with the proposed infrastructure are the collection and disposal of stormwater generated within the Project, the passage of stockwater race flows and the passage of overland flows generated in the upstream catchment beneath the Project.

The collection and disposal system will typically consist of roadside swales and stormwater disposal points at regular intervals along the Project. First flush basins and treatment ponds will also be required in some areas.

The stockwater races will be conveyed beneath the Project via inverted siphons. A smaller diameter pipe to convey dry weather or 'typical' flows with a second larger diameter pipe to pass any flood flows beneath the Project will allow the land drainage function of the water race network to operate. Upstream flood flows which may arrive at the upstream side of the Project in natural depressions will be conveyed beneath the Project in large diameter siphons.

The discharge of stormwater to land will occur at numerous locations along the Project. This will be via infiltration through the base of the swales, via soak pits, through drainage pits, overland flow soak pits, and Project ponds, also from the base of overland flow and secondary siphon structures.

The assessment has identified that the Project has the potential to impact on the existing environment with regards to water quality (stormwater runoff, groundwater and surface water), potential flooding issues and changes in the land drainage function of stockwater races and the water supply in groundwater wells.

The Project will affect the number of Average Daily Traffic (ADT) volumes predicted to travel along SH1 and by 2041 there will be an expected increase in volume on MSRFL and a reduction in volume along SH1 north of CSM2. The change in ADT volume as a result of the Project will alter the quality of the stormwater runoff being disposed to land.

As outlined within this report, the existing State highway and local road network in the vicinity of the Project provides little in the way of stormwater quality treatment with untreated runoff easily entering the environment via the stockwater race network. The design philosophy includes separation of runoff from the Project from the surrounding environment. Through the minimum of sheet flow over a grassed verge and flow along a treatment swale prior to soakage to land an improvement in the receiving environment groundwater quality is expected to be realised. This will be a positive effect of the Project.

In general, the design of the Project complies with the rules specified in the NRRP and applies good industry practice with the majority of the Project discharges being treated. In areas inside the 'less than 6 metres groundwater' zone, the treatment system will include first flush basins and organic filter media to mitigate the effects of stormwater discharge to land. There are several areas where it is not possible to provide grassed organic filter layers. The discharges disposed to land at these locations will not be generated within the area serviced by the Project drainage system. As such, the effects of these discharges on groundwater quality are expected to be less than minor.

It is considered that the design of the Maize Maze Pond and Ramp Pond mitigates the effects of contaminants generated in road runoff prior to discharge to the receiving environment.

Overall the effects of the discharges on groundwater quality are expected to be less than minor.

The design standard for the highway drainage system is the 100 year Average Recurrence Interval (ARI) rainfall event including an allowance for climate change, as recommended by the MfE in their local body guidance manual (MfE, 2008). Disposal to land has the potential to reduce downstream flooding due to the reduction in contributing area, a positive effect on the existing environment.

Utilisation of total storm detention in the 100 year 24 hour rainfall event will ensure that spilling to Upper Knights Stream in the Halswell River catchment via Montgomery's Drain will only occur in extreme rainfall and/or groundwater events where dilution will be significant. The flows will be treated through a first flush and detention basin system and as such any effect is expected to be less than minor.

Conveyance of overland flows will be passed beneath the Project via siphon arrangements. The conveyance of the siphon pipes and protective bunds will be to the 100 year ARI design standard to prevent flooding of the highway drainage system.

Wells in the area range between 17 m and 177 m but are typically from 20 m to 50 m in depth. There are a number of groundwater takes, predominantly for crop irrigation, that may be affected by the works which are either beneath the Project footprint or within close proximity to the outside edge of the stormwater disposal system. Closing of affected wells is proposed to minimise any potential effects from a large oil/chemical spill entering the groundwater system via a disposal point. Moving the location of some stormwater disposal points may be possible during detailed design. The precise location of new wells will need to be identified in the detailed design phase. The detailed design will need to consider drilling, lining, well development & testing and connection of power supplies.

Mitigation measures are proposed to avoid or mitigate any adverse effects. There are a number of aspects of the design philosophy, which have been implemented in the proposed design, to mitigate environmental effects including: the design standard applied, the dispersed drainage, disposal and treatment system, overland flow siphons and stockwater race siphons.

Some residual risks do remain, particularly with regards to flooding and over-design events (both groundwater and rainfall) but this will be mitigated through design and through the proposed resource consent conditions.

A range of design options and issues have been considered for the Project, including Project vertical alignment, the discharge of surface water runoff, stormwater treatment, the treatment of stockwater races and overland flow paths.

The traffic on the new Project carriage will generate a contaminant load as does the existing highway. The treatment from the existing highway is informal at best and at other places discharges are almost direct to the adjacent stock water race. As part of the Project on the main alignment and on the adjacent feeder roads, over and underpasses is a collection and treatment system for stormwater runoff. This provides for treatment of runoff by swales prior to discharge to ground. Analysis of the effectiveness of the swales and the effects of discharges of stormwater to ground are minor.

The assessment has calculated that the effects of stormwater discharge on potable groundwater quality are low at a distance greater than 30 m from the designation boundary. This means that existing groundwater abstraction wells for potable supply should not be adversely affected by the operation of the project provided they are at least 30 m distance from the designation boundary.

Overall the environmental impact of the proposed infrastructure is considered to be minor or less than minor due to the proposed mitigation measures.

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Appendix A | Figures

Figure 11 Project Layout

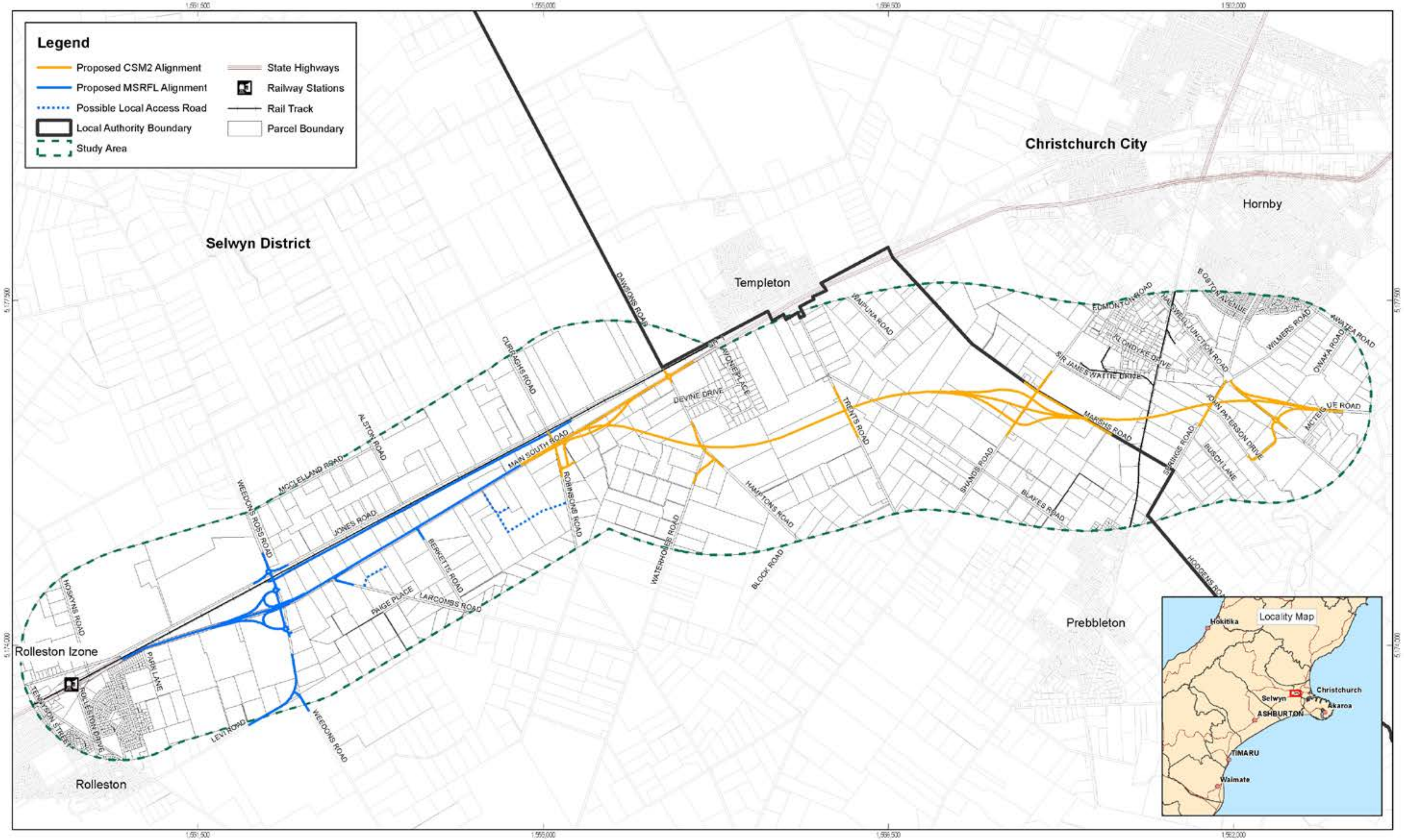


Figure 12 Land Zoning within 1 km of the Project

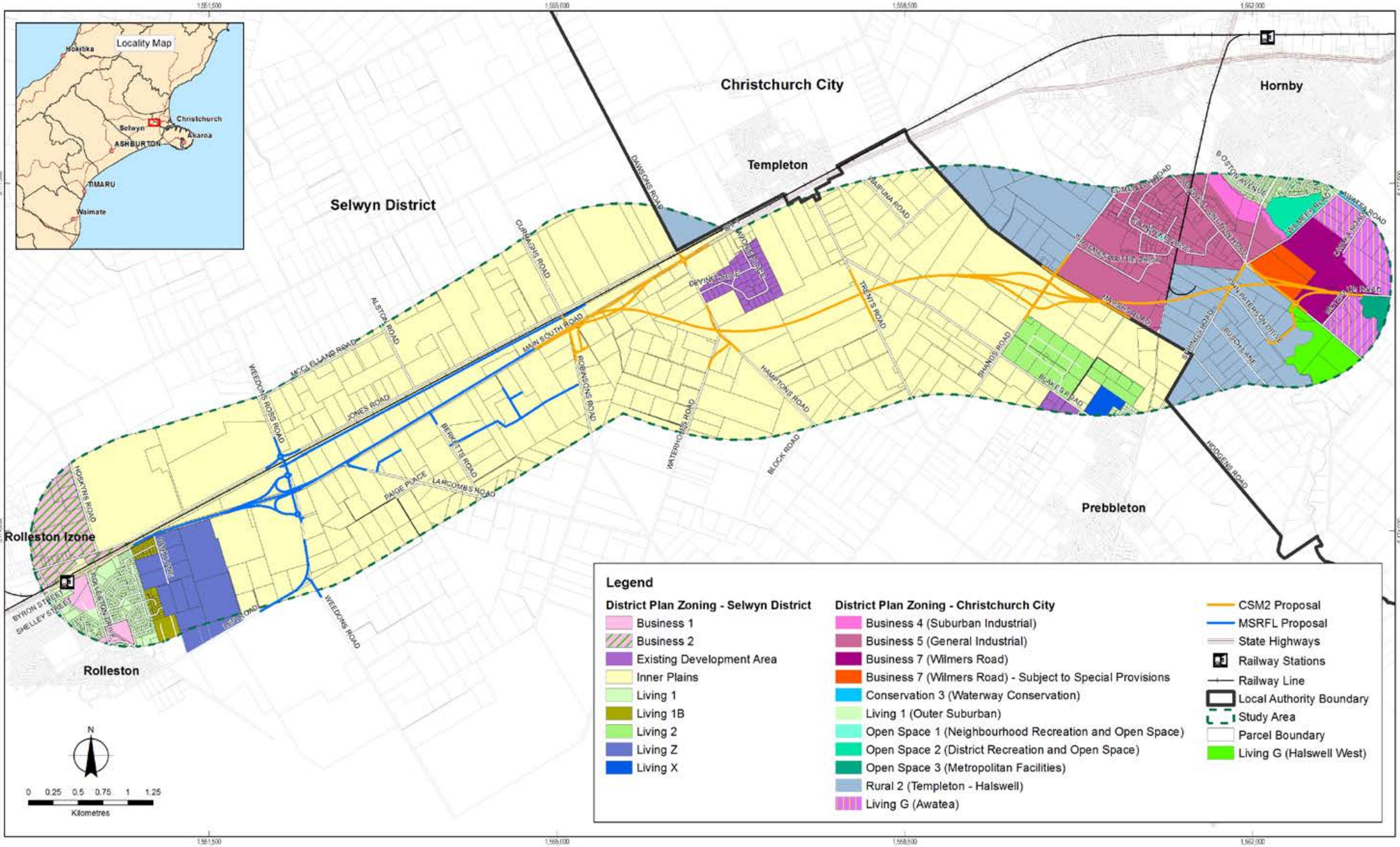


Figure 13 Stockwater Races within 1 km of the Project Overlaid on Groundwater Zoning

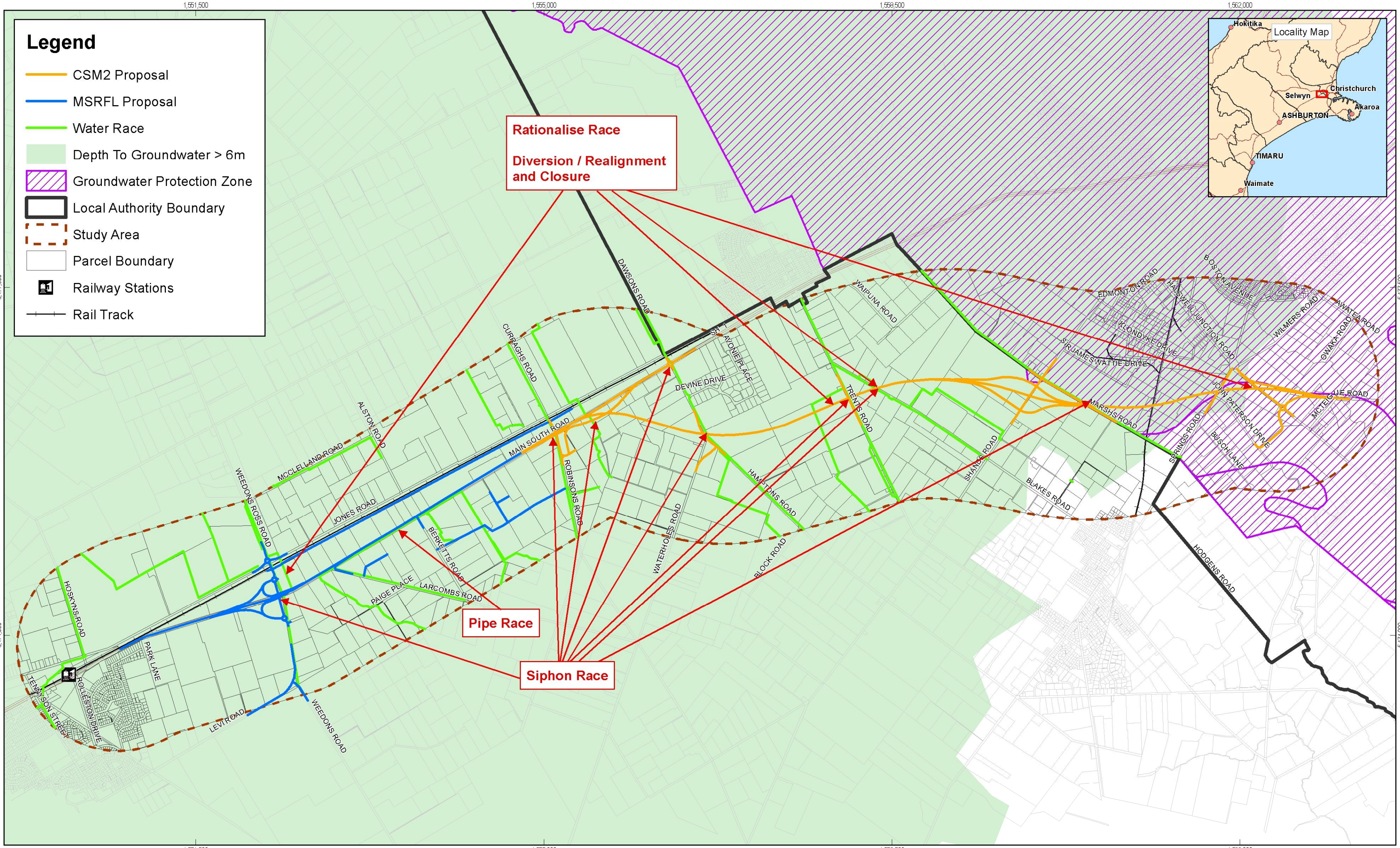


Figure 14 Design Groundwater Level

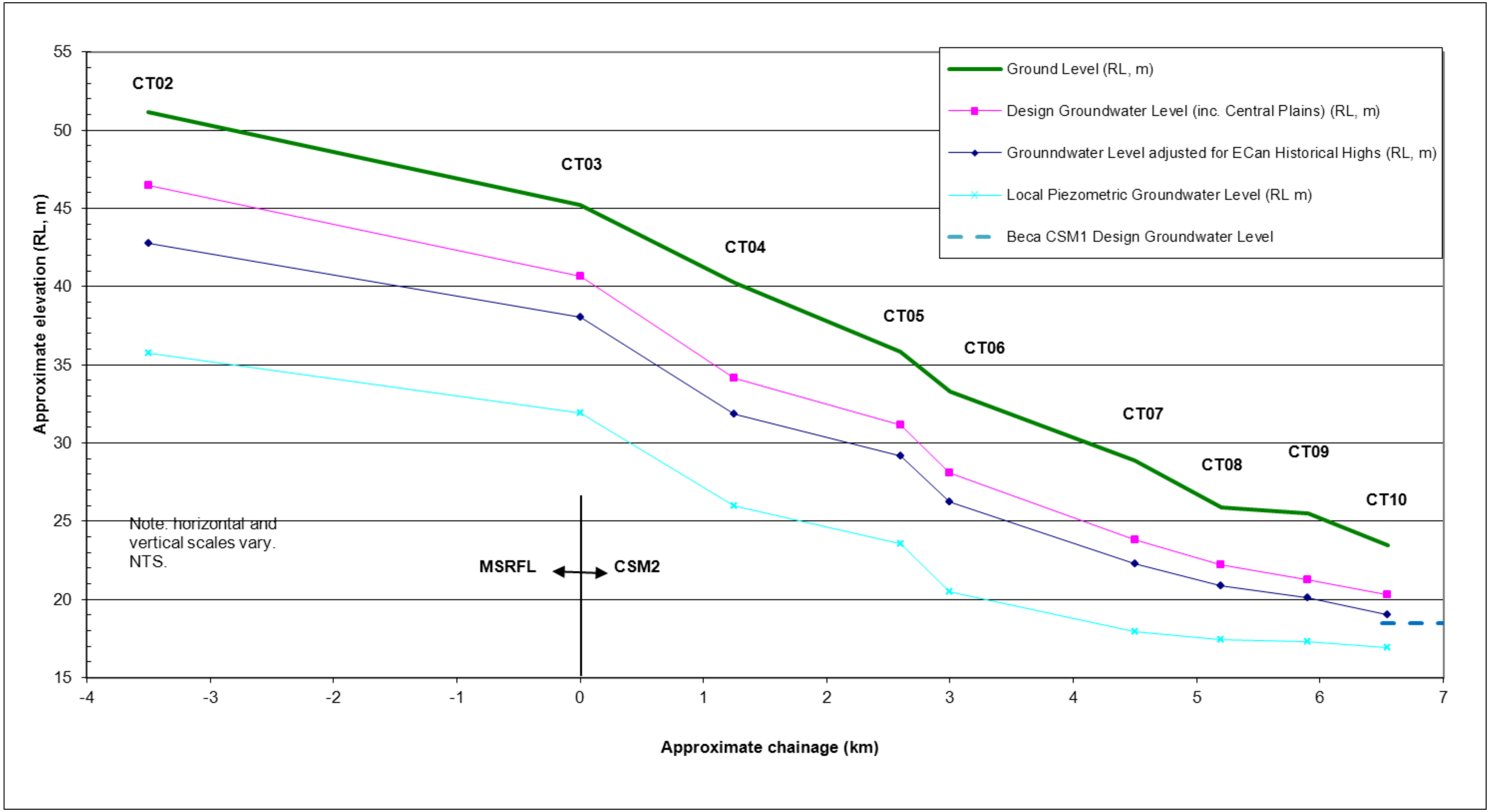


Figure 15 Water Supply Well Location Map

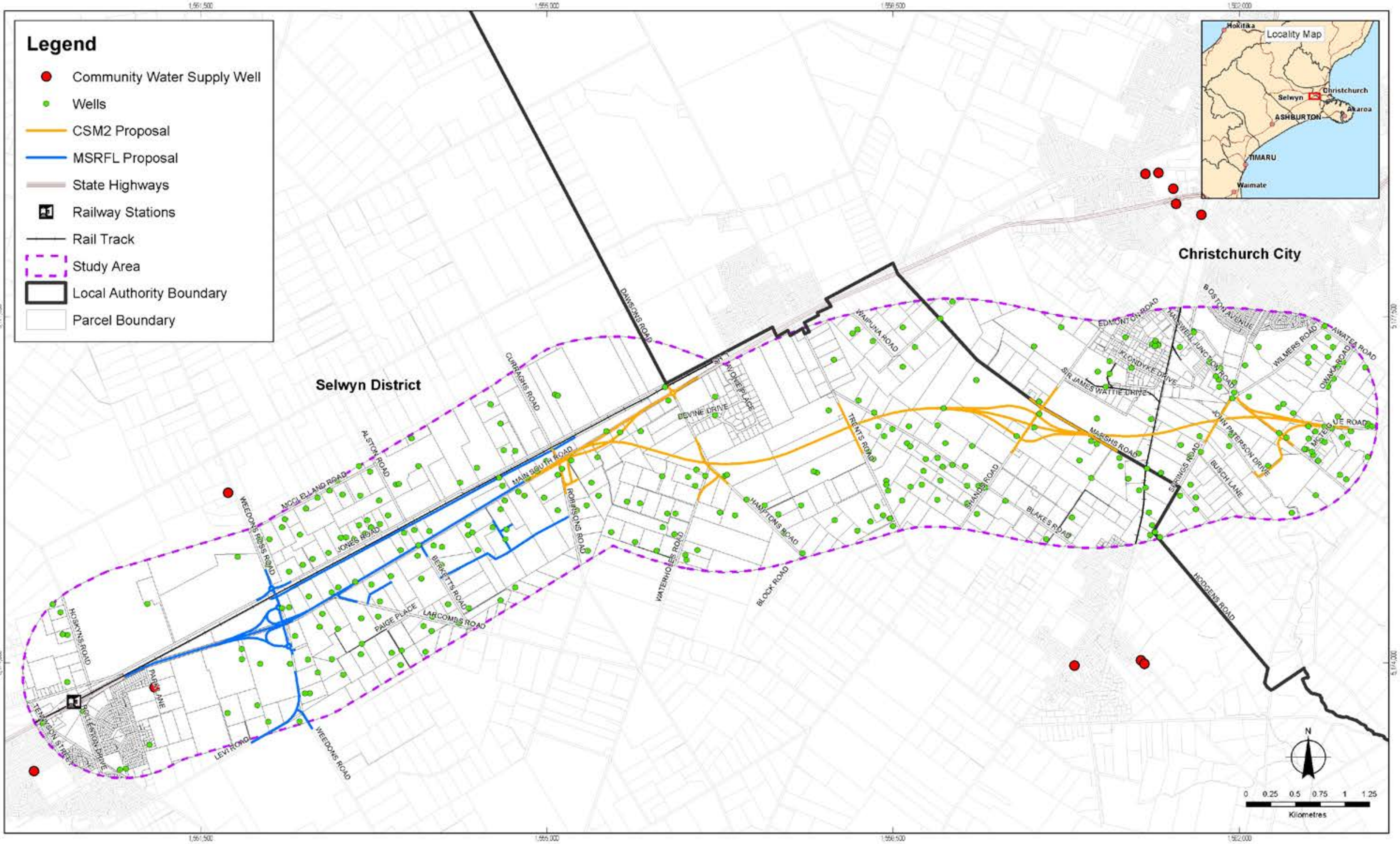


Figure 16 **Stockwater Changes Map**

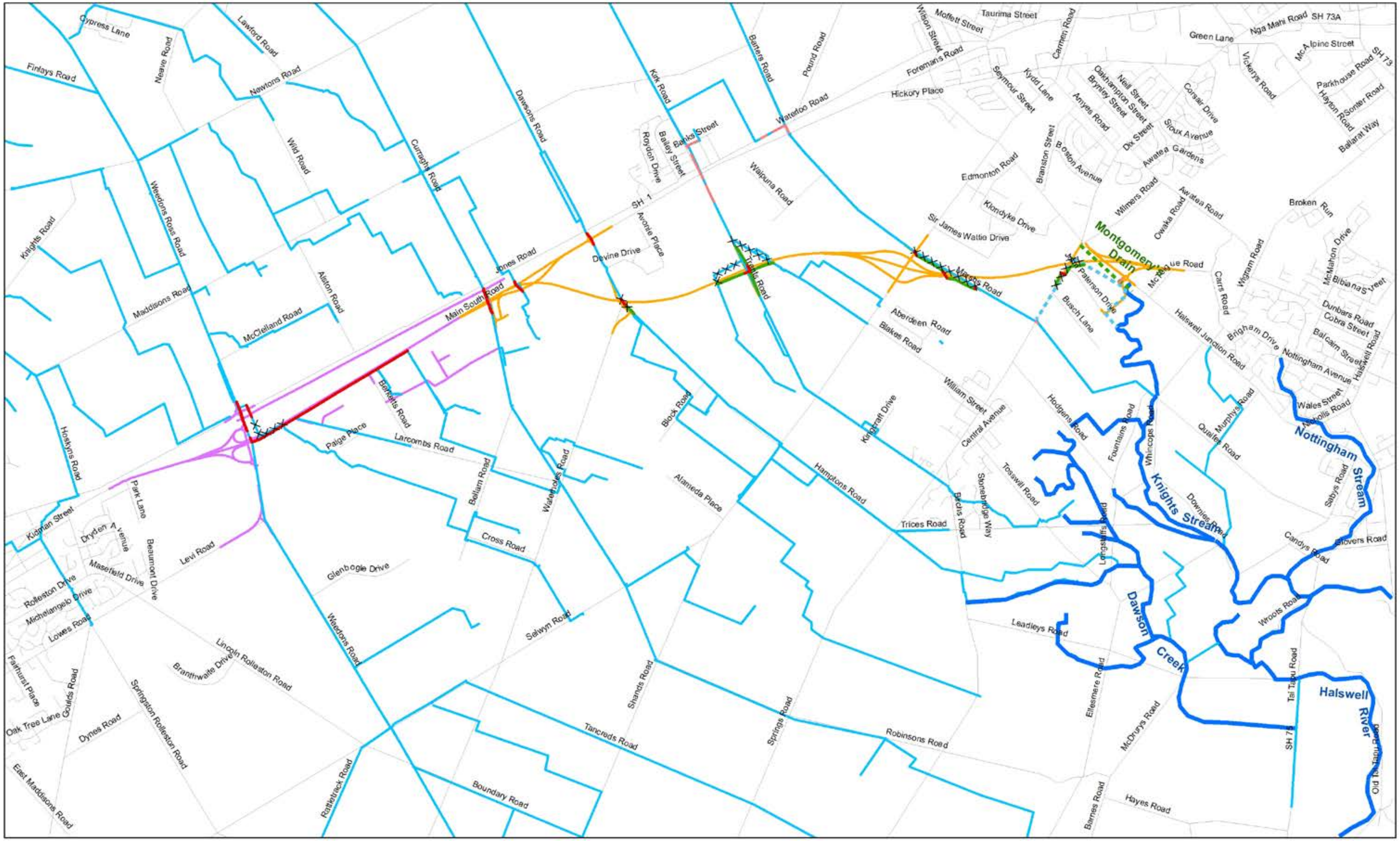
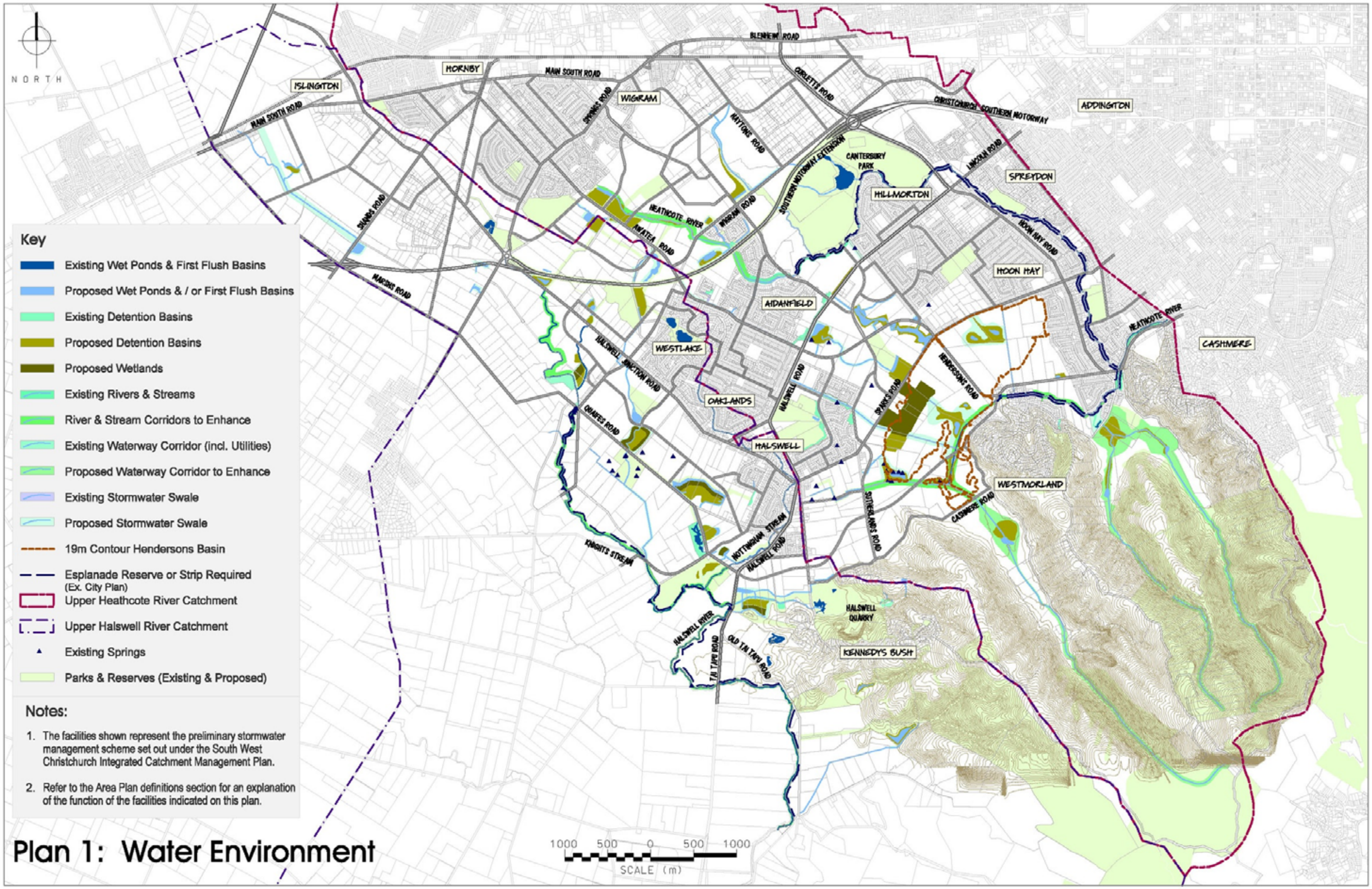


Figure 17 SWAP Final Plan 1 (south west)



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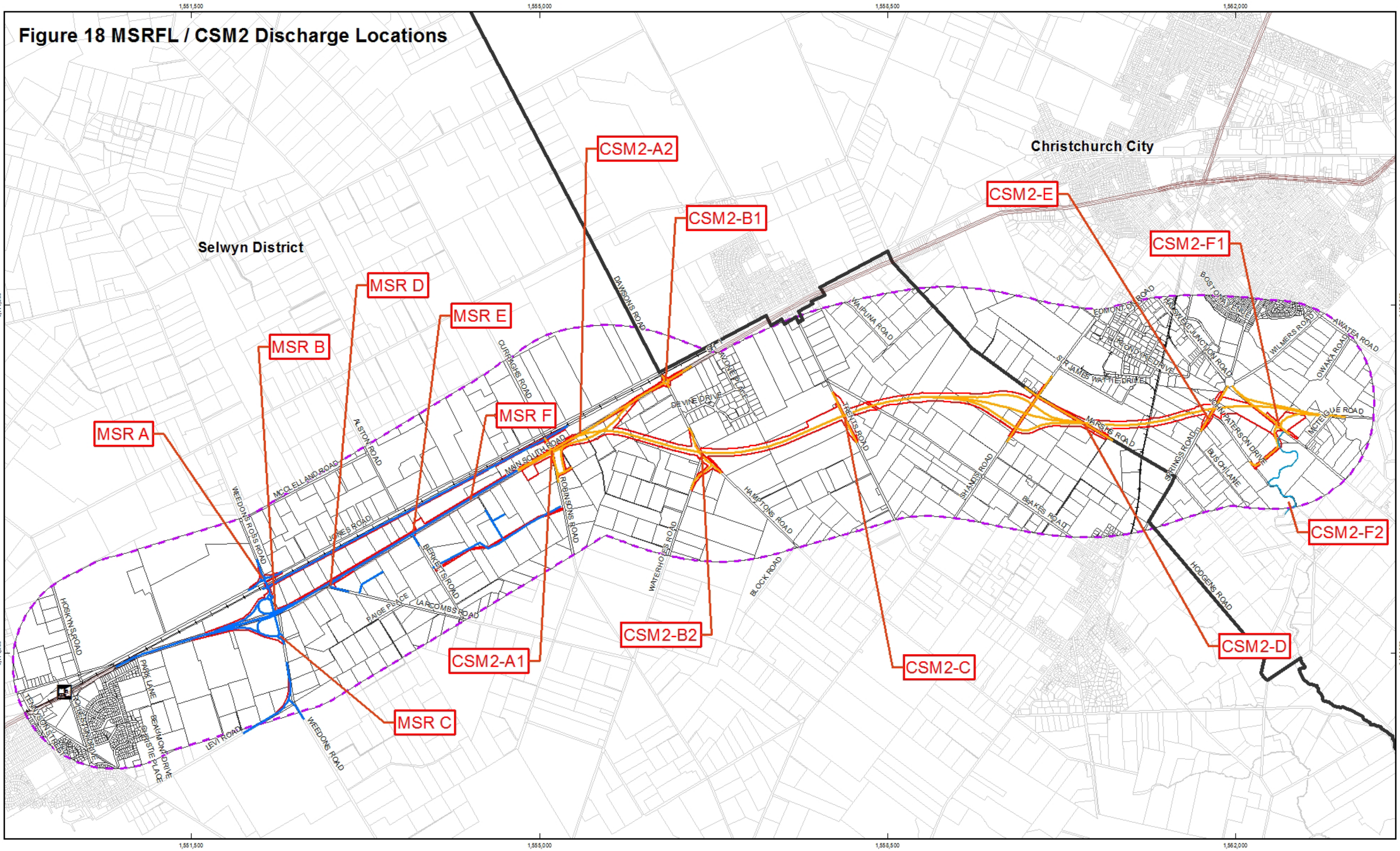
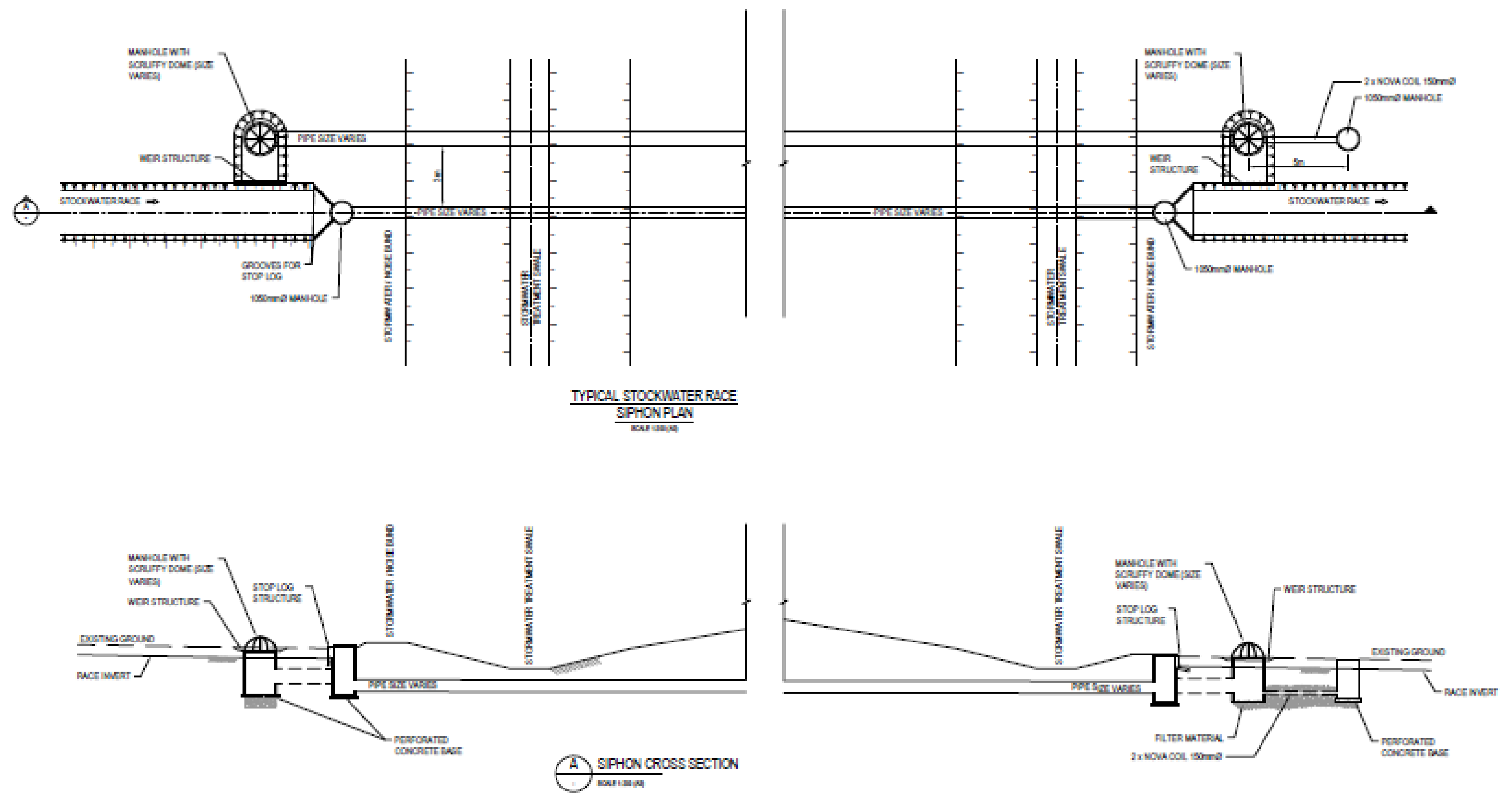
SOUTH WEST CHRISTCHURCH AREA PLAN

lp208105
Mar. 2009

Figure 18 MSRFL / CSM2 Discharge locations



Appendix B | Typical Details



FOR INFORMATION

A	FOR INFORMATION
	CK DGA CG 14.10.11

NZ TRANSPORT AGENCY
WAKA KOTAHI

GHD

Beca

DO NOT SCALE

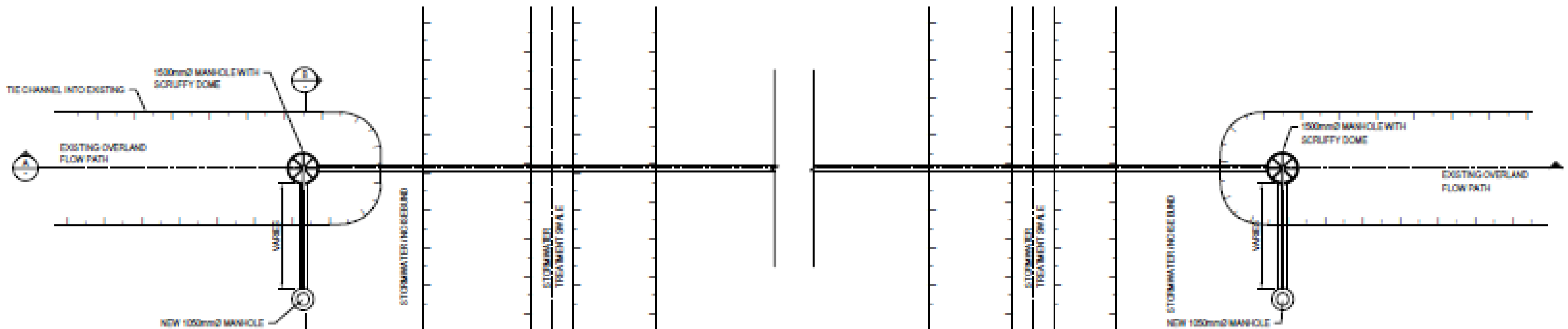
Drawn CK	Designed TP
Drafting Check	Design Check DGA
Approved CHRIS GREGORY	
Date 14.10.11	
A1 Scale	
A3 Scale 1:200	

Client: **NEW ZEALAND TRANSPORT AGENCY**

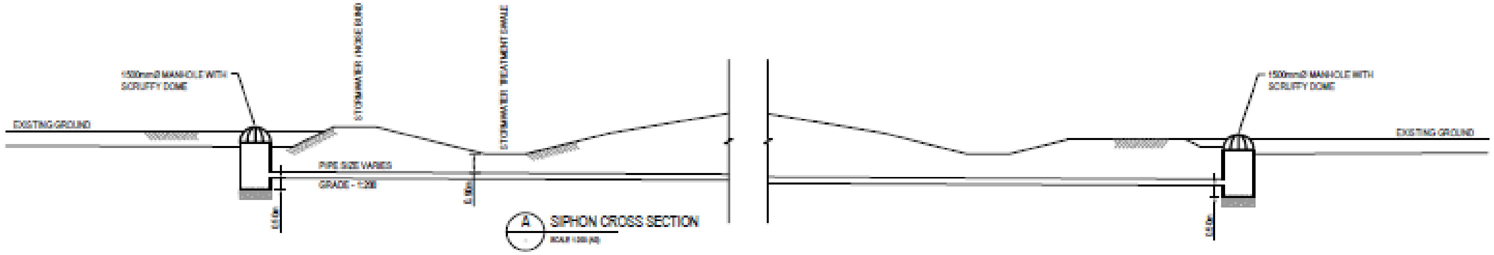
Project: **SH1 REGION 11 RP350/3.92 TO RP355/2.11 CHRISTCHURCH SOUTHERN MOTORWAY STAGE 2**

Title: **TYPICAL DRAINAGE DETAILS - SHEET 1 OF 5**

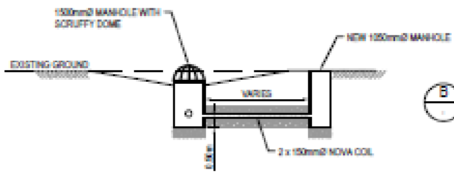
Drawing No. 62236-B-C451 Rev A



TYPICAL OVERLAND FLOW SIPHON PLAN



A SIPHON CROSS SECTION



B SIPHON DISPOSAL SECTION

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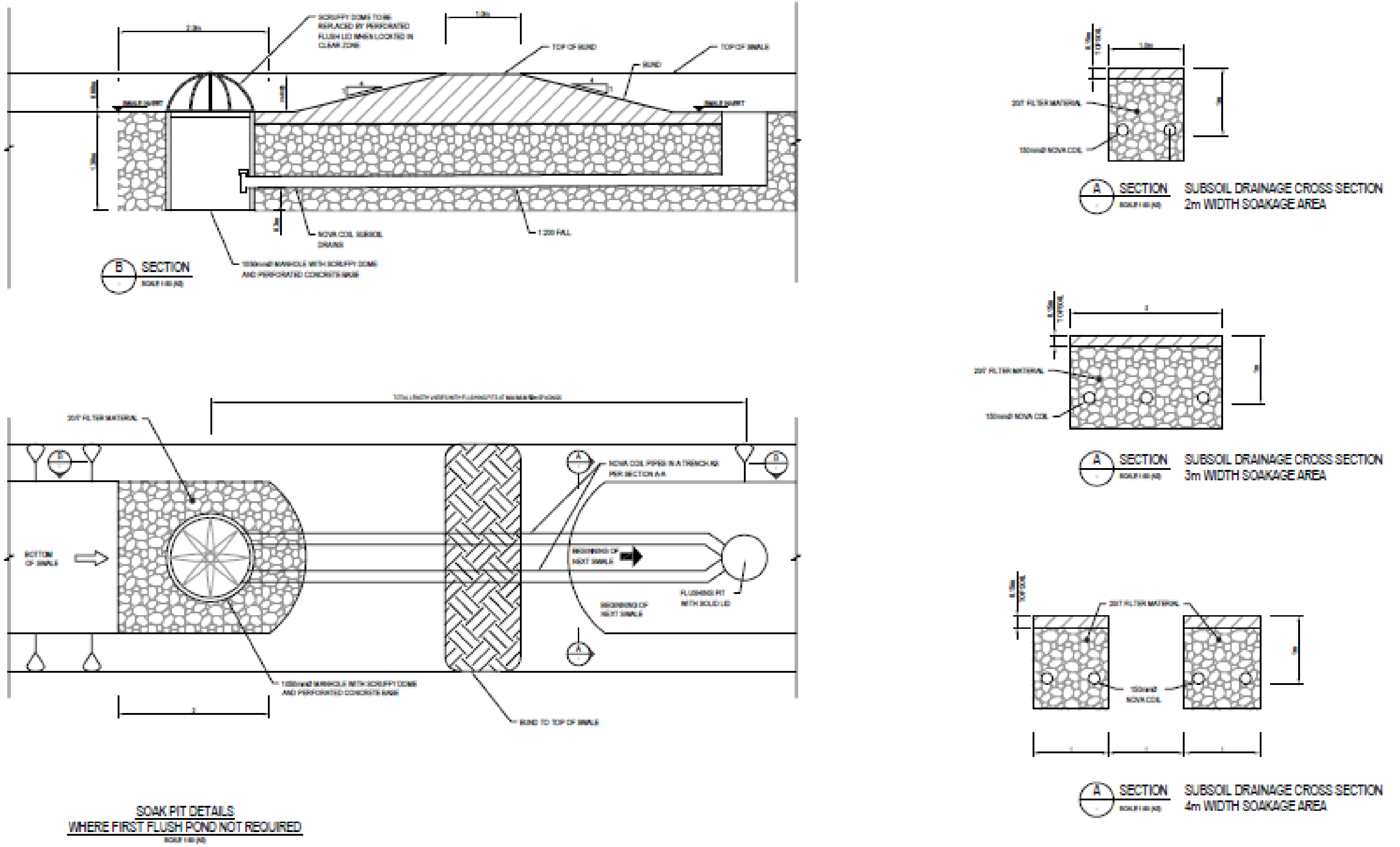
Drafting Check	BJB
Design Check	DGA
Approved	CHRIS GREGORY
Date	14.10.11
A3 Scale	1:200

Client	NEW ZEALAND TRANSPORT AGENCY
Project	SH1 REGION 11 RP350/3.92 TO RP365/2.11 CHRISTCHURCH SOUTHERN MOTORWAY STAGE 2
Title	TYPICAL DRAINAGE DETAILS - SHEET 2 OF 5
Drawing No.	62236-B-C452
Rev	A



SOAK PIT DETAILS
WHERE FIRST FLUSH POND NOT REQUIRED

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A	FOR INFORMATION	AMW	DGA	CG	14.10.11

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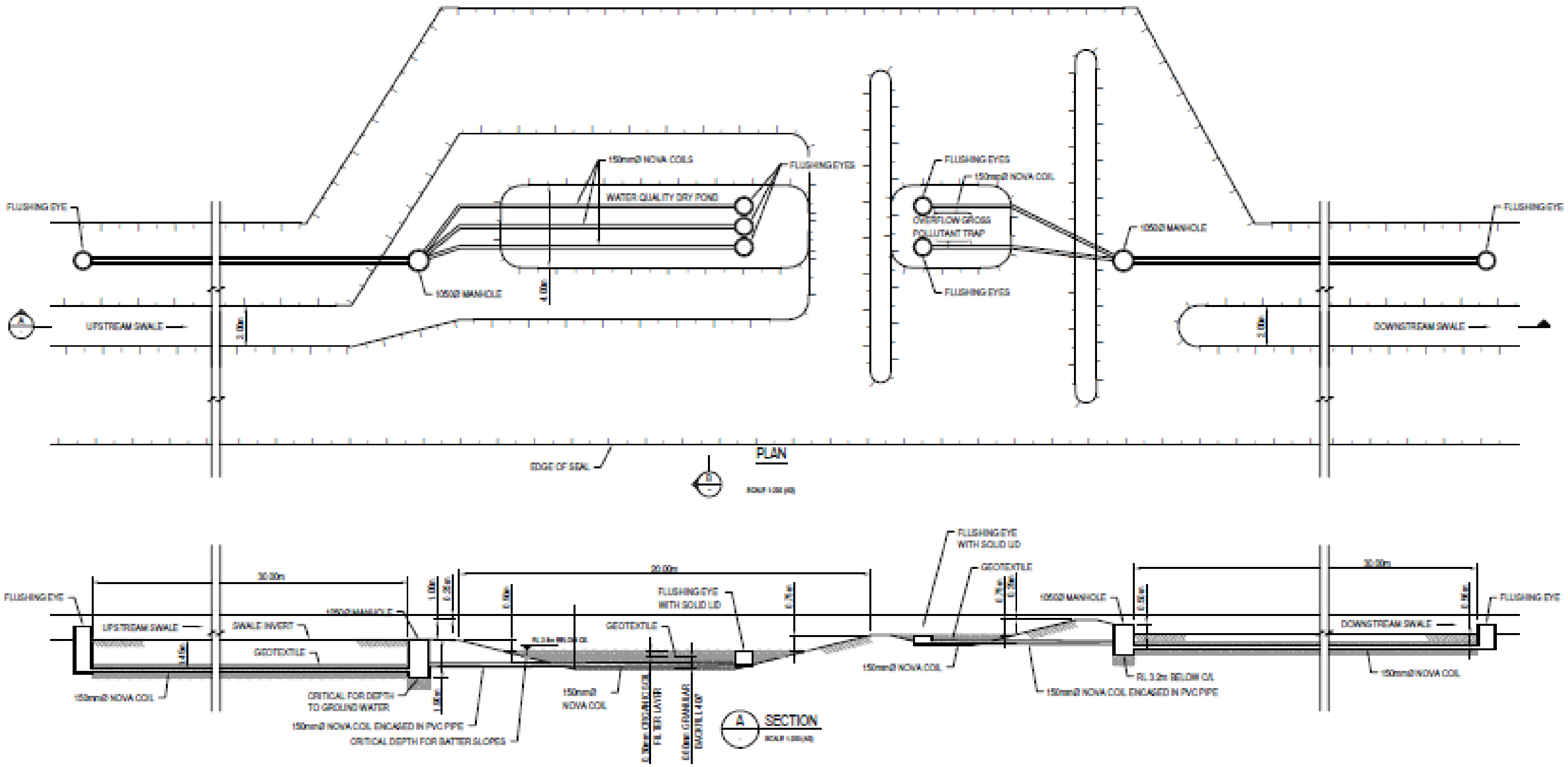
Approved CHRIS GREGORY	
Date 14.10.11	
A1 Scale	
A3 Scale 1:50	

Client: NEW ZEALAND TRANSPORT AGENCY
Project: SH1 REGION 11 RP350/3.92 TO RP355/2.11
CHRISTCHURCH SOUTHERN MOTORWAY STAGE 2
Title: TYPICAL DRAINAGE DETAILS - SHEET 3 OF 5

Drawing No. 62236-B-C453 Rev A



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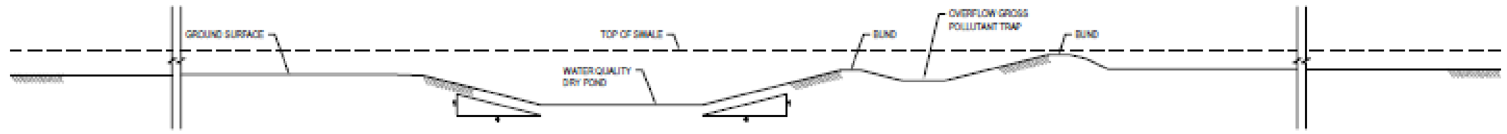
DO NOT SCALE

Drawn AMW	Designed TP
Drafting Check BSB	Design Check DGA
Approved CHRIS GREGORY	
Date	
A1 Scale	
A3 Scale 1:200	

Client: NEW ZEALAND TRANSPORT AGENCY
Project: SH1 REGION 11 RP350/3.92 TO RP355/2.11
CHRISTCHURCH SOUTHERN MOTORWAY STAGE 2
Title: TYPICAL DRAINAGE DETAILS - SHEET 4 OF 5

Drawing No. 62236-B-C454 Rev A



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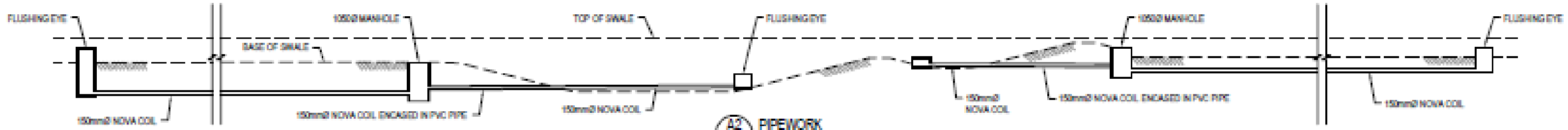


A2 PIPEWORK

FOR INFORMATION

No	Revision	Drawn	Checked	Approved	Date
A	FOR INFORMATION	AMW	DGA	CG	14.10.11

NZ TRANSPORT AGENCY
WAKA KOTAHI

DO NOT SCALE

Drawn: AMW
Designed: TP
Design Check: DGA
Approved: CHRIS GREGORY
Date: 14.10.11
A3 Scale: A3 HALF SIZE SHOWN

Client: NEW ZEALAND TRANSPORT AGENCY
Project: SH1 REGION 11 RP350/3.92 TO RP355/2.11
CHRISTCHURCH SOUTHERN MOTORWAY STAGE 2
Title: TYPICAL DRAINAGE DETAILS - SHEET 5 OF 5

Drawing No: 62236-B-C455 Rev: A

Appendix C | Ground Water Report

NZ TRANSPORT AGENCY
WAKA KOTAHI

Appendix C, Technical Report 3

Christchurch Southern Motorway Stage 2 and Main South Road Four Laning

Assessment of Groundwater Effects

November 2012

Quality Assurance Statement			
	Prepared by:	Mark Utting, Rebecca Morris	November 2012
	Reviewed by:	Ann Williams	November 2012
	Approved by:	Brent Meekan	November 2012

Contents

Appendices

Abbreviations used in this report

Abbreviation	Description
AEE	Assessment of Environmental Effects
Beca	Beca Infrastructure Ltd
bmp	below measurement point
CPW	Central Plains Water Enhancement Scheme
CSM	Christchurch Southern Motorway
CSM1	Christchurch Southern Motorway – Stage 1
CSM2	Christchurch Southern Motorway – Stage 2
ECan	Environment Canterbury the publicity name of Canterbury Regional Council
Fm	Formation
GHD	GHD Limited
HJR	Halswell Junction Road
L/s	Litres per second
Kh	Hydraulic conductivity in the horizontal direction
Kv	Hydraulic conductivity in the vertical direction
mBGL	metres beneath ground level
mRL	metres Relative Level
m/s	metres per second
RRO	Robinsons Road Overpass
SH1	State Highway 1

Executive Summary

An assessment of environmental effects (AEE) of the project on the groundwater environment has been completed to supplement the assessment of stormwater disposal and water quality effects prepared by GHD for the proposed Christchurch Southern Motorway Stage 2 (CSM2) and the Main South Road Four Laning (MSRFL) collectively referred to as "the Project." This groundwater AEE has focussed on CSM2 and the eastern portions of MSRFL and is based on data collected from various sources including the geotechnical investigations for the Project, hydrogeological studies for the first stage of the motorway (CSM1), groundwater models developed for the greater vicinity of the Project and other pertinent hydrogeological reports on the area. The data derived from these sources is not reproduced here but can be found in the various Project reports and other sources as listed in the reference section of this report.

The Project overlies the alluvial silts, clays, sands, and gravel deposits of the Springston Formation which contains a shallow unconfined aquifer consisting of sandy gravels with local cobbly zones and varying amounts of silts. The eastern portion of CSM2 near Halswell Junction Road (HJR) is underlain by soils with higher silt and clay contents than the western end near the proposed Robinsons Road Overpass (RRO). This shallow unconfined Aquifer 1 (as designated by Environment Canterbury (ECan) for the first aquifer within 20 m of ground surface in the Selwyn-Waimakariri area (Davey, 2006)) is the aquifer potentially affected by the CSM2 and MSRFL alignment.

Three possible effects on groundwater were identified:

1) a rise in groundwater level elevations ("mounding") beneath infiltration structures,

2) a limit to maximum groundwater level rises beneath RRO and HJR through pumping wells (RRO) and under-drains (HJR), and

3) potential contamination of groundwater through infiltrated stormwater runoff from the motorway.

These potential effects were assessed using a combination of 3-D MODFLOW computer models supplemented by 2-D (map view) analytical models to quantify groundwater flow directions in the greater Project area and flow rates and changes in water levels that would be caused by CSM2.

Before an assessment of effects could be carried out, the maximum high groundwater levels expected after the implementation of the Central Plains Water Enhancement Scheme (CPW[1]) were calculated using historical data from two long-term ECan monitoring wells together with assessments made by others as part of the consent application for the CPW. Maximum high levels of 39.6 mRL (beneath RRO) and 19.4 mRL (beneath HJR) were calculated. These levels are above the planned roadway at the low point of RRO and above the bottom of the proposed ponds at HJR. Groundwater abstraction wells (RRO) and an under-drain system (HJR) are planned to limit the CPW-induced water table rises at these two sites.

The effects of the mounding rises from infiltration of the 24-hour/100-year rainfall event were calculated to be about 1.6 m beneath RRO, about 25 mm 100 m away and not measurable beyond a distance of 250 m

[1] In this report we abbreviate the Central Plains Water Enhancement Scheme to CPW to be consistent with other reports prepared for the Project. Consenting documents not prepared for this Project use the acronym CPWES

from RRO. This mounding would be offset by groundwater abstraction well pumping prior to stormwater infiltration. Pumping would be started when groundwater levels rose to within 1 m of the base of the infiltration trench below Robinson Road. Pumped water would be directed to the stockwater race along Robinsons Road. Pumping would cause drawdown of groundwater levels in a cone of water table depression centred on the abstraction wells and would likely be obscured by the overall rise of groundwater levels that would occur throughout the area from infiltration of rainfall during the storm event. Water table mounding beneath the HJR ponds from the 24-hour/100-year event as indicated by a 2-D model of the Mushroom Pond would be 1.4 m to 2.6 m. However, such mounding would result in groundwater rising above the bottom of the ponds, thereby reducing storage volumes needed to attenuate discharge peaks. An under-drain system beneath the lined and unlined HJR ponds will reduce this mounding and CPW-increased groundwater levels.

The effects of the reduced water levels beneath RRO and HJR under high water conditions are considered to be less than minor. These systems will only be operated occasionally when water levels are near their maxima and will not lower groundwater levels below those that occur today or have occurred in the past. The frequency and duration of pumping cannot be accurately predicted using the available data. However, statistically, the maximum groundwater level is predicted to rise up to within 1 m of the low point of Robinsons Road (39.5 mRL) less than 5 % of the time after the CPW is in full operation and more likely closer to 1% of the time. Because of this uncertainty, allowing Robinsons Road to flood occasionally may remain a viable alternative to the pumping and water level control system modelled to support GHDs stormwater disposal design. When the RRO wells are pumped at a total of 100 L/s to limit the water level rise, the drawdown effects are estimated to be a drawdown of 10 mm at a distance of 1 km and about 1 m at a distance of 100 m from RRO. The HJR under-drain system will operate under gravity drainage and is estimated to produce less than 50 L/s. Because it is a gravity operated system, no existing or future groundwater users will be affected because any drawdown “cone of depression” caused by a nearby pumping well would shut off flow from the under-drain system before the under-drains could limit pumping from the nearby well.

This assessment of groundwater effects indicates that *with* the proposed monitoring and design features, the effects on groundwater levels caused by the construction and operation of CSM2 and the assessed portions of MSRFL will be less than minor. Water level rises will be controlled through pumping and under-drains. Disposal of the removed groundwater will be through diversion to surface water. The changes in water levels away from the Project caused by the proposed groundwater level control systems and the infiltration of stormwater are predicted to be much smaller than the natural variations in groundwater levels and the effects are considered to be less than minor.

1. Introduction

The NZ Transport Agency (NZTA) has engaged GHD Ltd (GHD) and its sub-consultant, Beca Infrastructure Ltd (Beca) to undertake an Assessment of Environmental Effects (AEE) of the proposed Christchurch Southern Motorway Stage 2 (CSM2). The proposed CSM2 is for a new four lane motorway extending from CSM1 at Halswell Junction Road for approximately 8 km to join Main South Road (SH1) to be "four laned" (MSRFL) near Robinsons Road. The combined CSM2 and MSRFL are collectively referred to as "the Project." Construction and operation of the Project has the potential to affect groundwater. This report identifies the possible effects of CSM2 on the groundwater regime and provides an assessment of these effects. Several options for mitigation were considered. These include monitoring, design modifications to control groundwater levels, design modifications to raise the roadways above projected high groundwater levels and the option of allowing temporary flooding to non-motorway road surfaces (specifically, Robinsons Road where it will pass under CSM2). These options have been presented to NZTA in a "risk memo" prepared by Beca (2012) and sent to GHD for presentation to NZTA. Based on discussions between GHD and NZTA and between GHD and Beca, the first two mitigation options listed above have been identified as the preferred mitigations presented in this AEE. Effects of the wider Project (specifically water quality) are addressed in a separate report, Assessment of Groundwater Quality Effects - Christchurch Southern Motorway Stage 2 (CSM2) and Main South Road Four Laning (MSRFL), (Beca, 2012).

2. Existing Environment

2.1 Geology

The geology beneath the upper 40+ m of the Project alignment consists of alluvial gravels, sands, silts and clays of the Springston Formation, with glacial outwash deposits of gravels, sands, silts and clays of the Burnham Formation lying directly beneath.

The Yaldhurst Member of the Springston Formation underlies much of the CSM2 alignment. Containing significant percentages of silts, sandy silts and clay, this finer-grained member forms the surficial layer over much of the CSM2 project area, in particular at the eastern ends and is generally about 0.1 m to 2.2 m thick.

The Halkett Member of the Springston Formation underlies the entire Project area, directly beneath the Yaldhurst Member (where present) or from ground surface downward where the Yaldhurst Member is absent. In the Project vicinity, the Halkett Member is coarser than the Yaldhurst Member and consists of sandy gravel to sandy gravel with cobbles with varying silt content to depths of greater than 21.5 mBGL (metres beneath ground level) as indicated by investigation boreholes along the proposed Project alignment.

Glacial outwash deposits of sand and gravel with interlayered silts and clays of the Burnham Formation underlie the Springston Formation. A review of well bore records on file with Environment Canterbury (ECan) indicates similar descriptions for both the Springston and Burnham Formation. Brown and Wilson (1988) indicate that "it is difficult to distinguish Springston Formation from the underlying Burnham Formation in the gravel with sand, silt and clay matrix penetrated by wells" confirming the similarity in descriptions in bore logs.

Appendix A in the New Zealand Transport Agency Report for Christchurch Southern Motorway Stage Two Geotechnical Interpretive Report (GHD-Beca, 2011) discusses subsurface geology and presents a long-section showing bore locations.

2.2 Hydrogeology

A general overview of the hydrogeology of the Project area is given in Technical Report 3 (Assessment of Stormwater Disposal and Water Quality Environmental Effects (GHD, 2012). In summary, groundwater beneath the project area flows from the west-northwest toward the east-southeast through a series of unconfined, semi-confined and confined aquifers consisting of permeable sands and gravels (with cobbly zones) separated by leaky aquitards consisting of silts, clays and fine sands. The Yaldhurst Member of the Springston Formation with its generally lower permeability tends to form a confining layer to the east of the Project area. ECan defines the shallow unconfined aquifer within the upper 20 m of ground surface in the Selwyn to Waimakariri area as "Aquifer 1" (Davey, 2006). Based on the depths of the water table (reported below) of less than 20 m, the Halkett Member with its generally higher permeability forms the shallow, unconfined aquifer beneath the project area. The Halkett Member (Aquifer 1) becomes locally confined, where the groundwater level rises above the fine-grained base of the overlying Yaldhurst Member. The sands, silts, gravels and clays of the underlying Burnham Formation are in direct hydraulic continuity with the sands, silts, gravels and clays of the Halkett Member. Together they form a leaky, interconnected aquifer system.

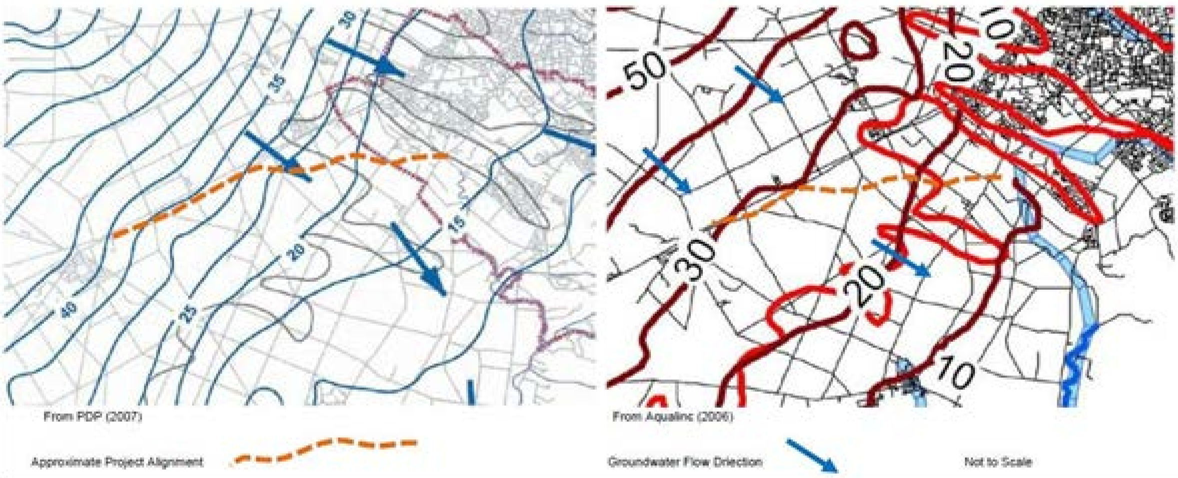
The depth to the water table within the uppermost aquifer beneath the CSM2 alignment varies from 17 m to 3 m beneath the proposed alignment depending on the year, season, and position along the alignment (generally deeper beneath the western portions and shallower beneath the eastern portions). Analysis of groundwater flow directions, hydraulic properties of the aquifer materials and water levels is provided below.

2.2.1 Groundwater Flow Direction and Gradients

Shallow groundwater beneath the proposed Project alignment flows generally from the west-northwest toward the east-southeast as indicated by both regional and project-specific analyses.

Two excerpts from water level contour maps submitted to support consent applications (PDP, 2007 and Aqualinc, 2006) show regional flow in the shallow Aquifer 1 (Figure 1.) The groundwater elevations shown in these excerpted figures were derived from water levels available from ECan. (We have added flow arrows placed at right angles to contours for clarity following standard hydrogeological practice.)

Figure 1 Groundwater Flow Direction and Contour Maps from PDP and Aqualinc



A more detailed analysis of flow direction and water level contours during times of highest groundwater levels for the site shows similar flow directions. In our analysis we have downloaded water levels from ECan's on-line data base for all wells within 10 km of the Project along with bore log information. The highest water levels for shallow wells (completion depths of 40 m or less) were then contoured using the contouring package Surfer. Outliers (typically low levels indicating pumping or lower water conditions) were discarded with the end result a composite-high-water level contour map (Figure 2). Although the water level data do not indicate a "snapshot" of water levels all taken at approximately the same time, they do indicate a best-approximation of high water conditions needed to assess environmental effects as well as provide input for analysis of the soakage structures planned for the Project. Figure 2 shows groundwater in Aquifer 1 flowing toward the southeast with a gradient of about 0.002 beneath the western end of the Project (near Robinsons Road) and about 0.001 beneath the eastern end (near Halswell Junction Road).

Figure 2 High Groundwater Flow Direction and Contour Map of the Project Vicinity

2.2.2 Hydraulic Properties of the Subsurface Materials

The hydraulic properties of the surficial deposits of the Yaldhurst and Halkett Members have been derived from two sources: existing analyses from various reports on the area available from ECan and field testing. Existing data includes values derived from pumping tests on file with ECan and representative values used in groundwater models (by others) developed and calibrated to data from the greater project area.

Table 1 Hydrogeological Parameters from Regional Models

Model Hydrostratigraphic Unit	Kh [m/s]	Kv [m/s]
Aqualinc (2007)		
Aquitard 1 (Springston Fm fines)	8×10^{-5} to 6×10^{-4}	4×10^{-9} to 3×10^{-8}
Aquifer 1 (Springston Fm sand/gravel/cobbles)	1×10^{-3} to 7×10^{-3}	1×10^{-5} to 7×10^{-5}
Aqualinc (2006)		
Aquitard 1 (Springston Fm fines)	3×10^{-8}	3×10^{-8}
Aquifer 1 (Springston Fm sand/gravel/cobbles)	1×10^{-3} to 3.5×10^{-3}	1×10^{-4} to $3.5 \times 10^{-}$

Infiltration testing conducted as part of the investigation of the material properties along the Project alignment (along with previous investigations for the CSM1 alignment) indicated a range of hydraulic conductivities of the surficial deposits of the Springston Formation (coarse-grained Halkett Member and fine-grained Yaldhurst Member). The results of these tests are summarised in Table 2.

Table 2 Summary of Infiltration Test Results

Test Site	TP601	TP602	TP603	TP604	TP606	TP607	TP608	AH07	AH08	TP601
Rate [m/s]	5.4×10^{-5}	7.2×10^{-4}	4.2×10^{-5}	1.3×10^{-6}	2.2×10^{-5}	9.4×10^{-6}	9.8×10^{-6}	1.8×10^{-5}	4.7×10^{-6}	5.4×10^{-5}

The values in the table are comparable to or slightly lower than those representing Aquifer 1 most likely because of the higher silt content of the surficial soils where testing occurred.

2.2.3 Water Levels

Water levels (depths to water and elevations) have been measured at 10 piezometers (CT-1 through CT-10) along the proposed Project alignment. These depths to water and elevations have been recorded four times per day from 7 January through 26 January 2011 in 10 piezometers and from 7 January 2011 through 13 February 2012 in one piezometer (CT10). The mean depths to water and elevations are summarised in Table 3. Piezometer locations shown in Appendix A of the New Zealand Transport Agency Report for Christchurch Southern Motorway Stage Two Geotechnical Interpretive Report (GHD-Beca, 2011) are reproduced in Appendix C of this AEE document.

Table 3 Mean Water Levels along the Project Alignment (from West to East)

Piezometer	CT01	CT02	CT03	CT04	CT05	CT06	CT07	CT08	CT09	CT10
MP [mRL]	54.90	51.19	45.21	40.28	35.84	33.31	28.89	25.86	25.53	23.47
Mean DTW [m bmp]	14.22	15.44	13.26	14.29	12.30	12.83	10.94	8.42	8.20	5.81
WL [mRL]	40.68	35.76	31.95	26.00	23.54	20.48	17.96	17.44	17.33	17.66

The mean January 2011 water levels reported in Table 3 fall within the range of depth to water values listed in the general overview on the hydrogeology of the Project vicinity described in Section 2.2.

3. Potential Groundwater Issues

Three potential groundwater issues have been identified:

- increases in groundwater level (mounding) beneath each infiltration structure,
- decreases in groundwater level from beneath drain/well systems installed to limit groundwater rises during periods of high groundwater levels, and
- the introduction of contaminants to the groundwater system from runoff from the motorway.

Ground settlement resulting from groundwater drawdown (often an issue of concern with motorway construction in other areas) is not an issue here because the project will not cause the lowering of the groundwater table below current levels or those with the effects of CPW added. Each of the three identified potential groundwater issues is briefly discussed below.

3.1 Increases in Groundwater Levels

Runoff from the motorway will be collected and diverted to infiltration structures consisting of grassed swales, ponds and soakaways. Water infiltrating at these structures will percolate downward to the water table where it will cause the underlying groundwater to rise and spread out as a "mound." The increase in groundwater level has the potential to affect local wells by causing the water levels in the wells to rise, resulting in a decreased lift and lower energy costs for pumping. Consideration should be given to whether a groundwater level rise has the potential to affect basements in adjacent buildings if the rise is large enough.

3.2 Decreases in Groundwater Levels

Subsurface drains and/or wells are planned to limit the future elevation of the water table beneath the ponds proposed for the Halswell Junction Road (HJR) interchange. Wells are planned for a similar purpose beneath the Robinsons Road Overpass (RRO) where the carriageway of Robinsons Road is to be completed approximately 6.5 m beneath current ground level. The lowering of groundwater levels from beneath the RRO through pumping for up to 25 days at 100 L/s whenever the water level rises up to within 1 m of the roadway surface with discharge to a stockwater race along Robinsons Road and the lowering of groundwater levels via under-drains from beneath HJR with gravity drainage and discharge to Upper Knights Stream might be considered groundwater "takes" with the potential to affect existing well water levels.

3.3 Introduction of Contaminants to Groundwater

The use of infiltration structures to direct runoff from the motorway to the ground and subsequently to groundwater in the shallow unconfined Aquifer 1 has the potential to introduce contaminants to groundwater, if present in the original runoff and if not removed through treatment prior to infiltration. Such contaminants have the potential to move with groundwater to down-gradient users or to discharge to surface water via springs or base flow.

4. Methodology

The general procedure for assessing the effects of groundwater level changes was to first quantify likely groundwater levels in the future under extreme “natural” high water levels and then assess the additional increases from the completion of CPW[2]. The effects of future changes to land use or other types of future development in the area beyond the effects of CPW were not included in the analysis. The resulting high levels were then used to generate water level contour maps for the potentially affected Aquifer 1. These contour maps have been used for all subsequent assessments on water level changes and identifying areas where existing wells could be affected.

Water level increases caused by infiltration structures and water level decreases caused by under drains or wells were assessed using three separate computer models developed using the 3-D groundwater flow modelling package MODFLOW and the Visual MODFLOW-Pro interface (Schlumberger, 2011). The first model was a regional model developed to replicate the shallow groundwater flow system of the greater CSM2 area (including much of MSRFL) over an approximate 11 km by 12 km area. The model was semi-calibrated to replicate groundwater flow contours that matched the predicted highest groundwater level that included the effects of CPW. Two sub-models, one for Robinsons Road Overpass (RRO) and one for Halswell Junction Road (HJR) were then developed by using the regional model as a guide for setting up aquifers, aquitards, water levels, and groundwater flow directions. Each of these two sub-models was then used to assess water level changes at their respective locations (RRO and HJR). The 3-D MODFLOW models were supplemented by the use of 2-D analytical models.

The methods of analysis and results are discussed below.

[2] The CPW Enhancement Scheme will direct surface water for irrigation of new and existing farm lands. The combination of new water applied to the region and the replacement of groundwater by surface water for existing farmlands currently irrigated by well water will cause groundwater levels throughout the region to rise. Details can be found in Weir (2007 and 2009).

5. Water Levels beneath the Project

The two methods for assessing groundwater levels in Aquifer 1 are described below. The first method gives a more detailed time-series based assessment of likely levels beneath two areas of interest, RRO and HJR. The second gives an assessment of likely water levels in the groundwater model vicinity that includes CSM2 and much of MSRFL.

5.1 Prediction of Water Levels Based on Long-term Water Level Data

Following is a description of the first method used to generate water level statistics for the shallow groundwater beneath the RRO and HJR areas.

- Water level data were downloaded from ECan's data base for wells that have been monitored over the long term near the east and west ends of the Project alignment: M36/0217 and M36/4018. Well locations are shown in Figure 3.

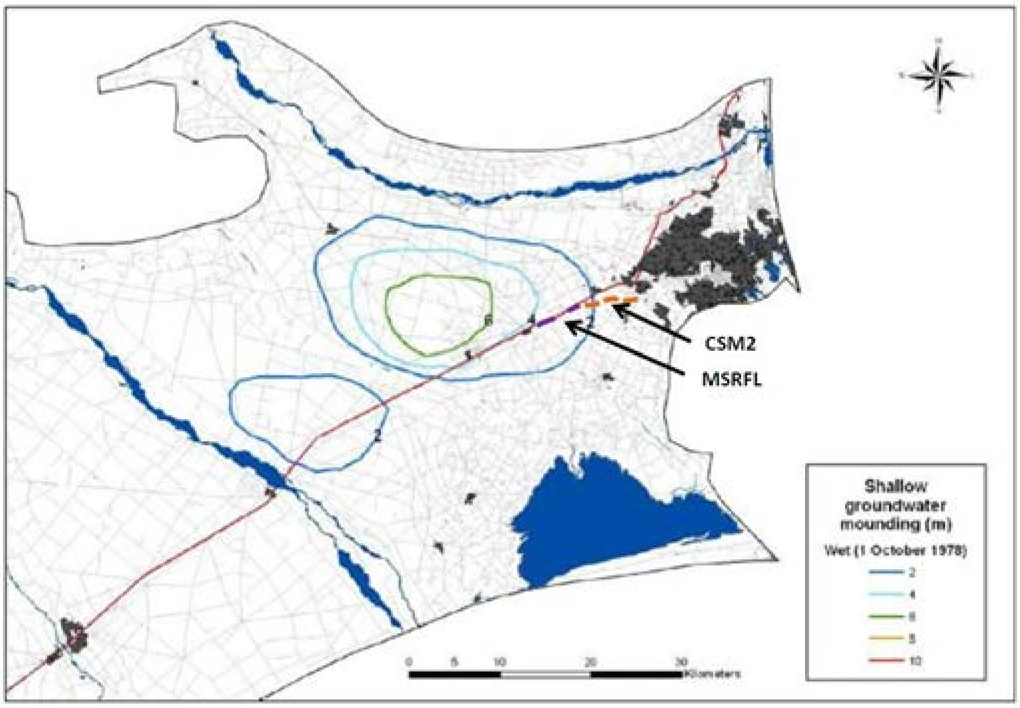
Figure 3 ECan and CSM2 Well Locations for Water Level Analyses

- The data were visually assessed and it became apparent that before 1988 water levels were only measured sporadically but from 1988 on, water levels were measured on a regular basis – mostly weekly. It was also apparent that the highest water levels were recorded during the post-1988 period. Data collected prior to 1988 were then discarded from the analysis.
- The water levels were then ordered from highest to lowest and the maximum, minimum, median, range and various frequency-based percentile water levels were calculated for each well.
- Water level data collected during January 2011 from the investigation piezometer closest to each site was then compared with the water levels from ECan wells measured at the same time and the relative

differences were then calculated. The analysis indicated that piezometer CT03/RRO groundwater levels were approximately 3.5 m below those in M36/0217 while the CT10 groundwater levels were about 1.0 m higher than those in M36/4018. Because the position of the Maize Maze ponds is 20 % closer to M36/4018 than CT10, the water levels below these ponds are likely to be 0.8 m higher than M36/4018. The water levels were linearly adjusted, accordingly.

- The range between highest and lowest water levels in the ECan wells was then scaled to represent the likely range in water levels at CT03 (RRO) and beneath the Maize-Maze pond (HJR). Scaling was by a linear adjustment based on distance between the investigation wells (CT03 and HJR) and the ECan wells. The distance between the ECan wells is approximately 11 km with CT03 about 27 % of this distance from M36/0217 while HJR is about 12 % of this distance from M36/4018. The range in water levels calculated for M36/0217 (7.47 m) was scaled down to a range of 6.19 m for CT03. The range in water levels calculated for M36/4018 (2.76 m) was scaled up to a range of 3.31 m for HJR. Water levels were adjusted using the changed scaling around the median value. In other words, the median value water level for the scaled and un-scaled range of water levels remained unchanged.
- The results are synthesised water level distributions for each site. These synthesised water levels represent our best estimate of current water level distributions at each site.
- The scaled and adjusted water levels were then further increased to account for the water level rise predicted from the CPW. An increase of 2.5 m was assigned to the groundwater levels at CT03 while a rise of 1.3 m was assigned to CT10 (HJR). The increase was calculated based on the predicted water level rises presented in the evidence of Julian Weir (2007) as part of the consenting process for CPW (Figure 4).

Figure 4 Maximum Water Level Rise Predicted for CPW (from Weir, 2007)



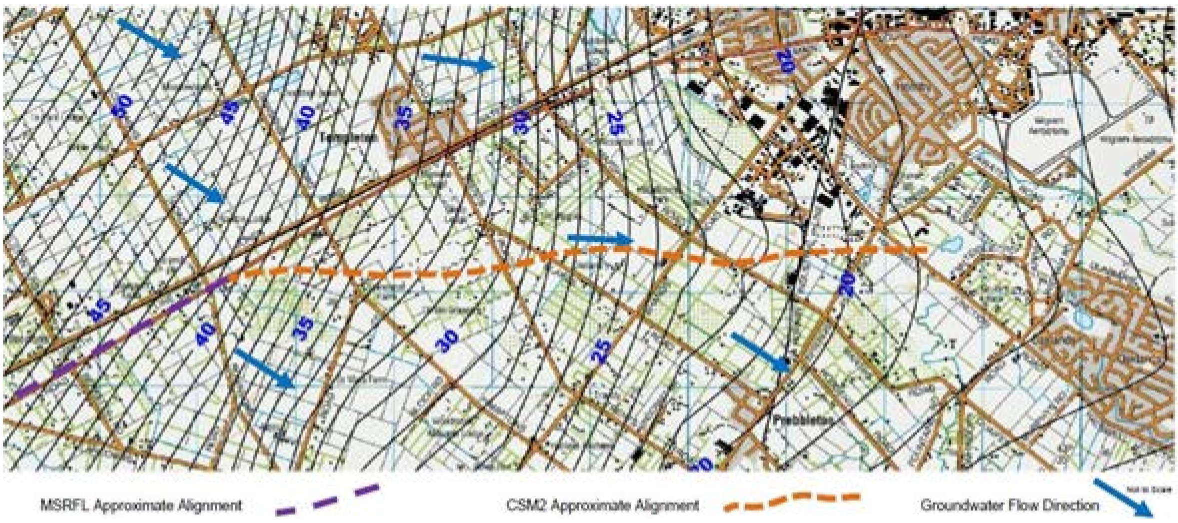
The water levels in Figure 4 represent the maximum levels during a wet period under the CPW "with dam" option which has not yet been consented. The increases in water level ("mounding") under the "no-dam" option (which has been consented) are only slightly smaller, about 100 mm less in the Project vicinity, as indicated in Weir (2009). To be conservative (assuming the highest water levels) we have applied the predicted mounding effects of "with dam" option.

5.2 Prediction of Water Levels Based on Long-term Water Level Data

The second method used to assess water levels beneath the RRO and HJR structures and the general area considered for the groundwater modelling was as follows:

- All wells in the ECan database within 10 km to 15 km of the proposed alignment were identified on 3 May 2012. Water level data were then downloaded and the highest levels for each well contoured using the program Surfer. The contour plot was reviewed and anomalies and outliers were investigated. Data from wells completed in deeper aquifers, obviously incorrect or otherwise not appropriate (pumping during measurement or completion in deeper aquifers), were then discarded and the water levels re-contoured. The result was a contour map of the greater project area representing the highest water levels recorded to date.
- The generated contour map was then compared with existing water level contour maps to verify that the general flow directions and hydraulic gradients were consistent with previous interpretations.
- The predicted effects of the CPW programme were then added to the levels used in the "highest-levels-to-date" plot. The effects were scaled with an increase of 1.3 m applied to the east end near HJR and an increase of 2.5 m applied to the west end near RRO. The result was a contour map of the project area representing the highest water levels recorded to date with the predicted CPW increase (Figure 5).
- The full effects of the CPW water level increases are expected to take 3 to 5 years, with 90% of the response to be seen within 2 to 4 years (Weir, 2007).

Figure 5 High Groundwater Flow Directions and Contours with Predicted CPW Increases



5.3 Water Levels – Robinsons Road Overpass

Based on the two different methods, we have calculated two possible high water levels for the RRO. The first method indicates a highest water level beneath the low point of Robinsons Road of 39.6 mRL while the second indicates a maximum level of 41.2 mRL. The results of the more detailed analysis (method 1) are more likely to represent future conditions at the RRO. However, because long-term groundwater level data are not available from beneath the RRO site, a level of uncertainty remains.

The results of the first method of analysis for Robinsons Rd are set out in Table 4.

Table 4 Results of Water Level Analyses at Robinsons Road

Water Level Exceedance Percentile [%>]	Water Levels at M36/0217 based on data from 1988 to date [mRL]	RRO Water Level Adjusted by range, difference in Jan 2011 levels and distance from M36/0217 [mRL]	RRO Water Level Adjusted by range, difference in Jan 2011 levels and distance from M36/0217 with predicted CPW increase of 2.5 m [mRL]
Max WL	41.9	37.1	39.6
5 % >	39.2	34.9	37.4
7.5 % >	38.2	34.1	36.6
10 % >	37.9	33.8	36.3
15 % >	36.9	32.9	35.4
25 % >	35.8	32.1	34.6
50 % >	34.5	31.0	33.5
75 % >	33.8	30.4	32.9
95 % >	33.2	29.9	32.4
Min WL	32.4	29.2	31.7

Planned Robinsons Road Low Point – 39.5 mRL

5.4 Halswell Junction Road

Using the two different methods, we have calculated two possible high water levels for the groundwater beneath the HJR and the Maize Maze pond. The first method indicates a highest water level beneath the proposed Maize Maze pond at HJR of 19.4 mRL while the second indicates a maximum level of and 18.8 mRL. As for Robinsons Road, the results of the more detailed analysis of the first method are more likely to represent future conditions at the HJR. However, because long-term groundwater level data are not available from beneath the HJR site, a level of uncertainty remains.

Details of the first method of analysis for Halswell Junction Road are set out in Table 5.

Table 5 Results of Water Level Analyses at Halswell Junction Road

Water Level Exceedance Percentile [%>]	Water Levels at M36/4018 based on data from 1989 to date [mRL]	HJR Water Level Adjusted by range, difference in Jan 2011 levels and distance from M36/4018 [mRL]	HJR Water Level Adjusted by range, difference in Jan 2011 levels and distance from M36/4018 with predicted CPW increase of 1.3 m [mRL]
Max WL	16.9	18.1	19.4
5 % >	16.4	17.5	18.8
7.5 % >	16.2	17.1	18.4
10 % >	16.0	16.9	18.2
15 % >	15.8	16.7	18.0
25 % >	15.5	16.4	17.7
50 % >	15.2	16.0	17.3
75 % >	14.9	15.6	16.9
95 % >	14.6	15.3	16.6
Min WL	14.2	14.8	16.1

Maize-Maze Pond Design Water Level = 19.75 mRL/Invert Level = 18.75 mRL

Ramp Pond Design Water Level = 19.00 mRL / Invert Level = 18.75 mRL

CSM2 Roadway ~ 21 mRL

6. MODFLOW Models

6.1 Regional Model

A regional steady-state, 3-D groundwater flow model was developed to help define groundwater flow conditions in the greater vicinity of the Project and identify down-gradient locations for assessment of water quality effects, and to allow the set up of two sub-models (RRO and HJR). Details of the model are included in Appendix A.

The two-layer model consisting of cells 100 m on a side was set up using the graphical interface Visual MODFLOW 2010 (Schlumberger, 2010). The model is centred about CSM2 as a rectangular region 5 km (N – S) by 12 km (W – E) with the bottom set at -40 mRL. The positions of the layers and the distribution of hydrogeological units were determined from investigation boreholes along the alignment and water well logs available from the ECan data base for areas away from the alignment. Hydraulic properties were assigned to each layer, assessed from a combination of values used in calibrated regional models developed by Aqualinc (2006 and 2007), field testing done as part of the Project and CSM1 investigations, and data from Beca on similar materials from other parts of Canterbury. A simple geological profile was modelled comprising 3 m of sandy silt overlying sandy gravel. A 1 m thick confining layer of clay, silt and sand (the "confining layer") present at the surface on the eastern end of the site was included in the model. Three zones of sandy gravel were also included in the model with the zone with the highest permeability representing a paleo-river channel. Review of bore logs from wells in the area show variable amounts of silts and clays at varying depths and in complex patterns between the wells resulting in water bearing zones separated by leaky, discontinuous aquitards. In order to develop a simplified model appropriate for the available data and purpose at hand, the effects of the silts and clays were modelled by assigning a vertical anisotropy to the sand and gravel aquifer materials. The initial values for hydraulic conductivities were adjusted through the calibration process with the adjusted values as set out in Table 6. The colours on the table correlate with the zones on the model cross-section presented in Figure A2 in Appendix A.

Table 6 Hydraulic Parameters in the Regional Model

Material Type	Model Layer	Hydraulic Conductivity	
		Kh (m/s)	Kv/Kh
Clay, silt and sand (confining layer)	1	1.0×10^{-6}	0.1
Sand and gravel with silt	1,2	1.2×10^{-3}	0.1
Sand and gravel	1,2	3.5×10^{-3}	0.1
Sand and gravel (paleo-river channel)	1,2	8.0×10^{-3}	0.1

The model was set up (see Appendix A for set-up details) with constant head boundaries placed to replicate the highest groundwater level contour map (Figure 5). These levels were then adjusted to replicate the maximum groundwater levels predicted for Aquifer 1 (layer 2 in the model) by adding the predicted effects of CPW.

Recharge at a rate of 58 % of precipitation was applied at the surface where gravel was present; and at a rate of 36 % of rainfall where clay was present. This equates to approximately 400 mm/yr and 250 mm/yr respectively assuming an average annual rainfall of 680 mm/yr based on data from the Halswell River Ryans Bridge Gauge (location shown in Appendix D). The Drain boundary function was used to model the stream to the east of the Project and to the south of Halswell Junction. The River boundary function was used to model the standing water bodies to the north of the proposed motorway.

The output of the model without the predicted effects of CPW is presented in Figure 6. The output of the model with the effects of CPW (Figure 7) was used to set up the two sub-models.

Figure 6 Regional Model without CPW - Steady State Contours

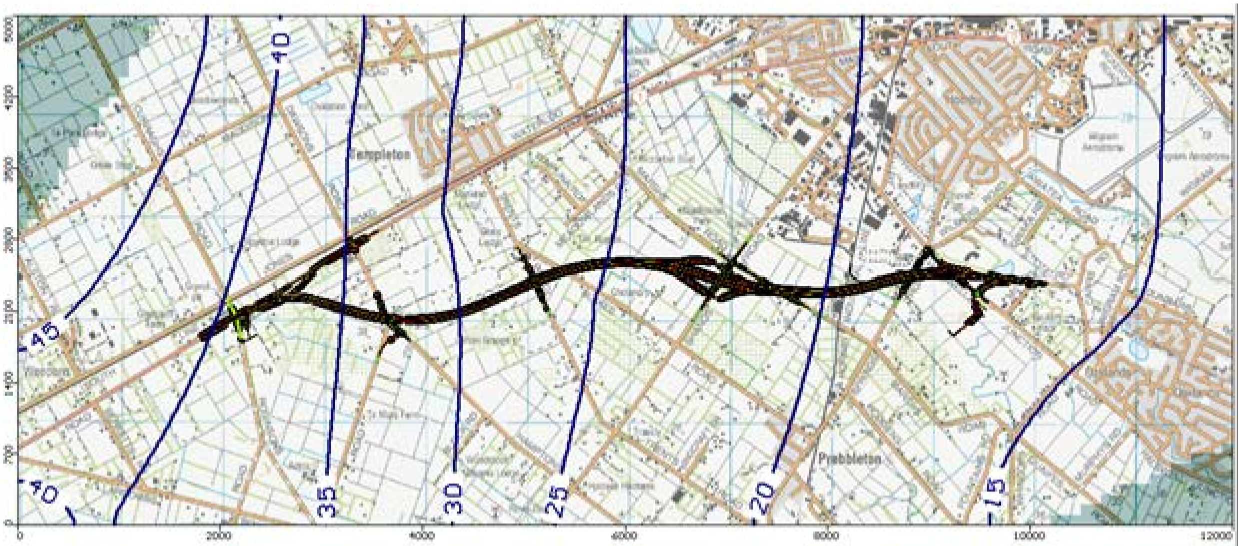
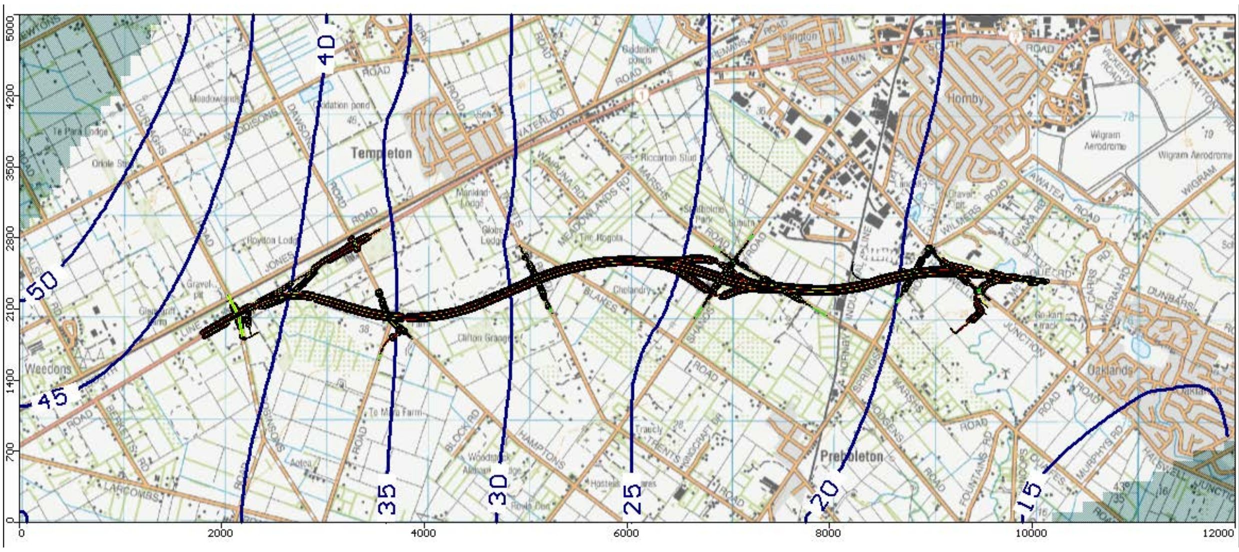


Figure 7- Regional Model with CPW - Steady State Contours



6.1.1 Contaminant Assessment using the Steady-State Model

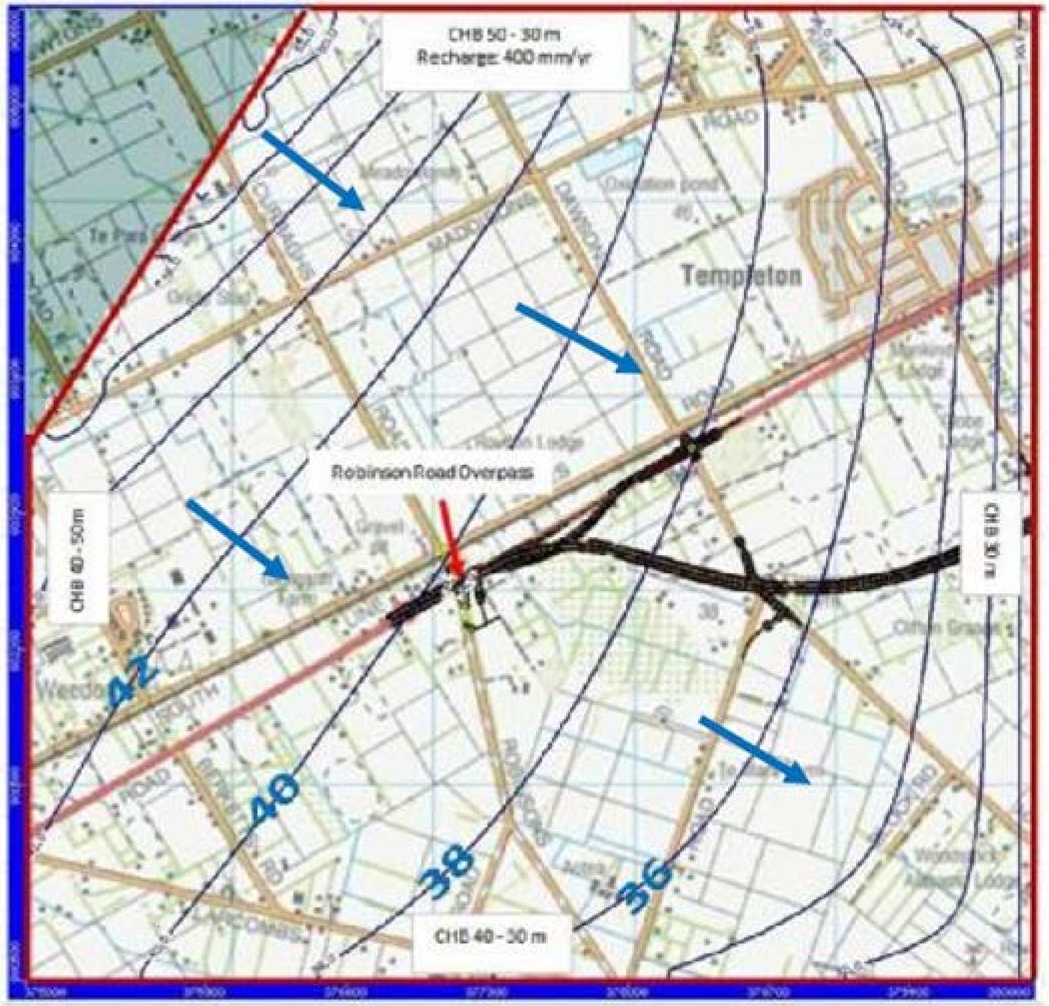
Water quality in Aquifer 1 has the potential to be affected by infiltrated stormwater along the project alignment. Wells within 500 m of the Project have been identified by GHD. The potential effects on groundwater quality and the wells in the Project vicinity are addressed in a separate report, Assessment of Groundwater Quality Effects - Christchurch Southern Motorway Stage 2 (CSM2) and Main South Road Four Laning (MSRFL), (Beca, 2012).

6.2 Robinsons Road Model

6.2.1 Model Set-Up

The Robinsons Road model was developed using Visual MODFLOW to assess potential mounding that would occur during infiltration of the 24-hour, 100-year rainfall event and pumping rates that would be likely to maintain groundwater levels 600 mm below roadway surface during high water level events. The three-layer model was developed from the regional steady-state model. Flow directions and heads (water levels) from the regional model were used to define a sub-model region approximately 5 km x 5 km centred on Robinson Road Overpass. The model grid was refined to 2 m x 2 m around the Robinson Road Overpass graduating out to cells 100 m x 100 m at the model edges. The model included an infiltration structure with an area of approximately 30 m x 60 m and a depth of 0.5 m below the lowest point of the road (39.5 mRL). Constant-head boundaries were assigned according to the water levels calculated from the regional model (with the CPW effects) as shown in Figure 8.

Figure 8 Calibrated Steady-state RRO Model Water Level Contours (mRL)



The RRO area is underlain by sandy gravels and cobbly sandy gravels of the Halkett Member of the Springston Formation with varying silt content, underlain by similar materials of the Burnham Formation, such that all three layers were modelled with a hydraulic conductivity of 1.2×10^{-3} m/s. Specific yields were estimated according to material type and typical values given in Johnson (1967). Rainfall recharge of 58 % of precipitation was applied at the surface layer modelled as sandy gravel. This rate is equivalent to 400 mm/yr based on an average annual rainfall of 680 mm/yr (Halswell Ryan's Bridge gauge). Calibration was achieved to generally replicate the groundwater levels and flow directions of the regional model simulating the maximum high water levels. These levels were adjusted to include the rise predicted by CPW and then reduced to allow for a water level of 39.6 mRL at RRO (as indicated in the more-detailed water level analysis). The calibrated model results are shown in Figure 8. Note that contours near the northeast and southeast edges of the model curve reflecting an "edge effect" of the model that is not representative of actual flow direction changes.

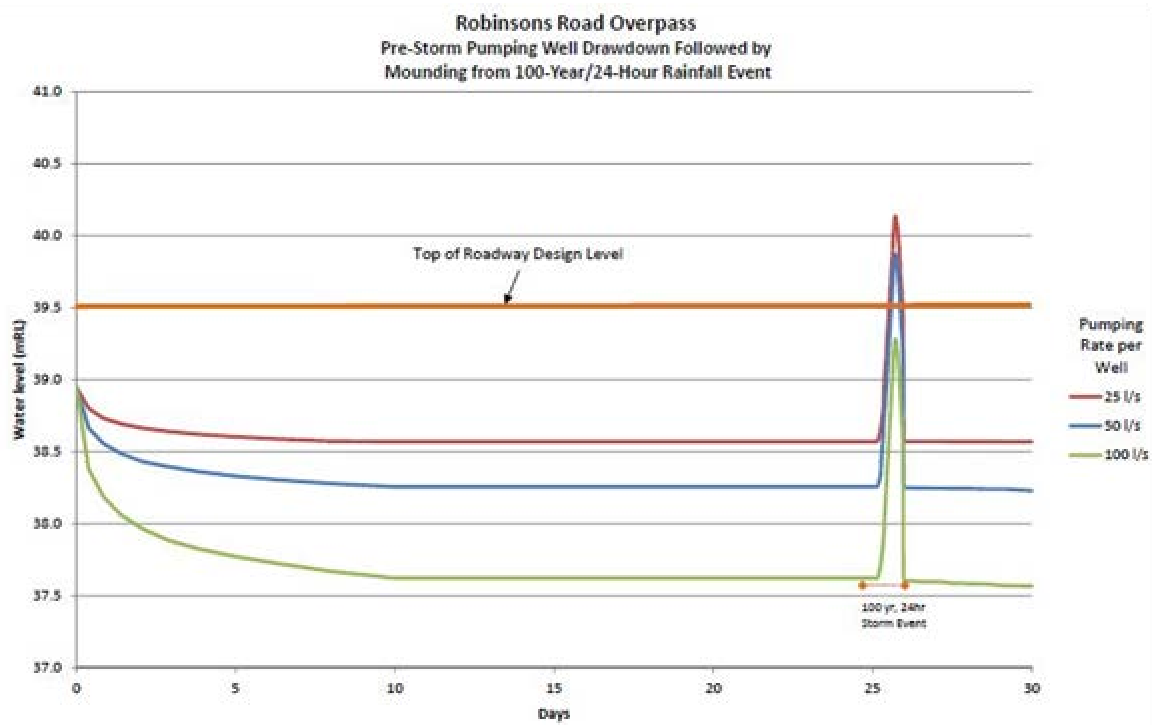
6.2.2 Pumping Rates to Limit Water Level Rise to 600 mm below the Roadway

Pumping simulations were modelled to assess possible abstraction rates that would be needed to keep the water level 600 mm below the roadway of Robinsons Road. Pumping was modelled using two wells, one at the northwest and one at the southeast end of the RRO infiltration structure. The simulation wells were screened over depths of 15 m to 45 m (equivalent to 30 to 0 mRL). The depth range was selected to allow pumping at all possible rates without concern about drawing the water level too deep in the simulation. Pumping rates from 25 to 100 L/s per well were modelled starting with an initial water level at 39.6 mRL. The wells were allowed to pump for 25 days to bring water levels down to a quasi-steady state and then the 24-hour, 100-year precipitation event was applied using runoff rates as calculated and supplied by GHD (T. Miller, 2012, pers. comm. on stormwater generation rates). Simulated pumping continued through the 100-year event to the end of the 30-day simulation by which time the rise in water levels beneath the roadway had returned to the pre-storm event level. Pumped water at rates of up to 10 L/s would be diverted to an infiltration pond 350 m southwest of RRO. Pumping at greater rates of up to 100 L/s would be diverted to a stockwater race along Robinsons Road. The hydrograph for water levels directly beneath the low point of Robinsons Road is shown in Figure 9. Note that the pumping rates are “per well” so the total pumping is twice as large.

Figure 9 shows that total pumping to lower the water level beneath the RRO during high groundwater events is predicted to be between 50 and 100 L/s. The actual pumping rate will depend on the hydraulic conductivity of the cobbly sandy gravel beneath the RRO. The value used in the analysis (1.3×10^{-3} m/s) is likely to be near the high end of the range of values and it is therefore not anticipated that it would be necessary to pump at rates higher than 100 L/s to achieve the required lowering of the groundwater level. Actual pumping rates will be assessed through testing of the wells when in place.

Figure 9 also shows that 25 days is unlikely to be necessary to lower the groundwater level to 600 mm below the roadway. A quasi-steady state (with most of the pumping induced drawdown) occurs within 6 to 10 days after pumping begins, with a majority of the drawdown occurring within the first two days.

Figure 9 Predicted Water Levels from Pumping Followed by 24-hour, 100-Year Rainfall



An assessment was made of drawdowns caused by pumping at 100 L/s for 25 days from beneath the RRO infiltration structure, with the parameter values used to predict the drawdowns over time at 50 L/s per well. The results of the analysis are shown in Figure 10.

Figure 10 Drawdown with pumping at 100 L/s under Maximum High Groundwater Conditions

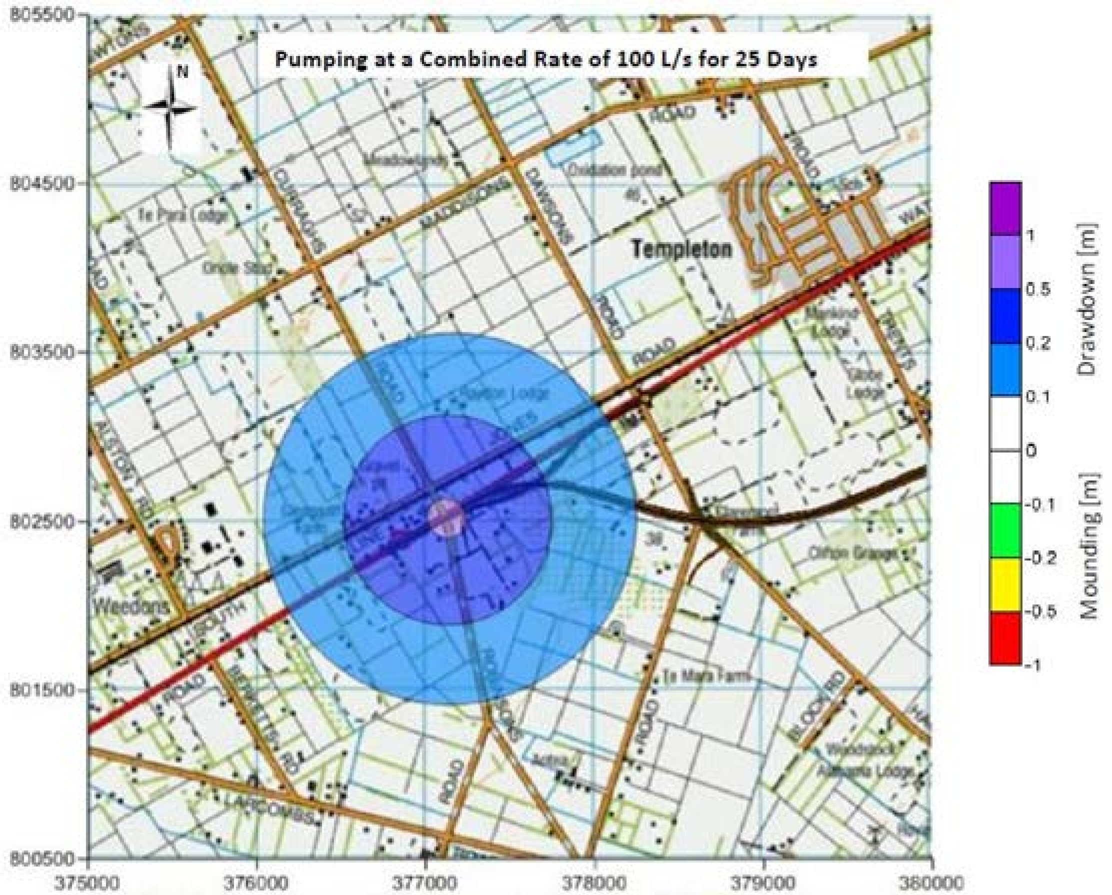
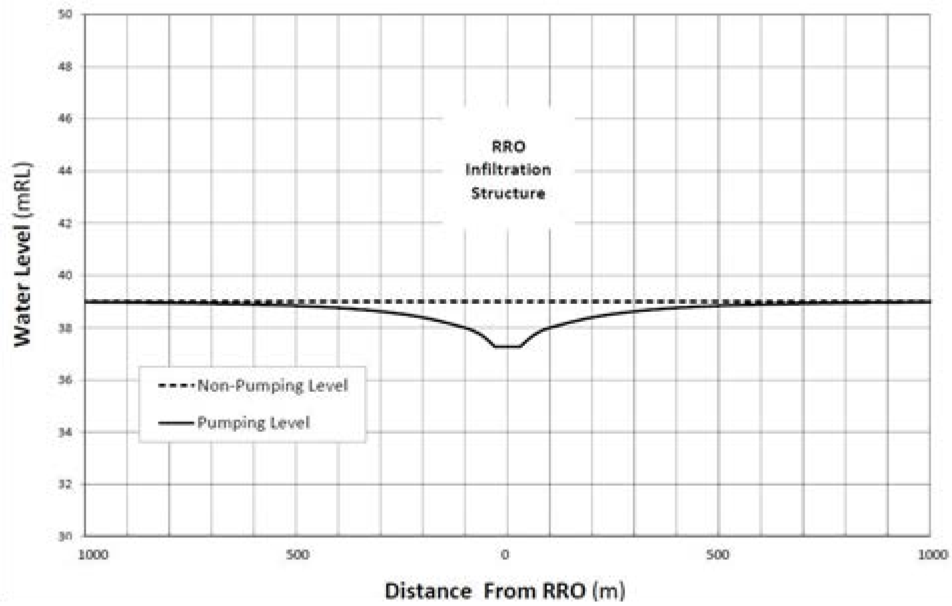


Figure 10 shows that a measurable drawdown of 100 mm could occur at a distance of 1 km with a drawdown of almost 0.5 m at a distance of 100 m from the RRO facility. Such drawdown would only occur during periods of maximum high water when the dewatering wells would be in operation.

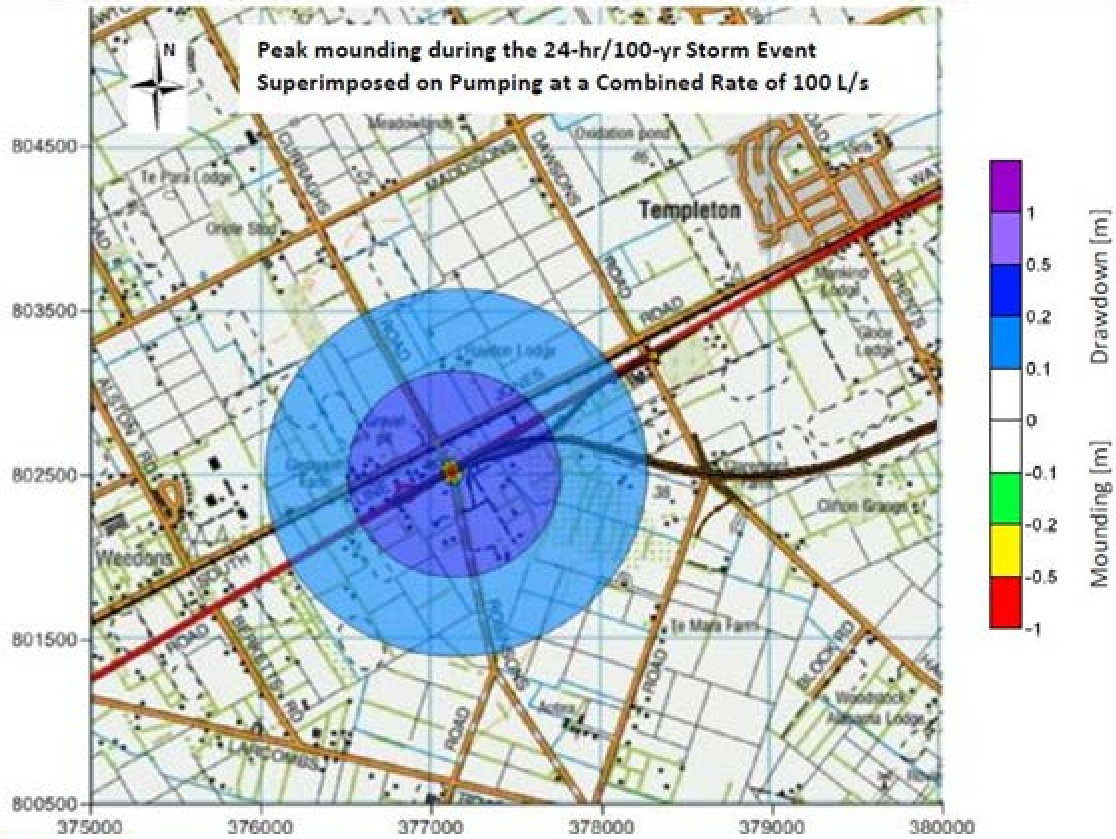
Figure 11 Drawdown with pumping at 100 L/s from Beneath RRO



A 2-D (plan view) analytical model was used to corroborate the results of the 3-D modelling. This model based on the method of Theis (1935) was used to assess drawdown assuming an aquifer thickness of 20 m, hydraulic conductivity of 1.2×10^{-3} m/s and a specific yield of 0.2, and a pumping rate of 100 L/s. The results are comparable to the 3-D modelling as shown in Figure 11. Details of the analysis are given in Appendix B.

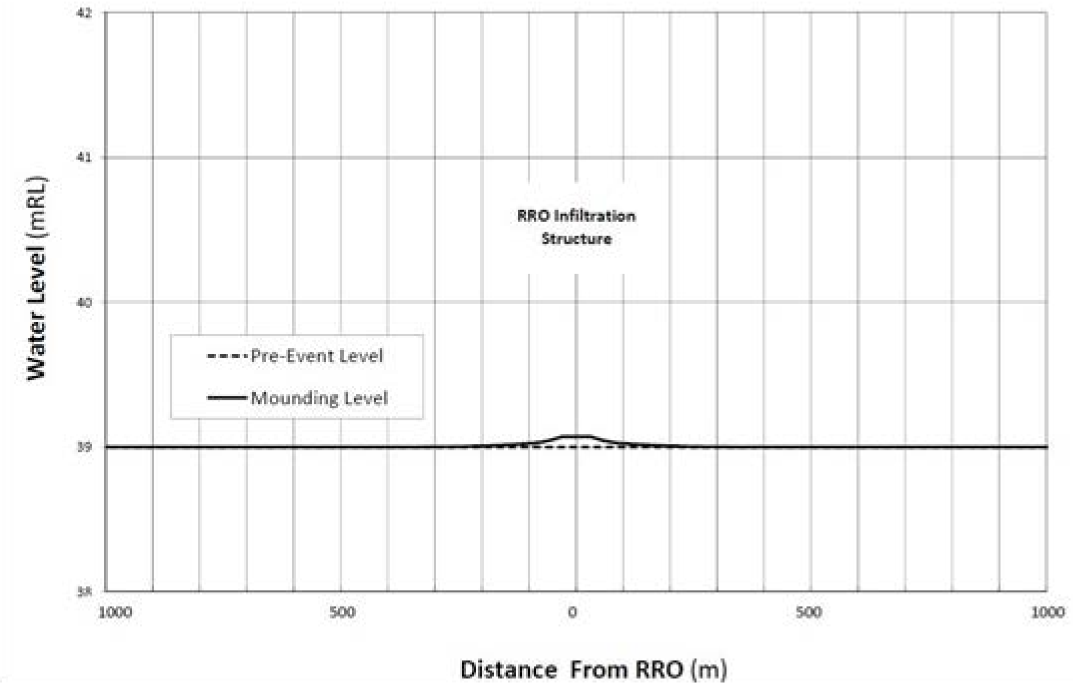
The combined effects of pre-storm event pumping drawdown followed by the mounding of the water table caused by the 24-hour/100-year precipitation were assessed using the 3-D RRO model. Figure 12 presents drawdown and mounding contours at the peak of the storm event. The contours indicate the difference between the water level before pumping began (39.6 mRL beneath RRO) at a rate of 100 L/s and the level of the water table at the time of peak runoff discharge to the infiltration structure beneath Robinsons Road. The figure shows that mounding effects are confined to the immediate vicinity of the RRO with the effects of pre-storm-event pumping causing an overall reduction of the groundwater levels below the predicted maximum levels with CPW away from RRO.

Figure 12 Mounding at Peak Storm Water Infiltration Combined with Drawdown from pumping at 100 L/s under Maximum High Groundwater Conditions



The 2-D analytical analysis of the mounding effects alone (no pre-event pumping) with an assumed water table level beneath RRO of 39 mRL indicates a relatively small water level rise. The groundwater mounding in Aquifer 1 from this 100-year event (Figure 13), is about 25 mm at a distance of 100 m from RRO with no measurable effects beyond a distance of 250 m from RRO at the end of the 24-hour event. Appendix B presents details of the analysis.

Figure 13 Mounding in Aquifer 1 Caused by the 24-hour, 100-Year Rainfall Event



6.3 Halswell Junction Road Model

6.3.1 Model Set-Up

The Halswell Junction Road model was developed using Visual MODFLOW to assess drainage rates needed to maintain groundwater levels below a design level of 18.75 m. The three-layer steady-state model was developed from the regional steady-state model. Flow directions and heads (water levels) from the regional model were used to define a sub-model region of approximately 2 km x 2.7 km centred on Halswell Junction Road. The model grid was refined to 1.5 m x 2.5 m around the HJR graduating out to 45 m x 75 m. MODFLOW's Drain boundary function was used to model Knights Stream to the southeast of the Project. Drain boundaries were also used to model the proposed drains beneath Maize Maze, Owaka, Ramp 1 and Ramp 2 ponds. The drains beneath the ponds were assigned an invert level of 18.75 m. Constant-head boundaries were assigned based on the water levels calculated from the regional model as shown in Figure 6.

The HJR area is underlain at the surface by the silts and clays with fine sand of the Yaldhurst Member underlain by the sandy gravel and sandy silty gravel of the Halkett Member (both of the Springston Formation) such that the upper layer was modelled with a hydraulic conductivity of 3 x 10^{-6} m/s while the lower two layers were modelled with a hydraulic conductivity ranging from 3 x 10^{-5} to 3 x 10^{-3} m/s, to assess the effects of varying silt content observed at different locations beneath CSM1 and CSM2. A vertical anisotropy (ratio of Kv/Kh) of 0.1 was assumed to account for the variable presence of leaky zones of silt and clay within the sand and gravel unit. Rainfall recharge of 36 % of precipitation was applied at the surface layer modelled as silts and clays with fine sand. This rate is equivalent to 250 mm/yr based on an average annual rainfall of 680 mm/yr (Halswell Ryans Bridge gauge). The model is bound by Constant Head Boundaries (CHB). Calibration was achieved to generally replicate the groundwater levels and flow directions of the regional model simulating the maximum high water levels that include the rise predicted by CPW with adjustment to allow for a water level of 19.4 mRL as indicated in the more detailed water level analysis (Table 4). The calibrated model results are shown in Figure 14.

Figure 14 Calibrated Steady-state HJR Model Water Level Contours.

6.3.2 Drainage Rates to Limit the Maximum Water Level to 18.75 mRL

Drainage simulations were modelled to assess drainage rates that would occur when the groundwater level rises to its predicted maximum level of 19.4 mRL below the Maize-Maze pond (with slightly lower levels predicted beneath the Owaka Basin and the Ramp Ponds). The purpose of these under-drains is to keep groundwater from rising into the storage and/or infiltration ponds to be constructed at the HJR intersection. Drainage would only occur when the water table rises above 18.75 mRL with a planned gravity discharge to Upper Knights Stream. During other times, the water table would lie below the bottom of the drains and no water could reach the invert outlet of the drains. Groundwater would continue to flow toward Upper Knights Stream, its local discharge point.

Table 7 shows that total drainage to maintain the water level beneath the Maize-Maze pond, the Owaka Basin and the Ramp Ponds during high groundwater events is predicted to be less than 50 L/s. The table presents the drainage rates based on two values of hydraulic conductivity representing a range of likely values of the Halkett Member beneath the HJR vicinity.

Table 7 Halswell Junction Road Drainage

Pond	Layer 2 K = 3.5x10^{-3}		Layer 2 K = 3.5x10^{-5}	
	Inflow to Drain (L/s)	Drawdown (m)	Inflow to Drain (L/s)	Drawdown (m)
Maize Maze	21.0	0.67	4.5	0.51
Owaka	11.1	0.53	2.7	0.44
Ramp 1	-	0.45	-	0.38
Ramp 2	-	0.42	-	0.35

The model indicates no discharge from under-drains modelled beneath the Ramp Ponds. This lack of discharge appears to result from the Owaka Basin and Maize Maze Ponds being situated up-gradient, therefore collecting the groundwater and drawing down the level lower than the drains at Ramp 1 and Ramp 2. Placement of drains beneath the Ramp Ponds at a slightly lower elevation (18.5 mRL) would likely allow drainage from beneath these ponds at rates similar to those of the Maize-Maze and Owaka drains. Figure 15 presents the drawdown contours caused by the pond under-drains. The drawdown (effect) is the difference between the predicted maximum water level with CWPES and the water levels after the drains have caused the water table to stabilise (steady-state conditions).

Figure 15 Drawdown beneath HJR Intersection Caused by Under-Drains at 18.75 mRL under Maximum High Groundwater Conditions

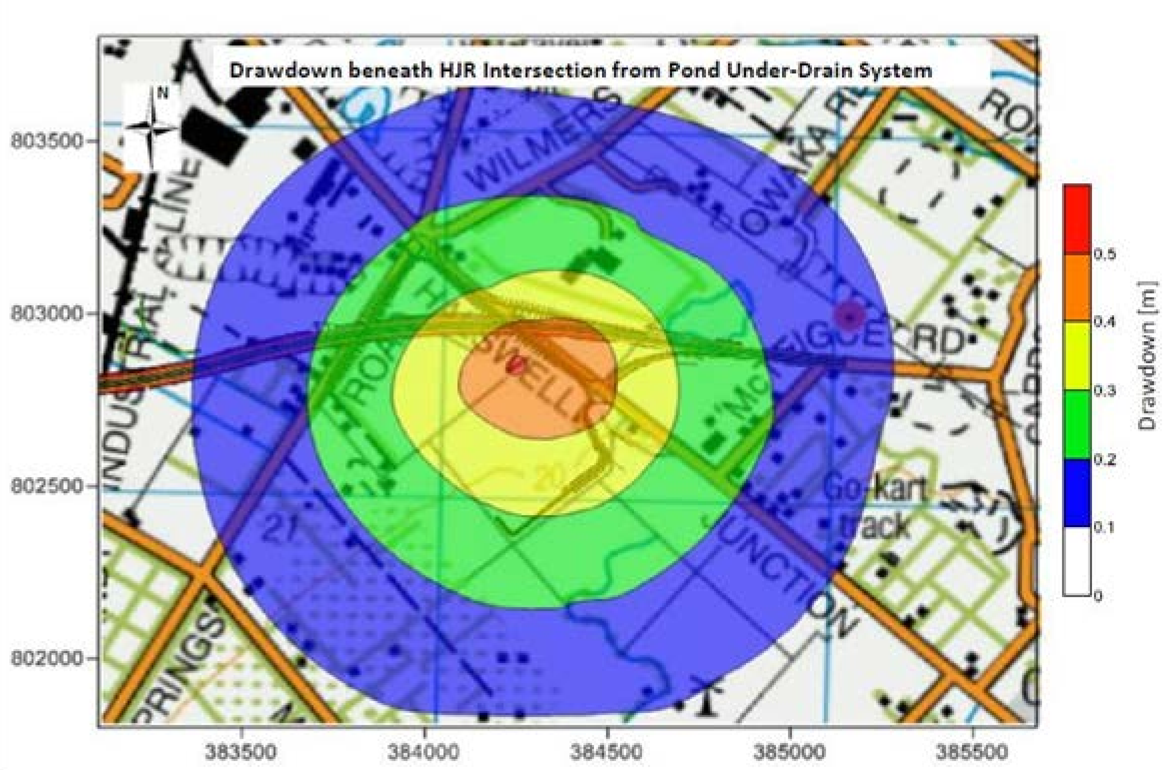


Figure 15 shows that the reduction of the maximum high groundwater levels with CPW is greater than 0.5 m beneath the Maize-Maze pond with a drawdown of about 100 mm at a distance of 1 km. These effects, however, are drawdowns below an elevated groundwater level that has not yet occurred and is therefore not a reduction beneath groundwater levels representing current conditions.

7. Assessment of Environmental Effects

7.1 Water Level Rises

The infiltration of stormwater is predicted to cause small water level rises in Aquifer 1. The rise beneath the RRO is expected to be in the order of 1.5 m directly beneath the structure. When groundwater levels are near their maximum predicted high of 39.6 mRL, this rise could lead to short-term flooding of the roadway (Figure 9). The model indicates that with pumping used to maintain the groundwater level below the base of the infiltration structure beneath Robinsons Road, flooding may be eliminated or may only last for a few hours. Without pumping, the roadway would remain flooded for an undetermined period as the duration and the recurrence interval of the water levels above the low point of Robinsons Road (39.5 mRL) under the CPW cannot be predicted from the existing data. Statistically, such a rise would only occur for less than 5 % of the time and more likely closer to 1 % of the time. However, it is not known when such rises would occur, especially as climatic parameters change over time.

Rises in water levels in Aquifer 1 from the 24-hour, 100 year rainfall event are expected to be much smaller away from the RRO. A rise (mounding of the water table) of about 25 mm is modelled 100 m from the RRO infiltration structure with no measurable mounding at distances greater than 250 m. Seasonal variations reported from ECan wells in the area are typically 2 m to 6 m. The large seasonal variations would mask rises caused by infiltrated stormwater. The effects of stormwater infiltration on groundwater levels are considered to be less than minor.

Water level rises beneath the HJR interchange are expected to be small because of the under-drain system planned for construction beneath the Maize-Maze, and Ramp Ponds and the Owaka Basin. The relatively low hydraulic conductivity of the surficial deposits beneath HJR will limit the ability of the ponds to infiltrate stored stormwater to the underlying Aquifer 1. As such the primary purpose of the ponds will be storage to limit peak discharge.

The ponds and basin will only infiltrate at low rates as discussed in Beca (2011). The results of the modelling of infiltration at Mushroom Pond (on the north side of HJR and overlying surficial deposits similar to those beneath the planned HJR structures) would cause the groundwater beneath the pond to rise by 1.4 m to 2.6 m in the absence of an under-drain. The modelling also indicates that it may take up to two weeks for the pond to fully drain without intervention in the form of pumping, gravity drainage or an under-drain system. Similar mounding and drainage rates are expected from the ponds to be constructed for CSM2 at the HJR interchange. However, such rises under high water level conditions would cause groundwater to rise above the bases of the ponds, reducing storage capacity and may cause lifting of pond liners (where these occur). The under-drain system proposed to limit the maximum water level rises beneath HJR will both assist in limiting mounding in Aquifer 1 and help to maintain the full storage function of the ponds. Seasonal variations in groundwater levels recorded in ECan wells range from 2 m to 6 m. Such variations would mask local mounding effects. The effects of mounding beneath the HJR facilities on groundwater are therefore considered to be less than minor.

7.2 Reduction of Maximum Water Levels

Pumping at RRO and gravity drainage from the under-drains at HJR will only occur when groundwater levels are 1.3 to 2.5 m higher than they have been in the past. Reductions caused by pumping or drainage will not affect any existing groundwater users because reductions would not lower groundwater below current levels or even the highest levels of the past. Only higher groundwater levels that might occur in the future would be lowered.

Surface water will be little affected as the discharge from the gravity drainage system will be directed to Knights Stream, its local discharge point without the dewatering system.

Pumping from beneath the RRO facility would be returned to surface and/or groundwater. Infiltration of the water pumped from beneath RRO would be discharged to a combination of a surface infiltration pond located about 350 m to the southwest of RRO and an infiltration trench (or soakaway) installed adjacent to Robinsons Road between 300 m and 800 m from the CSM2 crossing or directly to the adjacent stock water race. Nearby wells would also not be affected because if such a well was to pump at a rate high enough to lower levels at Robinsons Road, pumping from the RRO system would cease and allow the nearby well to pump at its consented rate. Therefore, the effects of water level limitation at the RRO and HJR facilities are considered to be less than minor.

7.3 Water Quality

Water quality in Aquifer 1 has the potential to be affected by infiltrated stormwater along the Project, if the stormwater contains contaminants. Contamination in the groundwater from the Project has been assessed through contaminant transport modelling and is presented in a separate report, Assessment of Groundwater Quality Effects - Christchurch Southern Motorway Stage 2 (CSM2) and Main South Road Four Laning (MSRFL), (Beca, 2012).

8. Options for Mitigation

Two design options have been identified to mitigate or avoid effects on groundwater. Details of RRO and HJR are given below. Alternatives considered but not selected for the Project are summarised after the preferred options.

8.1 Monitoring

Monitoring of water levels in a selection of existing wells and specifically installed piezometers in proximity to Robinsons Road and HJR will allow the indicated effects to be confirmed and provide a check that pumping occurs only as required.

8.2 Design

The risk of water levels rising above the maximum predicted levels as indicated in our assessment can be reduced to the new target maximum levels by a groundwater intervention strategy to intercept the groundwater and to discharge this groundwater away from the facilities and outside the zone of influence. This groundwater level intervention can be achieved through design at RRO and HJR.

8.2.1 Limiting maximum water levels beneath Robinsons Road

Limiting groundwater levels beneath the RRO through a combination of shallow wells, drainage pumping and re-infiltration is a planned mitigation. The effects of two wells constructed near the ends of Robinsons Road where it passes beneath CSM2 have been assessed. The wells could be constructed as part of the RRO. The actual quantities and rates of water to be pumped and re-infiltrated would be assessed as part of testing of the wells during their construction. The analysis discussed above indicates a likely pumping rate of less than 50 L/s from each of two wells, 300 mm in diameter completed to an estimated depth of 20 m.

Pumped water would be directed to a stockwater race along Robinsons Road. Field inspection of the stockwater race indicates that the bottom is coated with clays and fines that have settled out from the water carried by the race. The clays and fines would limit seepage such that the additional water introduced to the stockwater race is unlikely to result in a significant increase in seepage from the race to the groundwater system.

8.2.2 Limiting maximum water levels beneath HJR

Limiting water levels beneath HJR through the construction of under-drains (or wells) and gravity drainage to Upper Knights Stream is a planned mitigation. A horizontal, gravel-and-perforated-pipe under-drain system (or alternatively, four or more wells, 100 to 150 mm in diameter drilled to an estimated depth of 10 m deep) could be constructed beneath each pond to assist in intercepting the rising groundwater in these lower-permeability soils. The outlet level of the drains or elevation of the top of each well would be set by maximum water level desired beneath the ponds. In our analysis we used an elevation of 18.75 mRL, however, lower levels appear feasible. The under-drains or wells would be connected to a manifold sloping toward a discharge point into Upper Knights Stream. The sloping manifold would allow groundwater to be discharged under gravity without the need for pumping. (A pump could be used to increased flow and/or to flush the discharge lines.)

The removal of groundwater by gravity drainage through a manifold system would not affect any existing groundwater user because it would not lower groundwater below current levels. Only higher groundwater levels that might occur in the future would be lowered through this self-limiting system. Future groundwater users would also not be limited by this set up. The drawdown “cone of depression” of the water table induced by any well pumping hard enough, would lower the water levels beneath the ponds meaning that the gravity drainage would cease and the aquifer would respond as if the under-drains or dewatering wells did not exist. In a similar manner, surface water would be little affected as the discharge from the gravity drainage system would be directed to Upper Knights Stream, its natural discharge point without the dewatering system.

8.3 Mitigation Alternatives Considered but Not Recommended

The following alternatives to the above mitigations have also been considered.

8.3.1 Direct more runoff to surface water and less to groundwater at HJR

The stormwater system could be designed to discharge more runoff directly to surface water with less directed to groundwater via pond seepage. Additional storage facilities would be required to offset the reduced volume of storage in the unlined ponds caused by high groundwater levels above the pond floors. Additional discharge to surface water (via surface raceways or subsurface pipe lines eventually to Upper Knights Stream) would be needed to offset the reduced infiltration rates from the unlined ponds where gravity drainage would be significantly curtailed by groundwater levels above the pond floors. In addition, the lined ponds would have to be redesigned to allow for groundwater levels above pond floors to reduce the risk of liner lifting.

8.3.2 Raise CSM2 at HJR

Raising CSM2 by 1 m to 2 m would allow for construction of the unlined ponds at HJR to be raised by a corresponding amount allowing for a greater depth to water. Although this alternative may represent the highest cost of the listed options, it may allow for successful operation of the stormwater management system with a minimum of operational costs and pond storage volumes would not be limited by high groundwater levels.

8.3.3 Raising the level of Robinson Road beneath the RRO

Raising the level of the low point of Robinsons Road beneath the overpass by 1 m to 2 m would allow for water levels to be higher than those assessed with less risk of flooding. The utility of the road would be reduced however, as taller vehicles could not use the road if over height. The roadway would be available for use by the lower-height vehicles during wet periods when a deeper roadway would be flooded.

8.3.4 Raise CSM2 above RRO

Raising CSM2 by 1 to 2 m would allow Robinsons Road to be raised by a corresponding amount without the limitation of the lower clearance described above.

8.3.5 Allow Robinsons Road to Flood

Building the RRO and CSM2 as planned may result in flooding of Robinsons Road when groundwater levels are high and large rainfall events occur. The depth to water, recurrence interval and duration of such flooding events cannot be accurately predicted because of the uncertainty discussed above. However, the water level assessment indicates that water levels are likely to be above 37.4 mRL 5 % of the time and above 36.3 mRL10 % of the time. The duration of the flooding could be better assessed when in-situ constant-rate permeability testing has been conducted within the soils planned for infiltration trench (soakaway) construction beneath Robinsons Road at the detailed design stage.

9. Conclusions

This assessment of groundwater effects indicates that with the proposed monitoring and design features, the effects on groundwater levels caused by the construction and operation of CSM2 and the assessed portions of MSRFL will be less than minor. Water level rises will be controlled through pumping and under-drains. Disposal of the removed groundwater will be through diversion to surface water. The changes in water levels away from the Project caused by the proposed groundwater level control systems and the infiltration of stormwater are predicted to be much smaller than the natural variations in groundwater levels and the effects are considered to be less than minor.

The high permeability of the soils beneath the proposed carriageway of Robinsons Road where it will pass beneath CSM2 will allow for infiltration of stormwater falling on Robinsons Road. Under the conditions of maximum groundwater levels that include the effects of the CPW, the predicted groundwater levels would rise above the low points of Robinsons Road were no groundwater level control system to be implemented. Pumping of groundwater when it rises to a level near the base of the infiltration trench (soakaway) beneath the roadway before a major storm event, would significantly reduce or eliminate the flooding of Robinsons Road. Pumping would occur when groundwater levels rose to within 1 m beneath the roadway.

The lower permeability of the surficial soils will not allow for rapid infiltration and quick drainage of stormwater from the ponds proposed for the Halswell Junction Road interchange. The proposed under-drain system for this portion of the Project is designed to keep groundwater from rising above the bottom of the storage-ponds, keeping pond liners from lifting and helping to maintain full pond volumes for storage. The under-drain system will also help to drain the proposed infiltration-ponds and basin by removing infiltrated stormwater and draining it to Upper Knights Stream.

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Appendix A

3-D MODFLOW Groundwater Flow Models

Regional Model

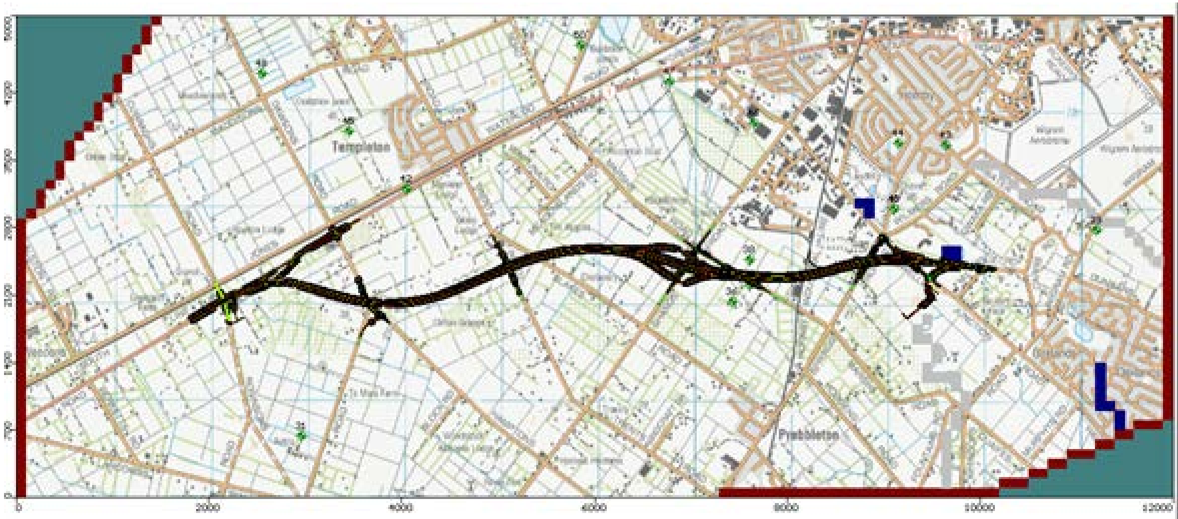
Purpose

A regional steady-state 3-dimensional groundwater flow model was developed to help define groundwater flow conditions in the greater vicinity of CSM2 and identify down-gradient locations for assessment of water quality effects, and to allow the set up of two sub-models (RRO and HJR). The model was set up using the graphical interface Visual MODFLOW 2010 Pro (Schlumberger, 2010).

Model Set Up

The model was set up with two layers consisting of cells with constant dimensions (in plan view) of 100 m x 100 m. The modelled region was 5 km (N – S) by 12 km (W – E), with a base elevation of 40 mRL, centred about CSM2. Figure A1 shows the model set up.

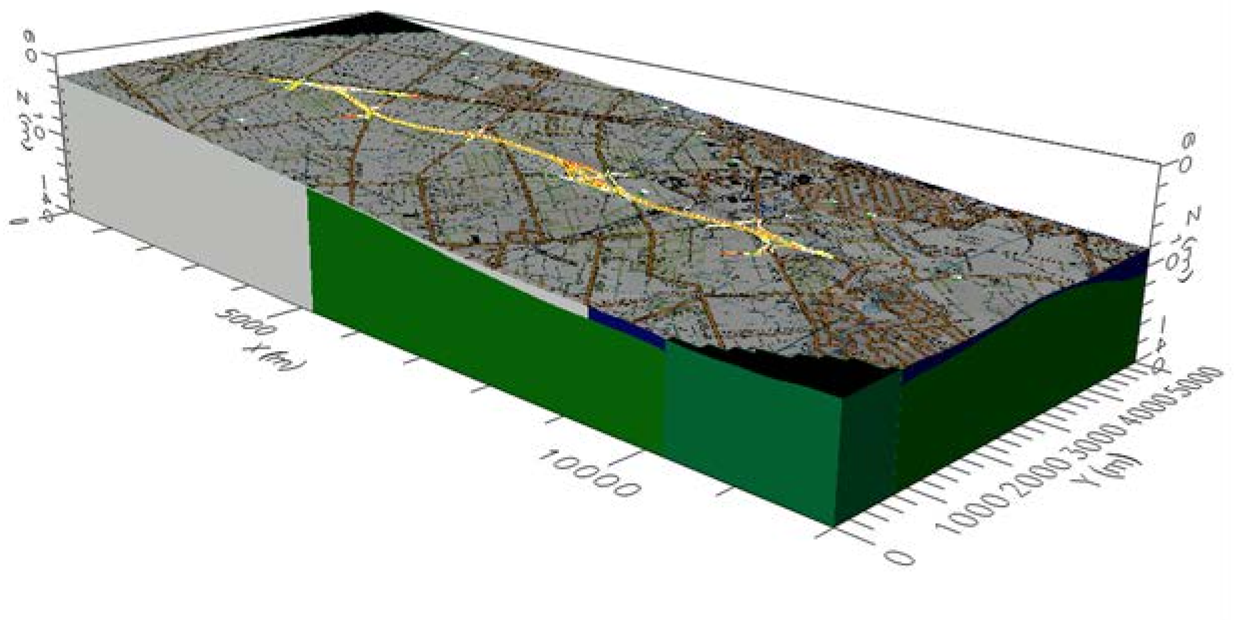
Figure A1 Regional Model Set-Up Showing Inactive Cells, Constant-Head Boundaries Rivers and Drains



Distribution of Hydrogeological Units

Hydrogeological units were assessed based on investigation bores along the alignment and bore logs available from ECan. The distribution of the subsurface materials, deposits of the Yaldhurst and Halkett Members of the Springston Formation and the Burnham Formation, was based on our interpretation of the information presented in the bore logs. A simple geological profile was modelled comprising 3 m of sandy gravel with silt overlying sandy gravel. The model included a 1 m thick low-permeability layer of clay, silt and sand present at the surface on the eastern end of the modelled region. The various units were grouped into four categories based on grain size, resulting in a two layer model with zones reflecting the varying permeabilities of the units. Figure A2 shows a cross-section of the model reflecting the different zones.

Figure A2 Model Set-Up Showing Layering and Hydraulic Conductivity Zones



Hydraulic Parameters

Hydraulic properties were assigned to each layer, assessed from a combination of values used in calibrated regional models developed by Aqualinc (2006 and 2007), field testing done as part of the CSM 1 and 2 investigations and Beca data on similar materials from other parts of Canterbury. The initial values were adjusted through the calibration process with the adjusted values presented in Table A1.

Table A1 – Hydraulic Parameters in the Regional Model

Material Type	Colour on Section	Model Layer	Hydraulic Conductivity	
			Kh (m/s)	Kv/Kh
Clay, silt and sand (confining layer)		1	1.0×10^{-6}	0.1
Sand and gravel with silt		1,2	1.2×10^{-3}	0.1
Sand and gravel		1,2	3.5×10^{-3}	0.1
Sand and gravel (paleo-river channel)		1,2	8.0×10^{-3}	0.1

Model Boundaries

Model boundaries are included in Figure A1.

Edge boundaries: The model was set up with constant head boundaries placed to replicate the highest groundwater level contour map (Figure 5 in the main body of the report). These levels reflect the maximum groundwater level in Aquifer 1 as calculated from the high water levels from wells in the ECan database plus the predicted effects of CPW. The constant head boundaries were set equivalent to the water level contours (Figure 5) where they intersected the lateral boundaries of the model. We have assumed no significant vertical flow gradients in the modelled area resulting in the same constant head value (equivalent to the intersecting water level contour value) assigned to all layers from top to bottom of the model at each edge boundary cell. The range of constant head values for the regional model is included in Figure A1.

Recharge Boundaries: Recharge at a rate of 58 % of precipitation was applied at the surface where sandy gravel are present; and at a rate of 36 % of rainfall where sand, silt gravel and clay are present. This equates to approximately 400 mm/yr and 250 mm/yr respectively based on an average annual rainfall of 680 mm/yr as calculated from Halswell Ryan's Bridge gauge data.

Drain Boundaries: The Drain boundary function was used to model Knights Stream to the southeast of the proposed motorway and Halswell Junction Road.

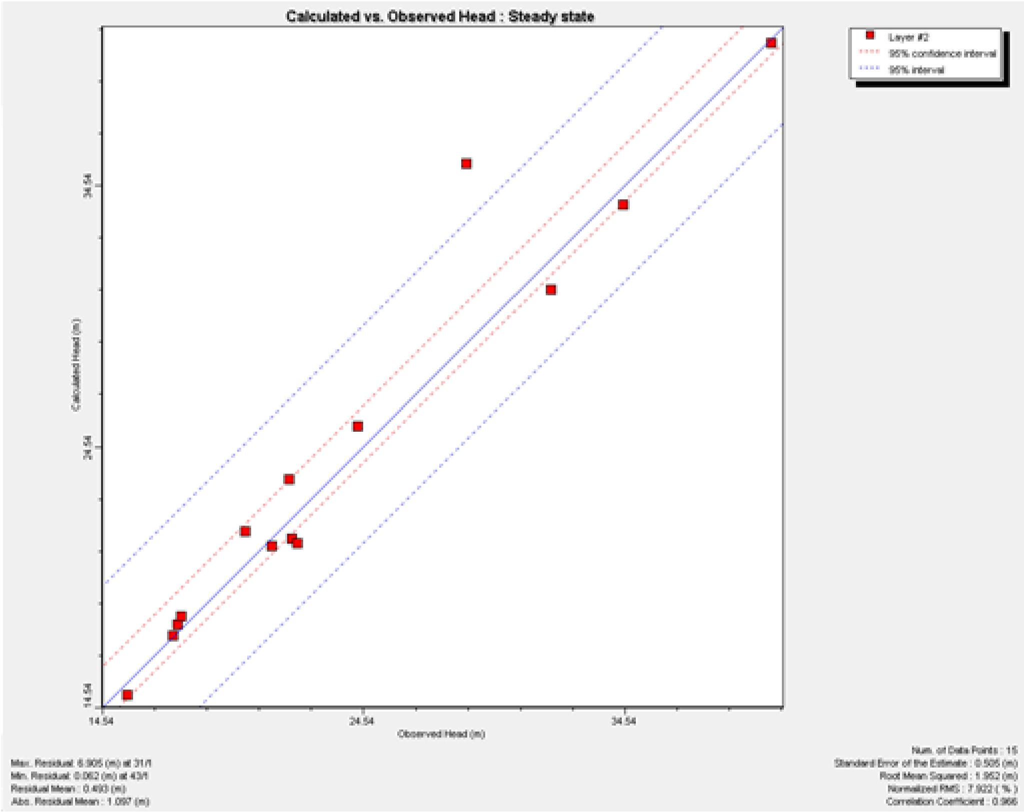
River Boundaries: The River boundary function was used to model the standing water bodies to the north of the proposed motorway. Stock water races were not modelled. A field reconnaissance of the area revealed that the bottoms of the stock-water races in the area are generally lined with settled silts and clays providing a low-permeability layer that impedes seepage losses.

Base boundaries: The bottom of the model is a no-flow boundary with no groundwater moving in or out of the model.

Calibration and Sensitivity Analyses

The model was calibrated to generate an acceptable match to the maximum high water levels indicated in the ECan database. Hydraulic conductivity values were adjusted within a range of ± one order of magnitude until the model-calculated water level contours were similar to those generated by Surfer using the ECan high water levels without the CPW predicted effects. The model calibrated with a normalised RMS error of 7.9 % which is less than the maximum normalised RMS error criterion of 10 %, indicating an acceptable calibration. A plot of ECan vs model-calculated water levels results for the wells near CSM2 is shown in Figure A3.

Figure A3 Model Calibration Plot with Statistics



The water budget statistics (Table A2) show that a mass-balance was reached during calibration. Inflow verses outflow discrepancies were 0 %.

Table A-2 Water Budget for the Regional Steady-State Model

Cumulative Volumes Report [m^3]	Rates for Time Step Report [m^3/day]
IN:	IN:
Storage = 0 [m^3]	Storage = 0 [m^3/day]
Constant Head = 258262384 [m^3]	Constant Head = 258262.3906 [m^3/day]
Wells = 0 [m^3]	Wells = 0 [m^3/day]
Drains = 0 [m^3]	Drains = 0 [m^3/day]
MNW = 0 [m^3]	MNW = 0 [m^3/day]
LAKE SEEPAGE = 0 [m^3]	LAKE SEEPAGE = 0 [m^3/day]
Recharge = 49941020 [m^3]	Recharge = 49941.0195 [m^3/day]
ET = 0 [m^3]	ET = 0 [m^3/day]
River Leakage = 138240 [m^3]	River Leakage = 138.24 [m^3/day]
Stream Leakage = 0 [m^3]	Stream Leakage = 0 [m^3/day]
General-Head = 0 [m^3]	General-Head = 0 [m^3/day]
Total IN = 308341632 [m^3]	Total IN = 308341.6562 [m^3/day]
OUT:	OUT:
Storage = 0 [m^3]	Storage = 0 [m^3/day]
Constant Head = 308335616 [m^3]	Constant Head = 308335.625 [m^3/day]
Wells = 0 [m^3]	Wells = 0 [m^3/day]
Drains = 6033.1436 [m^3]	Drains = 6.0331 [m^3/day]
MNW = 0 [m^3]	MNW = 0 [m^3/day]
LAKE SEEPAGE = 0 [m^3]	LAKE SEEPAGE = 0 [m^3/day]
Recharge = 0 [m^3]	Recharge = 0 [m^3/day]
ET = 0 [m^3]	ET = 0 [m^3/day]
River Leakage = 0 [m^3]	River Leakage = 0 [m^3/day]
Stream Leakage = 0 [m^3]	Stream Leakage = 0 [m^3/day]
General-Head = 0 [m^3]	General-Head = 0 [m^3/day]
Total OUT = 308341664 [m^3]	Total OUT = 308341.6562 [m^3/day]
IN - OUT = -32 [m^3]	IN - OUT = 0 [m^3/day]
Discrepancy = 0%	Discrepancy = 0%

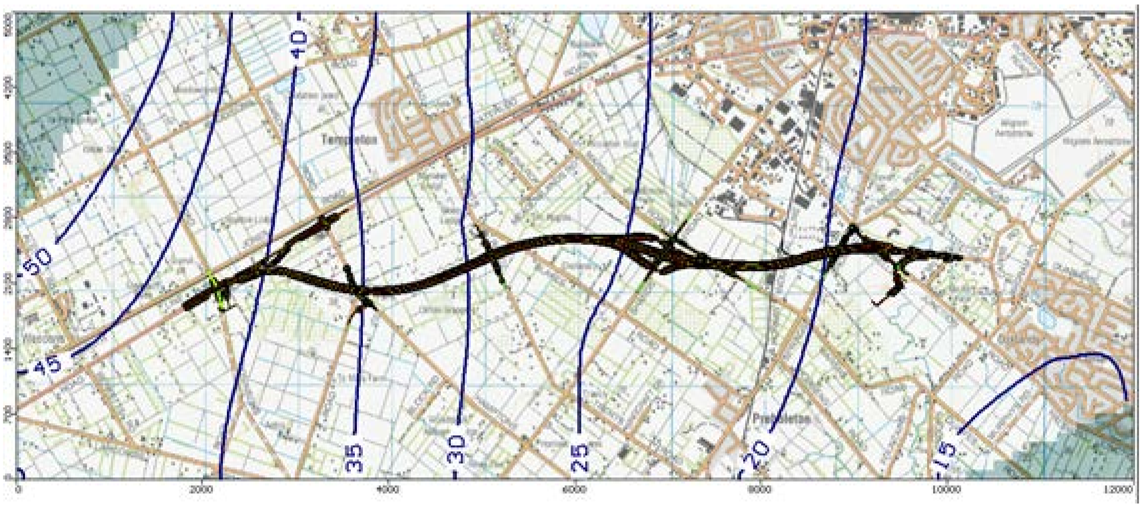
Sensitivity analyses were not conducted.

Model Results

The output of the model is presented in Figure A3 (without CPW) and A4 (with CPW).

Figure A3 - Regional Model without CPW - Steady State Contours

Figure A4 - Regional Model with CPW - Steady State Contours



Robinsons Road Model

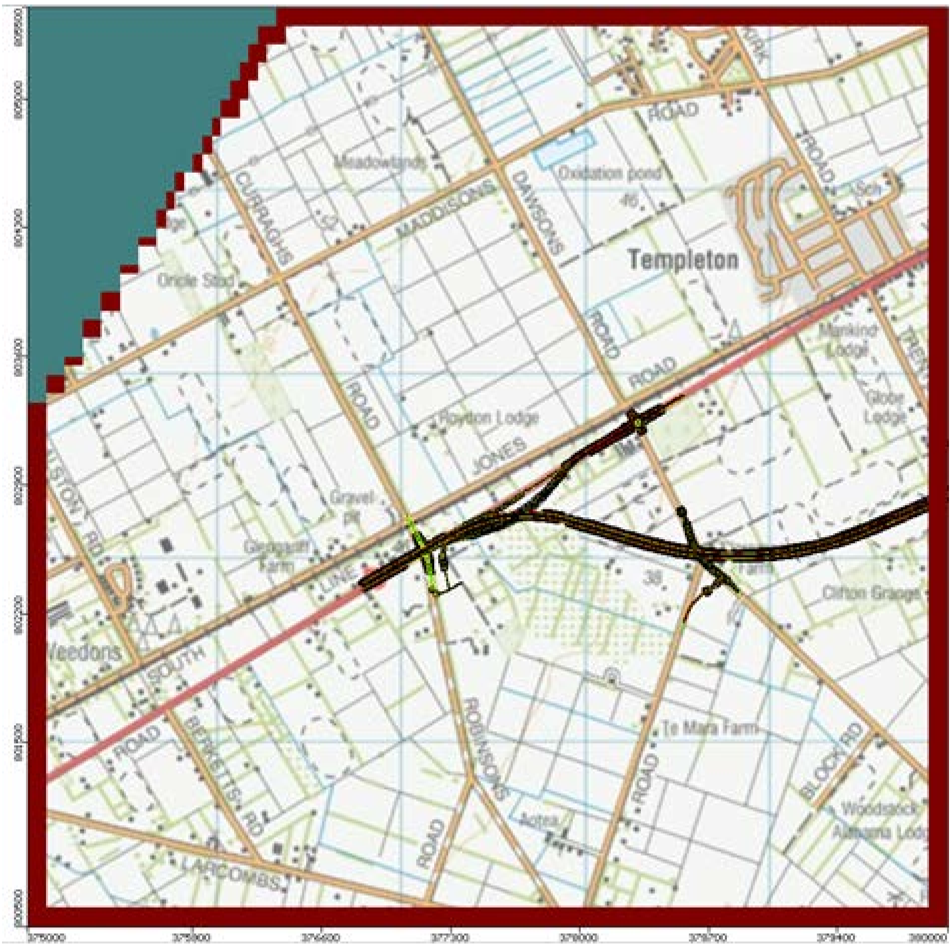
Purpose

The Robinsons Road model was developed using Visual MODFLOW 2010 to assess potential mounding that would occur during infiltration of the 24-hour, 100-year rainfall event and pumping rates that would be likely to maintain groundwater levels 600 mm below roadway surface during high water level events.

Model Set Up

The three-layer model was developed from the regional steady-state model. Flow directions and heads (water levels) from the regional model were used to define the sub-model region. The model was set up with cells 2 m x 2 m around the Robinson Road Overpass graduating out to cells 100 m x 100 m at the model boundaries. The sub-model region was approximately 5 km x 5 km centred on Robinson Road Overpass centred about CSM2 with a modelled base elevation of -40 mRL. Figure A5 shows the model set up.

Figure A5 - RRO Model Set-Up Showing Inactive Cells, Constant-Head Boundaries and Infiltration Structure

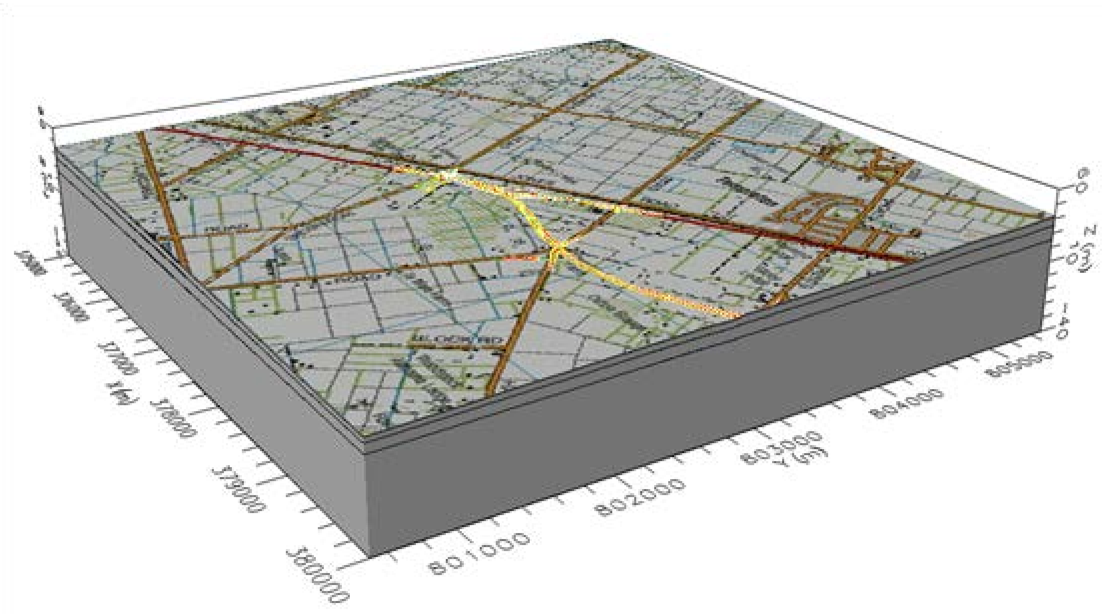


Boundaries Constant-Head Infiltration Structure Inactive Cells

Distribution of Hydrogeological Units

Hydrogeological units were assessed based on investigation bores along the alignment and bore logs available from ECan. The distribution of the subsurface materials, deposits of the Yaldhurst and Halkett Members of the Springston Formation as well as the Burnham Formation, was based on our interpretation of the information presented in the bore logs. A simple geological profile was modelled comprising a single unit of sandy gravels and cobbly sandy gravels with varying silt content of the Halkett Member of the Springston Formation and the Burnham Formation. Figure A6 shows a cross-section of the model.

Figure - A6 RRO Model Set-Up Showing Layering and Single Hydraulic Conductivity Zone



Hydraulic Parameters

The RRO area is underlain by sandy gravels and cobbly sandy gravels with varying silt and clay content such that all three layers were modelled with a single value for hydraulic conductivity of 1.2 x 10-3 m/s. The effects of leaky aquitards were addressed by assigning a vertical anisotropy to the sand and gravel deposits. Specific yields were estimated according to material type and typical values given in Johnson (1963). The values used in the model are shown in Table A3.

Table A3 - Hydraulic Parameters in the RRO Model

Material Type	Model Layer	Hydraulic Conductivity	Specific Yield	
		Kh (m/s)	Kv/Kh	[-]
Sand and gravel, cobbly sand and gravel with varying silt and clay	1,2,3	1.2×10^{-3}	0.1	0.20

Model Boundaries

Edge boundaries: The model was set up with constant head boundaries placed to replicate the highest groundwater levels calculated from the regional model with CPW as shown in Figure 8 (in the main body of the report). These levels reflect the maximum groundwater level in Aquifer 1 as calculated from the high water levels from wells in the ECan database plus the predicted effects of CPW. The constant head boundaries were set equivalent to the water level contours (Figure 6) where they intersected the lateral boundaries of the model. We have assumed no significant vertical flow gradients in the modelled area resulting in the same constant head value (equivalent to the intersecting water level contour value) assigned to all layers from top to bottom of the model at each edge boundary cell.

Recharge Boundaries: Recharge at a rate of 58 % of precipitation was applied at the surface as sandy gravels are representative of the entire surface of the sub-model. This equates to approximately 400 mm/yr based on an average annual rainfall of 680 mm/yr as calculated from Halswell Ryan's Bridge gauge data.

Infiltration Structure: The Recharge boundary function was used to model an infiltration structure with an area of approximately 30 m x 60 m with a depth of 0.5 m below the lowest point of the Robinsons Road (39.5 mRL).

River Boundaries: The River boundary function was not used in the sub-model. A field reconnaissance of the area revealed that the bottom of the stock-water races in the area are generally lined with settled silts and clays providing a low-permeability layer that impedes seepage losses.

Base boundaries: The bottom of the model is a no-flow boundary with no groundwater moving in or out of the model.

Calibration and Sensitivity Analyses

The model relied on the calibration of the regional model as there are no monitored well water levels within the modelled area. The hydraulic conductivity values were those within the range of the calibrated regional model.

The water budget statistics (Table A4) for the peak infiltration time step of the transient model (at about 6PM of the 24-hour, 100-year event that began at midnight) show that a mass-balance was reached (within 0.62 %) during this time step but that a small cumulative error or 4.5 % of water volume for the entire simulation up to that point. The simulation began with a 10 year period of no pumping and no infiltration to allow the model to reach a steady state prior to pumping began. The cumulative error was like from tiny mass-balance differences added up over the 10 years.

Table A4 - Water Budget for the RRO Transient Model

Cumulative Volumes Report [m^3]	Rates for Time Step Report [m^3/day]
IN:	IN:
Storage = 22533258 [m^3]	Storage = 4872.5723 [m^3/day]
Constant Head = 532862656 [m^3]	Constant Head = 150557.25 [m^3/day]
Wells = 0 [m^3]	Wells = 0 [m^3/day]
Drains = 0 [m^3]	Drains = 0 [m^3/day]
MNW = 0 [m^3]	MNW = 0 [m^3/day]
LAKE SEEPAGE = 0 [m^3]	LAKE SEEPAGE = 0 [m^3/day]
Recharge = 88002400 [m^3]	Recharge = 39336.3047 [m^3/day]
ET = 0 [m^3]	ET = 0 [m^3/day]
River Leakage = 0 [m^3]	River Leakage = 0 [m^3/day]
Stream Leakage = 0 [m^3]	Stream Leakage = 0 [m^3/day]
General-Head = 0 [m^3]	General-Head = 0 [m^3/day]
Total IN = 643398314 [m^3]	Total IN = 194766.127 [m^3/day]
OUT:	OUT:
Storage = 4555.729 [m^3]	Storage = 13922.9092 [m^3/day]
Constant Head = 672907456 [m^3]	Constant Head = 175327.3281 [m^3/day]
Wells = 111153.5625 [m^3]	Wells = 4320.0103 [m^3/day]
Drains = 0 [m^3]	Drains = 0 [m^3/day]
MNW = 0 [m^3]	MNW = 0 [m^3/day]
LAKE SEEPAGE = 0 [m^3]	LAKE SEEPAGE = 0 [m^3/day]
Recharge = 0 [m^3]	Recharge = 0 [m^3/day]
ET = 0 [m^3]	ET = 0 [m^3/day]
River Leakage = 0 [m^3]	River Leakage = 0 [m^3/day]
Stream Leakage = 0 [m^3]	Stream Leakage = 0 [m^3/day]
General-Head = 0 [m^3]	General-Head = 0 [m^3/day]
Total OUT = 673023165.2915 [m^3]	Total OUT = 193570.2476 [m^3/day]
IN - OUT = -29624851.2915 [m^3]	IN - OUT = 1195.8794 [m^3/day]
Discrepancy = -4.5%	Discrepancy = 0.62%

A sensitivity analysis of this Modflow model was not conducted.

Model Results

The output of the steady-state RRO model is presented in Figure A7.

Figure A7 Steady-State RRO Model Output

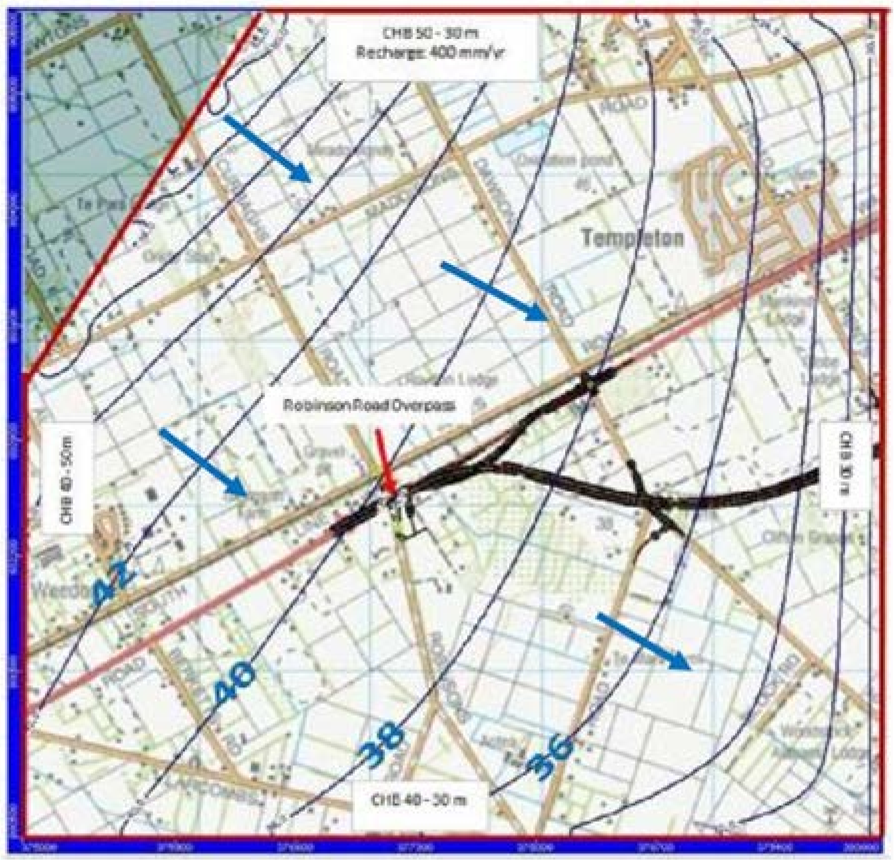
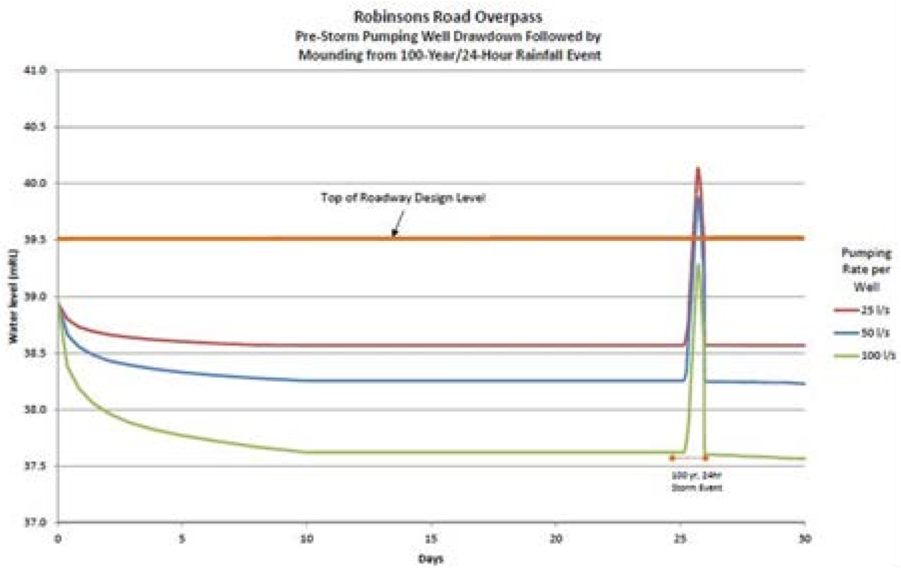


Figure A7 Steady-State RRO Model Output

The steady-state model was then used to model transient pumping and infiltration. The results are shown in Figure A8 as a hydrograph of water levels directly beneath the infiltration structure.

Figure A8 – Predicted Water Levels from Pumping Followed by 24-hour, 100-Year Rainfall



Halswell Junction Road Model

Purpose

The Halswell Junction Road model was developed using Visual MODFLOW to assess drainage rates needed to maintain groundwater levels below a design level of 18.75 m.

Model Set Up

The two-layer steady-state model was developed from the regional steady-state model. Flow directions and heads (water levels) from the regional model were used to define the sub-model region of approximately 2 km x 2.7 km centred near Halswell Junction Road. The model grid was refined to 1.5 m x 2.5 m around the HJR graduating out to 45 m x 75 m. The model has a base elevation of -40 mRL. Figure A9 shows the model set up.

Figure A9 RRO Model Set-Up Showing Boundaries and Infiltration Structure

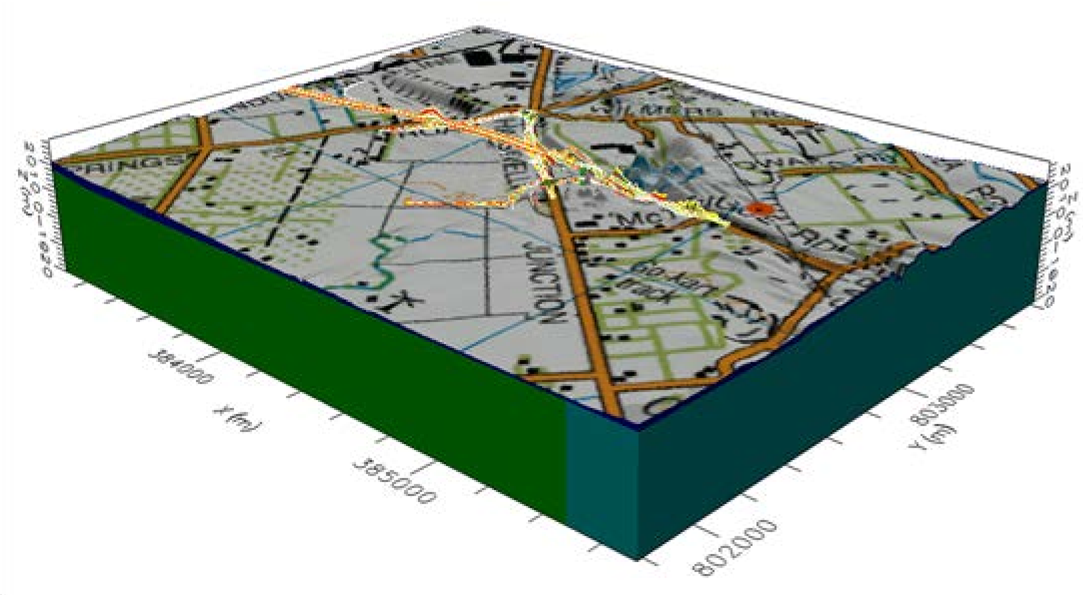
Boundaries | Constant-Head | Drain | River

Distribution of Hydrogeological Units

The distribution of hydrogeological units beneath HJR was assessed from investigation bores along the alignment and bore logs available from ECan. The HJR area is underlain at the surface by the silts and clays with fine sand of the Yaldhurst Member underlain by the sandy gravel and sandy silty gravel of the Halkett

Member (both of the Springston Formation) and the underlying Burnham Formation Figure A10 shows cross-sections of the model.

Figure A10 Model Set-Up Showing Layering and Hydraulic Conductivity Zones



Hydraulic Parameters

Layer 1 was modelled with a hydraulic conductivity of 3 x 10-6 m/s while the lower layer was modelled with a hydraulic conductivity ranging from 3 x 10-5 to 3 x 10-3 m/s, to assess the effects of varying silt and clay content observed at different locations beneath CSM1 and CSM2. A vertical anisotropy (ratio of Kv/Kh) of 0.1 was assigned to account for the variable presence of leaky aquitard zones of silt and clay within the sand and gravel unit. The values of hydraulic conductivity used in the model are listed in Table A5.

Table A5 – Hydraulic Parameters in the HJR Model

	Material Type Colour on Section			
Clay, silt and sand (confining layer)	1		1×10^{-6}	0.1
Sand and gravel	2		3.0×10^{-3}	0.1
Sand and gravel (higher silt and clay content)	2		3.0×10^{-5}	0.1

Model Boundaries

Edge boundaries: The model was set up with constant head boundaries placed to replicate the highest groundwater levels calculated from the regional model with CPW as shown in Figure 8 (in the main body of the report). These levels reflect the maximum groundwater level in Aquifer 1 as calculated from the high water levels from wells in the ECan database plus the predicted effects of CPW. The general-head boundaries were set to replicate the water level contours (Figure 6) where they intersected the lateral boundaries of the model. We have assumed no significant vertical flow gradients in the modelled area resulting in the same constant head value (equivalent to the intersecting water level contour value) assigned to all layers from top to bottom of the model at each edge boundary cell.

Recharge Boundaries: Recharge at a rate of 36 % of precipitation was applied at the surface as sandy gravels are representative of the entire surface of the sub-model. This equates to approximately 250 mm/yr based on an average annual rainfall of 680 mm/yr as calculated from Halswell Ryan's Bridge gauge data.

Drain Boundaries: The Drain boundary function was used to model drains beneath the proposed Maize-Maze pond, the Owaka basin and the Ramp 1 and 2 ponds. The drains were modelled with outlet levels at 18.75 mRL. MODFLOW's Drain boundary function was also used to model Knights Stream to the south of the proposed motorway.

River Boundaries: The River boundary function was used to model the pond at Wilmer's quarry and reported leakage from the Halswell Junction pond. (Note that the Wilmer's quarry lake boundary shown in Figure A9 does not coincide with the pond shown on the topographic base map. The pond on the map has been filled in since the map was made with quarrying now occurring at a location.) Stock water races were not modelled. A field reconnaissance of the area revealed that the bottoms of the stock-water races in the area are generally lined with settled silts and clays providing a low-permeability layer that impedes seepage losses.

Lake Boundaries: The Lake boundary function was not used in the sub-model.

Base boundaries: The bottom of the model is a no-flow boundary with no groundwater moving in or out of the model.

Calibration and Sensitivity Analyses

The model relied on the calibration of the regional model as there are no monitored well water levels within the modelled area. The hydraulic conductivity values were those of the calibrated regional model.

The water budget statistics in Table A6 show that a mass-balance was reached during calibration. Inflow verses outflow discrepancies were less than 0.5 %.

Table A6 Water Budget for the HJR Steady-State Model

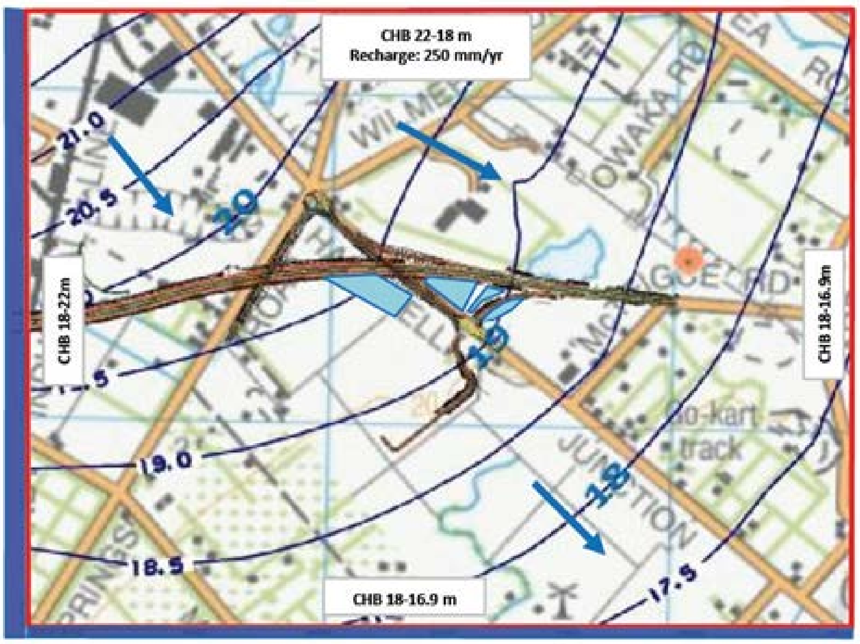
Cumulative Volumes Report [m^3]	Rates for Time Step Report [m^3/day]
IN:	IN:
Storage = 0 [m^3]	Storage = 0 [m^3/day]
Constant Head = 0 [m^3]	Constant Head = 0 [m^3/day]
Wells = 0 [m^3]	Wells = 0 [m^3/day]
Drains = 0 [m^3]	Drains = 0 [m^3/day]
MNW = 0 [m^3]	MNW = 0 [m^3/day]
LAKE SEEPAGE = 0 [m^3]	LAKE SEEPAGE = 0 [m^3/day]
Recharge = 1316.1903 [m^3]	Recharge = 1316.1903 [m^3/day]
ET = 0 [m^3]	ET = 0 [m^3/day]
River Leakage = 500.5704 [m^3]	River Leakage = 500.5704 [m^3/day]
Stream Leakage = 0 [m^3]	Stream Leakage = 0 [m^3/day]
General-Head = 72568.5781 [m^3]	General-Head = 72568.5781 [m^3/day]
Total IN = 74385.3389 [m^3]	Total IN = 74385.3389 [m^3/day]
OUT:	OUT:
Storage = 0 [m^3]	Storage = 0 [m^3/day]
Constant Head = 0 [m^3]	Constant Head = 0 [m^3/day]
Wells = 0 [m^3]	Wells = 0 [m^3/day]
Drains = 655.145 [m^3]	Drains = 655.145 [m^3/day]
MNW = 0 [m^3]	MNW = 0 [m^3/day]
LAKE SEEPAGE = 0 [m^3]	LAKE SEEPAGE = 0 [m^3/day]
Recharge = 3937.4653 [m^3]	Recharge = 3937.4653 [m^3/day]
ET = 0 [m^3]	ET = 0 [m^3/day]
River Leakage = 5331.6665 [m^3]	River Leakage = 5331.6665 [m^3/day]
Stream Leakage = 0 [m^3]	Stream Leakage = 0 [m^3/day]
General-Head = 64773.3008 [m^3]	General-Head = 64773.3008 [m^3/day]
Total OUT = 74697.5776 [m^3]	Total OUT = 74697.5776 [m^3/day]
IN - OUT = -312.2388 [m^3]	IN - OUT = -312.2388 [m^3/day]
Discrepancy = -0.42%	Discrepancy = -0.42%

A sensitivity analysis was conducted using two values of hydraulic conductivity to indicate a possible range of drainage rates with results presented below.

Model Results

The output of the steady-state model is presented Figure A11.

Figure A11 Steady-State HJR Model Output without Drains



The semi-calibrated steady-state model was used to model drainage from beneath the ponds with the system proposed by GHD. The results are shown in Table A7 for the range of hydraulic conductivities used in the sensitivity analysis.

Table A7 – Halswell Junction Road Drainage Results

Pond	Layer 2 K = 3.5×10^{-3}		Layer 2 K = 3.5×10^{-5}	
	Inflow to Drain (l/s)	**Drawdown (m)**	**Inflow to Drain (l/s)**	**Drawdown (m)**
Maize Maze	21.0	0.67	4.5	0.51
Owaka	11.1	0.53	2.7	0.44
Ramp 1	–	0.45	–	0.38
Ramp 2	–	0.42	–	0.35

References

Johnson, A. I., 1963. Specific Yield – Compilation of Specific Yields for Various Materials, USGS OFR 63-59.

Schlumberger, 2010. Visual MODFLOW Pro, Version 2010.1, Waterloo.

Appendix B

2-D Analytical Groundwater Flow Models

2-D Analytical Models: Drawdowns and Mounding using Theis Simulations

The distal effects of transient pumping and mounding were calculated with an analytical model using the Theis (1935) equation:

$s=Q/(4*P*T) * W(u)$

where:

s = the change in water level [m]

Q = pumping rate [m^3/day]

T = aquifer transmissivity [m^2/day]

$W(u) = -0.577216-\ln(u)+u-u2/2*2!+ u3/3*3!-u4/4*4!...+u25/25*25!-...$ (the "well function")

$u = (r^2 * S)/(4 * T*t)$

r = the radial distance from the well [m]

S = aquifer storativity [unitless]

t = pumping time [days]

We have applied the Theis equation as approximated by the Taylor expansion series to the 25th term. The small residual error remains at large distances and small times. However, the time and distances in the analyses presented below do not introduce any measurable error.

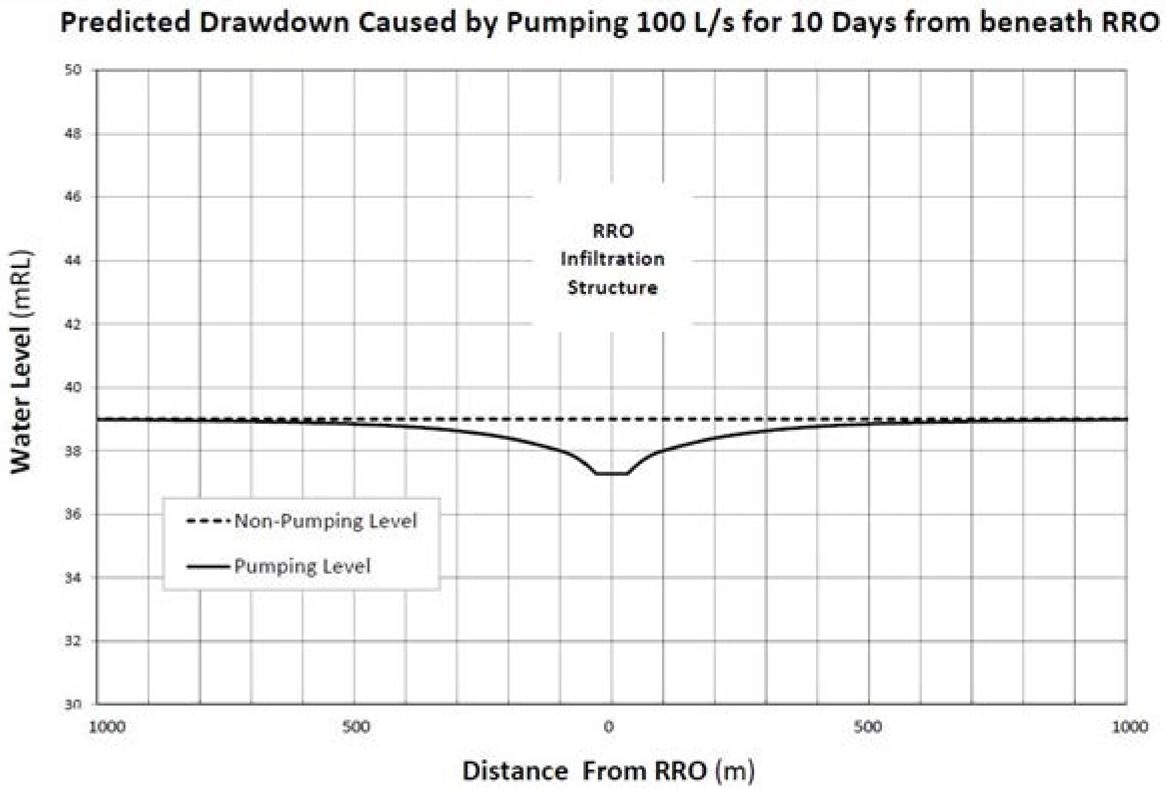
Reference:

Theis, C.V., 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of well using groundwater storage, Am. Geophysical. Union Trans., vol. 16, pp. 519-524.

Drawdowns caused by Pumping to Limit Maximum Groundwater Levels beneath RRO

We have used one pumping well placed beneath the infiltration structure to approximate the effects of two wells, one at each end of the structure. Transmissivity was calculated by assuming that the aquifer was 20 m thick with a hydraulic conductivity of 1.2 x 10^{-3} m/s. Storativity was equivalent to the specific yield (estimated at 0.2 based on the values listed in Johnson, 1963). Pumping was input as 100 L/s (2075 m^3/day) to replicate two wells pumping at 50 L/s. Pumping was assumed to continue for 10 days. The initial water level was 39.0 mRL. The distance-drawdown plot is shown in Figure B1.

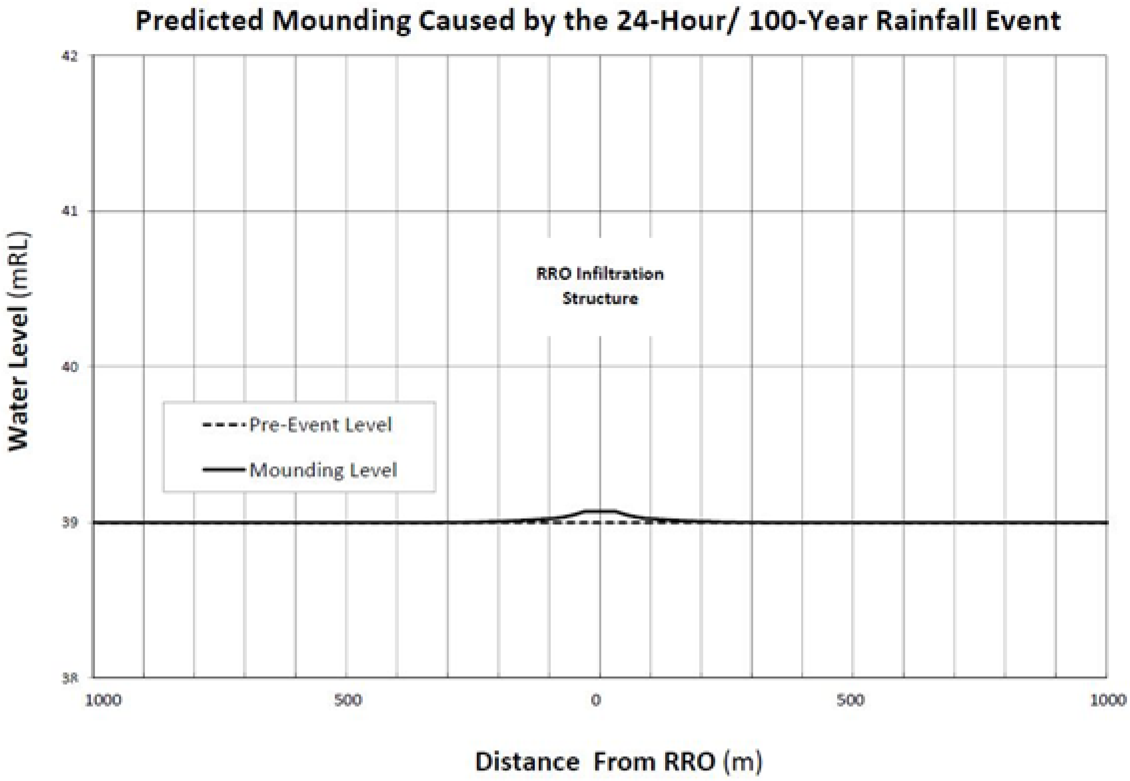
Figure B1- Drawdown with pumping at 100 L/s from Beneath RRO



Mounding caused by Infiltration of Stormwater beneath RRO

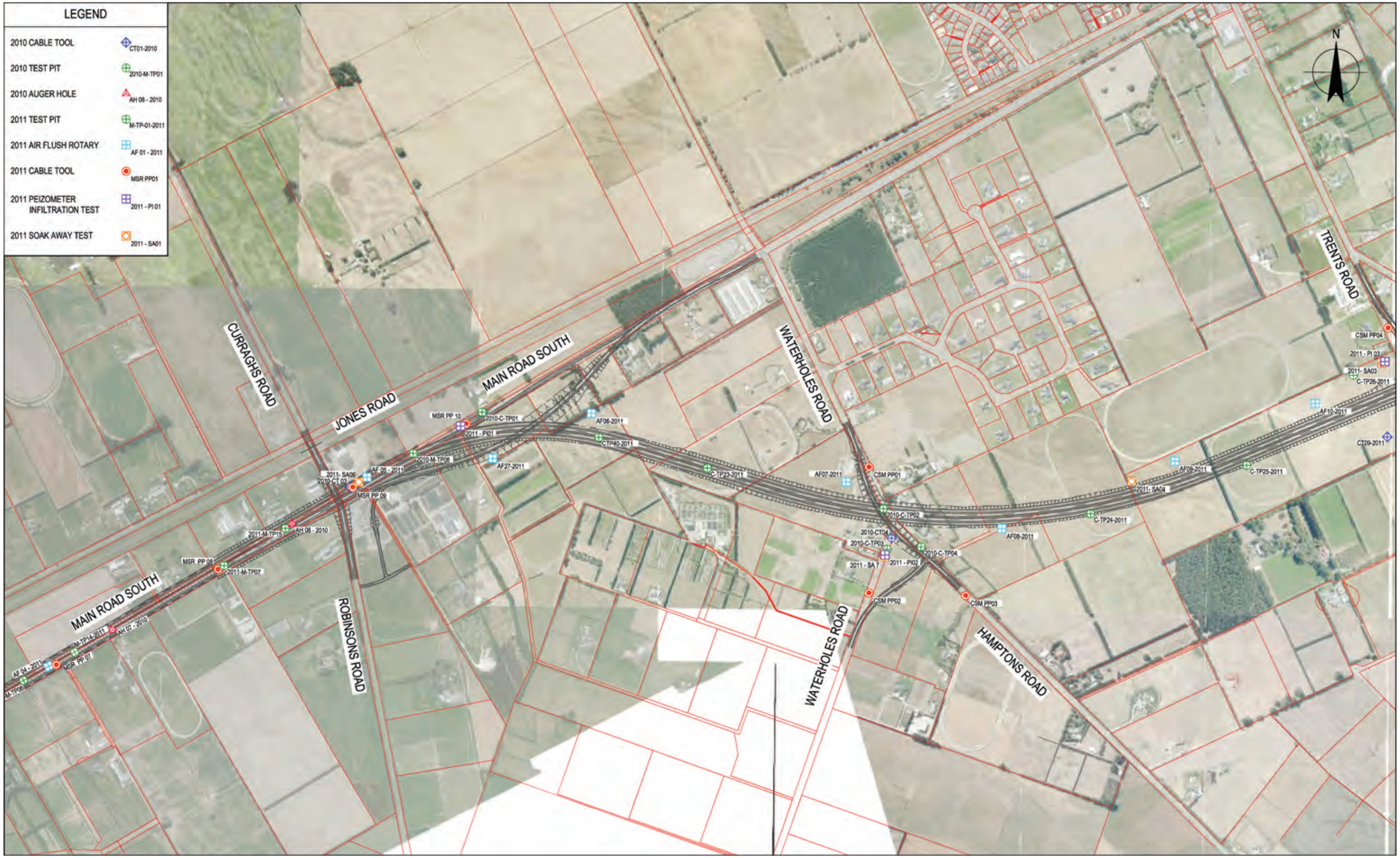
We simulated the infiltration of stormwater from beneath RRO as one injection well located at the centre of the infiltration structure. Transmissivity was calculated by assuming that the aquifer was 20 m thick with a hydraulic conductivity of 1.2 x 10^{-3} m/s. Storativity was equivalent to the specific yield (estimated at 0.2 based on the values listed in Johnson, 1963). Infiltration was input as the average of the "Q vs t" rates provided by GHD for the 24-hour, 100-year rainfall event. This average rate was applied over a 24 hour simulation period and is equivalent to a rate of 7 L/s (605 m^3/day). The initial water level was assumed to be 39.0 mRL. The distance-mounding plot is shown in Figure B2.

Figure B2 – Mounding away from the Infiltration Structure beneath RRO



Appendix C

Project Investigation Bore and Well Locations



No	Revision	Drawn	Checked	Approved	Date
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Project SH1 REGION 11 RP350/3.92 TO RP365/2.11
CHRISTCHURCH SOUTHERN MOTORWAY 2
Title GEOTECHNICAL PLAN
SHEET 1 OF 3

Drawing No:

Plot Date: 20 October 2011 - 12:23 PM Plotted by: Cameron Hall/Christchurch/GHD/AU Cad File No: G:\51\28982 CSM2\05 Geotech\CADD\Drawings\62236-B-Q201_Q203.dwg

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Title **GEOTECHNICAL PLAN** **SHEET 2 OF 3**

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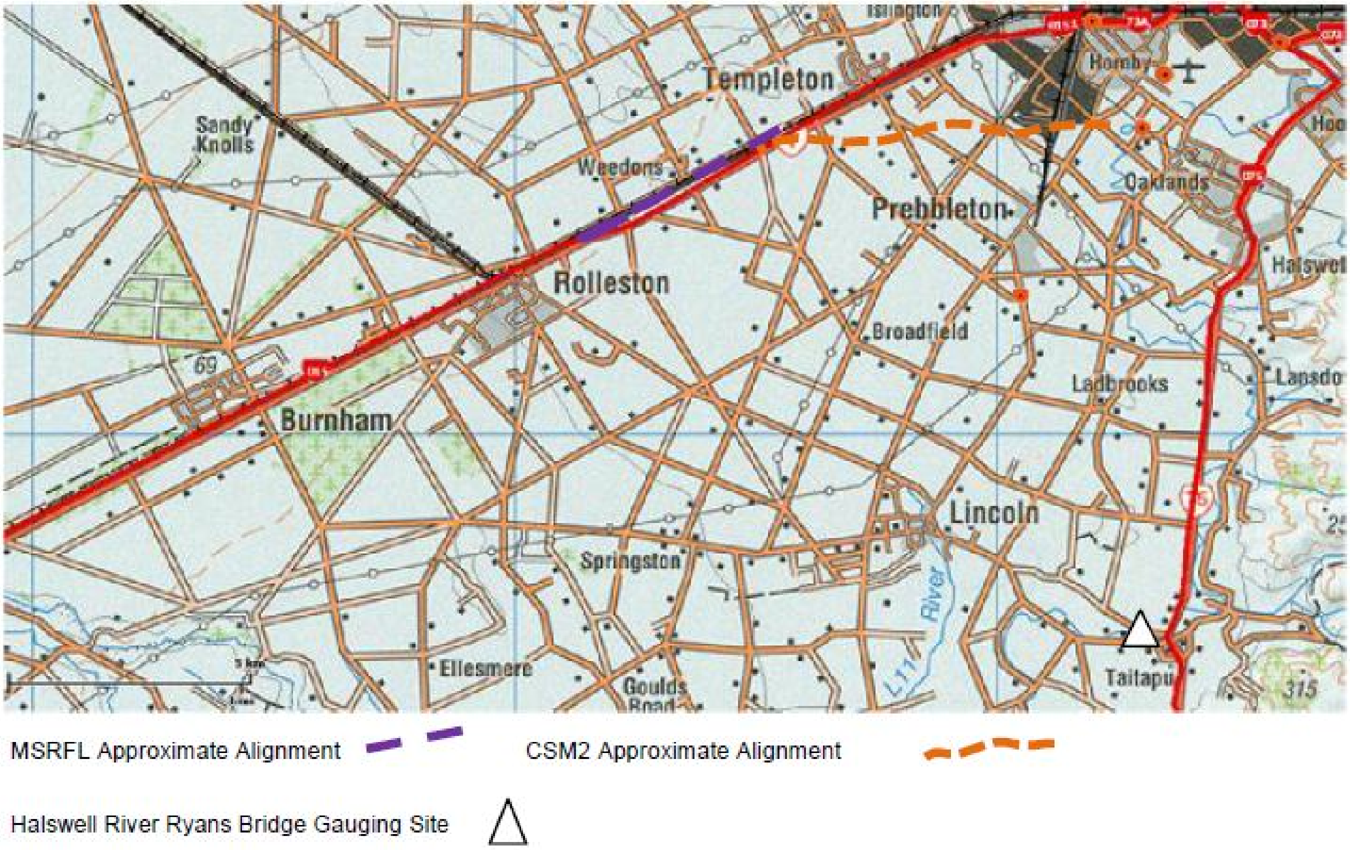
CHRISTCHURCH SOUTHERN MOTORWAY 2
Title GEOTECHNICAL PLAN SHEET 3 OF 3

Drawing No: 62236-B-Q203 Rev: A

Appendix D

Halswell River Ryans Bridge Gauging Station Location Map from ECan



Location of Halswell River Ryans Bridge Gauging Station (ECan, 2012)

Appendix D | Surface Water Modelling Report

Appendix D, Technical Report No 3

Christchurch Southern Motorway Stage 2 and Main South Road Four Laning

Surface Water Modelling Report

November 2012

Quality Assurance Statement			
GHD Beca	Prepared by:	Tony Miller	November 2012
	Reviewed by:	Ed Reid Mary O'Callahan	November 2012
	Approved by:	Gary Payne	November 2012

Contents

Glossary of terms

Abbreviations used in this report

Abbreviation	Description
MSRFL	Main South Road Four Laning
CSM2	Christchurch Southern Motorway – Stage 2
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
Beca	Beca Infrastructure Ltd
CCC	Christchurch City Council
CPW	Central Plains Water Enhancement Scheme
CSM1	Christchurch Southern Motorway – Stage 1
GHD	GHD Limited
HJR	Halswell Junction Road
LiDAR	Light Detection And Ranging (A method of gathering contour data from aerial methods)
NZTA	NZ Transport Agency
SWAP	South West Area Plan (CCC, April 2009)
WWDG	Christchurch City Council, Waterways, Wetlands and Drainage Guide 2011

1. Introduction

1.1 Background

The NZ Transport Agency (NZTA) has engaged GHD/Beca to develop the Christchurch Southern Motorway Stage 2 (CSM2) and Main South Road Four Laning (MSRFL) project (collectively referred as "the Project").

The objective of this assessment is to understand the flooding effects generated by the Project and the inter relationship with the surface water flows associated with the recently constructed Christchurch Southern Motorway Stage 1 (CSM1) project and the Christchurch City Council (CCC) – South West Area Plan (SWAP), in relation to the Halswell River catchment. This report forms Appendix D to "Technical Report 3: Assessment of Stormwater Disposal and Water Quality Environmental Effects" prepared in support of the Notice of Requirement and resource consent applications lodged for the Project.

More specifically, this assessment involved the development of a surface water model to interface with the Assessment of Groundwater Effects report.

This report:

- details the assumptions made and methodology used, and the resulting outcome of these modelling investigations;
- determines the effects of the CSM1 project, this Project, and the CCC works proposed as part of the SWAP project on upper catchment discharges into the Halswell River catchment; and
- provides appropriate recommendations for the mitigation of flood risk effects.

1.1.1 CSM1

The south western extent of the CSM1 ties into the local road network at the Halswell Junction Road (HJR). Associated works included the upgrade of HJR through to Main South Road and the upgrading of the HJR pond.

The Contractor for the CSM1 project (Fulton Hogan) carried out excavation to form the Owaka Basin as part of the SWAP project under direction of CCC. The CSM1 project has relied on stormwater capture, treatment and attenuation of flows at three ponds in the vicinity of HJR i.e. the HJR Pond, Mushroom Pond and Lee Basin.

The CSM1 project construction (as of September 2012) is substantially complete.

1.1.2 SWAP

The CCC adopted the SWAP in April 2009. It provides a framework to manage urban and business growth in the South-West area of Christchurch.

Under the Greater Christchurch Urban Development Strategy more than 10,000 new households will be established and about 200 hectares of industrial land will be developed in the next 35 years, making it one of the South Island's largest urban growth areas.

Since the adoption of the SWAP, CCC staff have developed an Implementation Plan. This is a detailed programme of actions for 2012-2015 to work towards achieving the objectives of the SWAP. It should be noted that the implementation of some planned project work has been delayed as a result of earthquake recovery work taking precedence.

Following the CCC adoption of the SWAP, the Council developed and lodged a network discharge consent application for the stormwater discharges associated with the SWAP. That consent (CRC 120223) was granted in 2012 and has now been given effect to.

1.2 Objectives

The following points summarise the key objectives for the hydrological study undertaken for this assessment:

- Understand the relationship between groundwater and surface water at the HJR area;
- Understand the effects of Project discharges to the Maize Maze and Ramp Ponds;
- Understand the potential effects of the catchments contributing to Montgomery's Drain (and ultimately the Upper Knights Stream, which discharges to the Halswell River), including:
- The filling of the pond and its overflow &spillway regime;
- The effect on the discharge to Montgomery's Drain;
- The effect of diverting this flow to the Owaka Basin (as per the CCC direction and as proposed by the SWAP);
- Understand the effect of the discharge to Wilmers Quarry;
- The effect on filling of Owaka Basin and Wilmers Quarry Basin and subsequent overflow to Montgomery's Drain; and
- Understanding the existing and future discharges to the Upper Knights Stream.
- To ascertain whether the stormwater discharged from the Project causes peak flows to Montgomery's Drain and Upper Knights Stream to increase or decrease. If a decrease can be demonstrated, then the effects on the Halswell River and the existing flooding downstream of the confluence where the Upper Knights Stream joins the Halswell River, then any flooding effects of the Project will have been avoided.

2. Stormwater Modelling

2.1 Introduction

This project involves the assessment of flows to each of the ponds adjacent to HJR[1], sizing of the drainage for upgrade to existing and sizing of the proposed new components and the recommendation of an overall remedial solution.

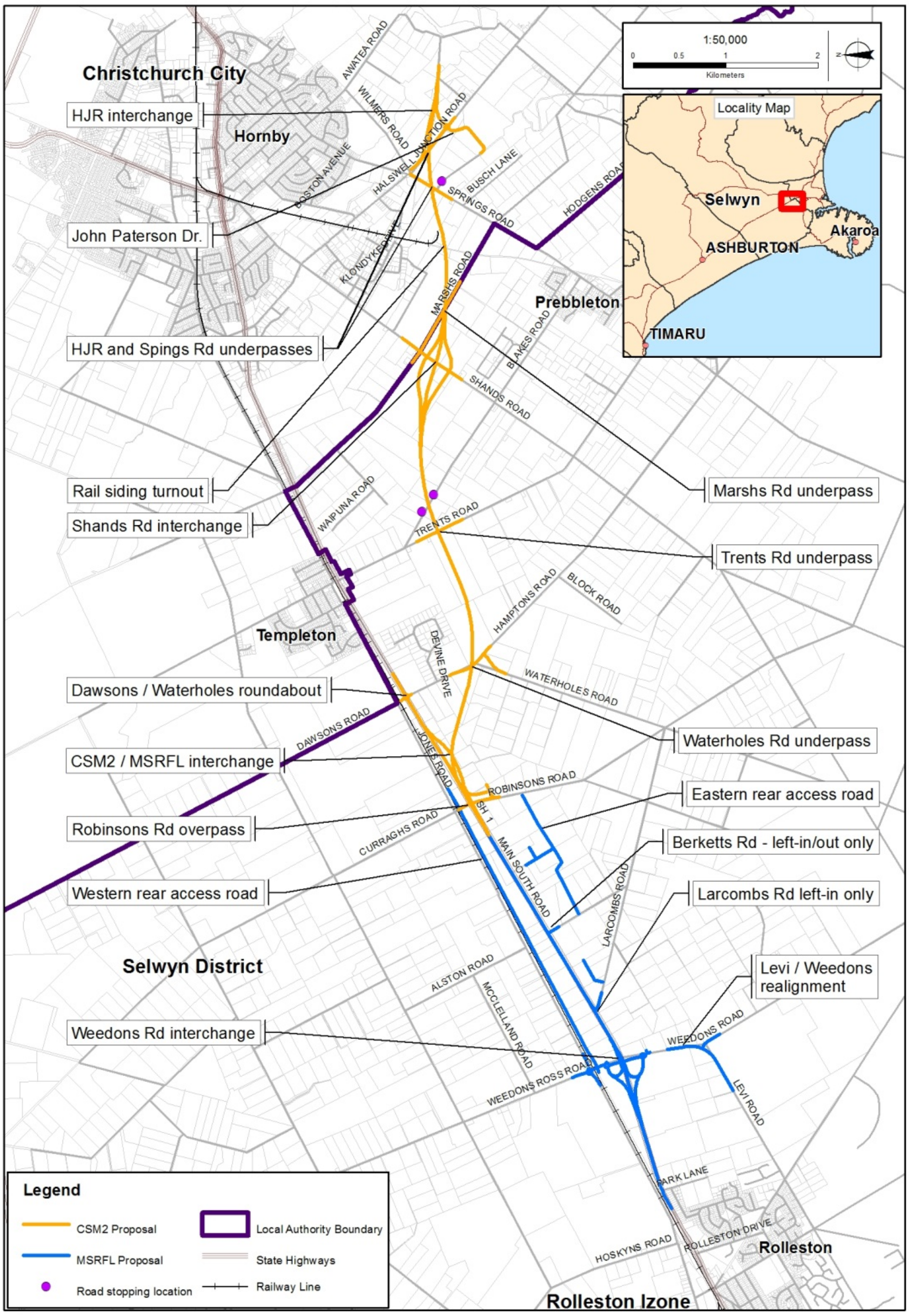
2.2 Study Area Details

2.2.1 Location

The study area is located at Halswell Junction Road, Hornby, Christchurch as shown on Figure 1.

[1] This includes Halswell Junction Road pond, Mushroom Ponds, Ramp Ponds, Maize Maze Ponds, Owaka Basin and Wilmers Quarry Basin

Figure 1 Proposal location map



2.2.2 Catchment

The total hydrological area in the model is 115 ha. This includes a 100[2] ha catchment associated with the HJR Pond. The balance area is confined within the adjacent CSM1 and CSM2 project area and immediate surrounds.

[2] 100 ha source - Ken Couling , CCC

2.2.3 Topography and Geology

Anecdotal evidence from the CCC[3] advises that within the 100 ha catchment, there are permeable areas and a limited formal primary drainage network. Notable exceptions are the reticulation along HJR and within the business area to the east of Curletts Road.

There is no significant evidence of ponding and flooding following heavy rain[4] . From this and bore logs along the alignment, we infer that the soil profile probably contains bands and areas of higher permeable sands and gravels.

For the study area adjacent to the motorway, Technical report 11 states that the ground profile log is characterised by "grey brown fine to coarse sand, and, fine to coarse gravels with cobbles and occasional boulders". There is a variation of silts and fine sands in the upper layers. More notably the surface layers contain higher proportions of silts and fine sands than the layers at depth. Bore logs and discussions with the authors of Technical Report 11 demonstrate that permeability rates can be expected to be lower towards the coast as opposed to the higher rates encountered further inland.

2.2.4 Land Use

The study area is largely zoned B4 (Suburban Industrial), B5 (General Industrial) and B7 (Business Wilmers Quarry) with areas of B1 (Local Centre) & L1 (Outer Suburban).

The proportion of each land use and impermeable surface area percentage for Maximum Probable Development is presented in Table 1.

Table 1 Impermeable Surface Coverage in the Model Sub Catchments

Sub Catchment	Description	Catchment Node	Area (Ha)	Impermeable %age in the Model (%)
A	Ind/Com	14	100	65
Z	CSM1&2 to Mushroom	Z	5.8	70
G1	CSM2 to Maze	G1	6.07	40.7
H1	Maze Forebay	H1	0.6	90
I1	Maze P1	I1	0.6	50
J1	Maze P2	J1	0.6	50
M	CSM2 Ramp	M	0.95	90
O	Ramp Ponds	O	0.64	50

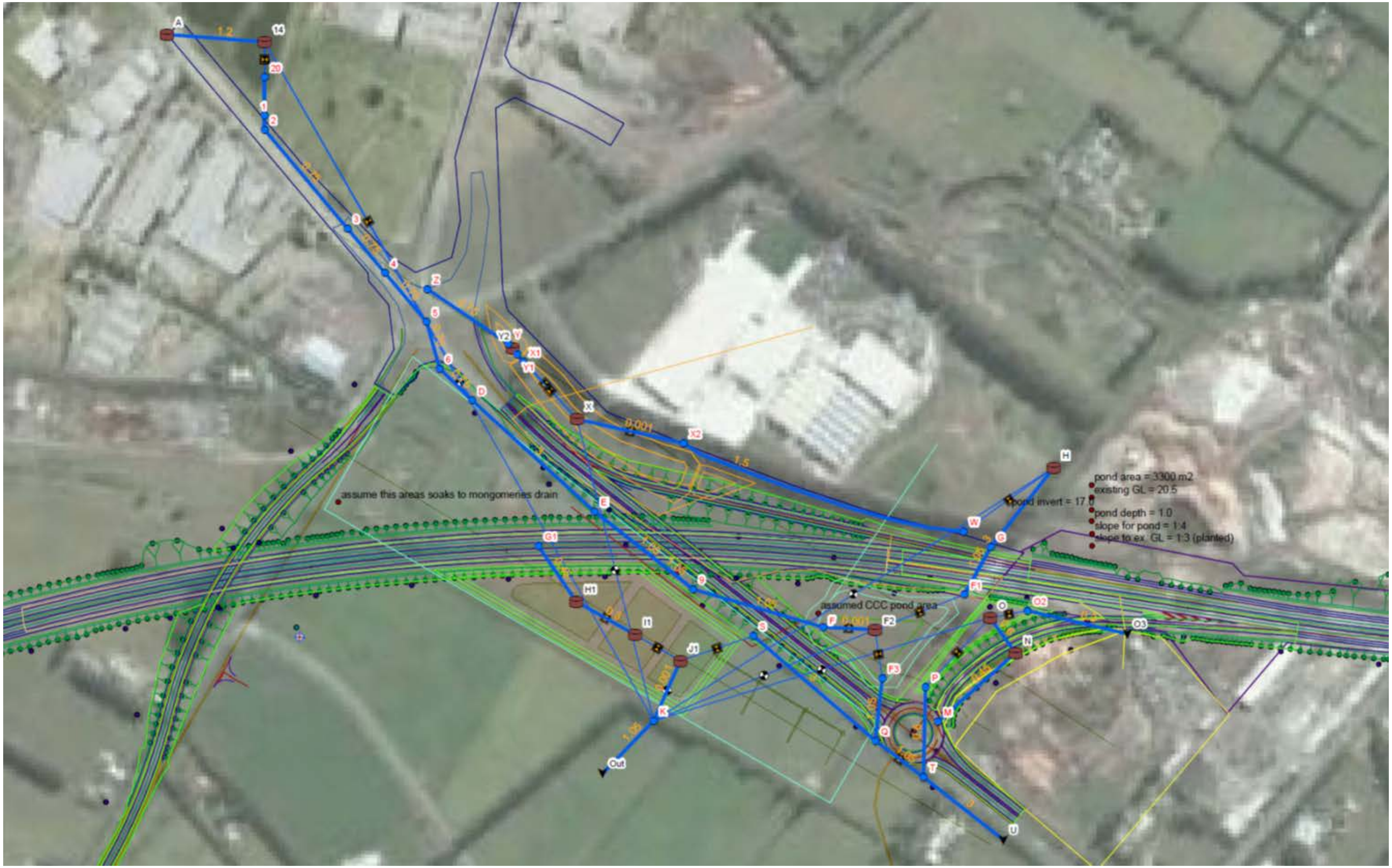
[3] Ken Couling

[4] Ken Couling, pers comm., August 2012

2.3 Drainage System in the Model

The drainage system in the study area is comprised of a combination of pipes, open channels and basins. The extents of the modelled network are shown in Figure 1.

Figure 2 MIKE Urban Model Network (see following page)


assume this areas soaks to mongomeries drain
pond area = 3300 m2
existing GL = 20.5
pond invert = 17.0
pond depth = 1.0
slope for pond = 1:4
slope to ex. GL = 1:3 (planted)
assumed CCC pond area
Out

2.4 Assessment Approach

The general process that was utilised in this investigation was to:

- Model the catchment areas draining to the various ponds;
- Assume drain rate to ground from pond base (and any associated disposal fields) and compare the assumptions against ground water model assumptions;
- Test the assumed drain rate against the ground water model, and;
- Model the dynamic flow through the ponds to show fill and spill rates.

2.5 Hydrological Model

2.5.1 Methodology

The model was configured based on the following:

- No sub-catchment boundary data was provided by the CCC for the larger 100 Ha catchment. The sub-catchment data used for the hydrological calculations (including areas, imperviousness, slopes and lengths) were estimated based upon the author's local knowledge of the area and discussions with the CCC;
- The 100 ha catchment was assumed to be 65% impervious. The CCC had advised that the formal stormwater drainage piped network does not have universal coverage. Based on anecdotal evidence there is no indication of extensive surface flooding or ponding from regular rain events;
- The remaining7 sub-catchments included in the hydrological model were derived from the CSM1 and CSM2 area catchments;

Rainfall rates were derived from the 24 hour storm for the 10% AEP and 24, 48 and 60 hour storm for the 1% AEP event (10 and 100 year storms). Climate change was also considered as set out in the Christchurch City Council, Waterways, Wetlands and Drainage Guide 2011(WWDG) update.

2.5.2 Catchment Delineation

Sub catchment data used included area, imperviousness, length and slope for parameter calculation purposes. No catchment boundary shape file was provided by the CCC. The omission of the shape file was considered not to impact on the model results.

2.5.3 Rainfall – Runoff Model

The catchment hydrology was simulated using the Mouse UHM Module.

2.6 Hydrodynamic Models Model Build

The hydrodynamic modelling software used in this assessment was DHI MIKE Urban.

2.6.1 Pond Volumes and Physical Controls

The as-built records from the CSM1 project, as-built drawings from the HJR pond outlet controls and design levels for the CSM2 works were correlated and entered into the model to establish a fair record of the existing and proposed infrastructure.

The model was run and undersized design elements adjusted to ensure that the model represented an outcome that could reasonably be expected at the end of the detailed design process.

2.6.2 Model Domain Description

The MIKE Urban hydraulic module component included the pipe network and open channel network for this assessment as well as specific orifice and weir controls expected to be installed during construction.

2.6.3 Hydraulic Model Configuration

The hydraulic network for the system reticulation was developed including consideration of the following aspects:

- The system reticulation is based on our understanding of the SWAP and the CCC intent e.g. flows from the HJR pond to spill to Owaka Basin, etc;
- Weirs, orifices, typical open channel cross sections, basin geometry information is as assumed by the hydraulic modeller;
- All pipes and channels in the model have been assigned with accepted standard friction loss factors. The roughness coefficients used in the model are summarised in Table 2 below:
- All manhole and structure nodes were assigned with accepted standard friction loss factors. Generally manholes were assigned a MOUSE Classic (Engelund) Km value of 0.25;
- Manhole, pipe, culvert and entry point blockage has not been considered in this study. It is assumed that maintenance of these units to prevent blockage will be addressed via a consent condition requiring the preparation of an operation and maintenance plan for stormwater infrastructure;
- Catchpits were not explicitly modelled as part of this study. It was assumed that all stormwater runoff would enter the reticulation network if adequate pipe capacity were available. It is assumed that maintenance and the effective operation of the catchpits will be addressed via the operation and maintenance plan for stormwater infrastructure.

Table 2 Hydraulic Parameters

Type	Adopted Model Value
Pipe	0.012
Open channel	0.030

3. Modelling Information

3.1 Simulations Overview

The model simulations undertaken for this Model Development reporting component of the assessment are presented in Table 3 below.

Table 3 Model Development Simulations

Simulation	Details
10 year rainstorm	For design purposes
100 year rainstorm	For design purposes
100 year rainstorm with climate change (as per WWDG)	For design purposes

3.2 100 Year Rainfall Event

The 10 and 100 year ARI rainfall event was simulated in order to generate results files to be used for the purpose of design.

The 100 year ARI rainfall event simulation the model files are:

Hydrology and hydraulics simulation ID is “100Y_24HBase”.

3.2.1 Simulation Parameters

The following parameters were used for the runoff and network model simulations.

- Runoff Model
  - Time Step = 300 seconds
  - Model Type = UHM
  - Model simulation: Start time 01/01/2012 12:00 am / End Time 2/01/2012 5:00 pm
- Network Model
  - Time Step:
    - Minimum = 60 seconds
    - Maximum = 60 seconds
  - Factor = 1.0
  - Model Type = Dynamic Wave
  - Model Simulation: Start time 01/01/2012 12:00 am / End Time 2/01/2012 5:00 pm

4. Modelling Assumptions & Discussion

4.1 Catchments

The initial round of modelling considered catchments and flows to the HJR Pond and flows out of that pond, as well as catchments and flows that fall on the Project area and the other CCC ponds in the vicinity, including:

- CSM2 catchments that drains to Project ponds near HJR;
- Maize Maze Pond;
- Mushroom Pond;
- Ramp Pond;
- Owaka Basin; and
- Wilmers Quarry.

The original catchment that drains to the Lee Basin (CSM1 Pond) was adjusted and reduced to take account of the CSM2 Project. The Lee Basin catchment will be reduced to accommodate the Project with the western end of that catchment proposed to be drained to the Ramp Pond.

The Lee Basin drains to soakage and does not contribute to the flows in Montgomery's Drain even through the pond does contribute to ground water re-charge.

4.2 HJR Pond

The pond catchment has been advised by the CCC as;

- Being 100 Ha in area;
- The stormwater line leading to the pond has flush jointed pipe that is known to leak (significant) quantities of water;
- The pond is lined at and below pond normal operating level; and
- When the pond rises above this normal operating water level a significant degree of leakage (to ground) has been observed (i.e. when the pond is full but below primary spill level, the pond drains away over a numbers of days (rather than months) back to a nominal low water level).

The catchment has an overall gradient similar to the falls experienced by the Canterbury Plains, (approx. 1:250) with fall from Main South Road down HJR to the HJR Pond. We would expect a flat catchment to have a reasonable degree of attenuation within the catchment as onsite storage.

In order to simulate this catchment in the model, we have allowed for the collection of catchment runoff to an additional storage model node upstream of the pond node, but with a 1050 mm dia. conduit to the pond. This is to simulate onsite storage within the catchment which has the potential to moderate peak catchment inflows.

As part of the CSM1 project the HJR Pond size has been substantially increased. As-builts of the completed pond were not available for this study; however the low water and outlet controls were available. The spillway pond shape at the higher pond level was available. Quantities and areas were extracted and entered into the model. From these quantities we have confidence that the pond volume is modelled with a reasonable degree of accuracy.

This HJR Pond currently fills, spills and drains into Montgomery's Drain via a 750 mm dia. overflow pipe under HJR. In order to comply with the SWAP, the outlet to this pond is to be redirected to the recently formed Owaka Basin as part of the CSM2 project. This outlet was therefore routed into the Owaka Basin in the model.

4.3 Owaka Basin

The Owaka Basin has been set out in the SWAP and its construction is consented in the network discharge consent for the SWAP area.

The Owaka Basin has been designed by the CCC with a base area of 6,900 m² and an invert average level of 28.53 m (CCC drainage datum) or 19.48 m (Lands and Survey / project datum).

The inlet to the basin from the realigned Montgomery's Drain will enter the pond on the western side of the pond from under the motorway. Initial pipe size is a single 1050 mm dia.

The pond source for filling is from the overflow from the HJR Pond plus its own catchment.

There are two potential spill points from the basin:

- To Wilmers Quarry. This will occur when the water level in the pond has risen above the pipe invert level of RL = 20.55 m. In the future it may be possible to raise the spill level to Wilmers Quarry. The edge of carriageway at the low point on the CSM2 alignment has a channel level of approximately RL = 21.15 m. In order to protect the pavement a maximum target pond level of 20.75 m has been chosen. Although this is only 400 mm below the pavement level, a proposed secondary drainage system will need to collect pavement drainage and feed this away from the project area. Notional pavement drainage level will be 1.1 m below the channel level or RL = 20.05 m at the project low point;
- Overflow back to Montgomery's Drain. This is achieved by weir flow to simulate a manhole overtopping and piped flow back to the drain.

The Owaka Basin outlet to Wilmers Quarry has been constructed as part of CSM1. The connection consists of twin 1050 mm dia. pipes, with minimal fall. Flow in the pipe will be initiated when the level in either the Wilmers Quarry or Owaka Basin has risen above RL = 20.55 m.

No formal design has been carried out by the CCC for the operation of storage in the Wilmers Quarry and Owaka Basin. Based upon the geometry of both basins, and clearance to the motorway carriageway on the CSM2 alignment, a primary weir outfall from the Owaka Basin has been assumed in the model, by using a 1050 mm dia. manhole. A 750 mm dia. pipe to the Montgomery's Drain will then pass under the HJR embankment.

The model reflects this outlet control with a 3.0 m long weir at RL = 20.75 m to discharge to the drain.

4.4 Wilmers Quarry

The Wilmers Quarry site at 46 Wilmers Road is owned by the CCC and has an area of 4.1 Ha. The CCC has identified this as a potential area for ponding, subsequent to the preparation of the SWAP. The storage area within the quarry has been included in the model, however the catchment details were not available so were not included in the first stage of modelling. If the 28ha catchment details had been included in the model, the modelled flow to Montgomery's Drain would increase, however we expect that the modelling report conclusion would not differ.

Based upon LiDAR the quarry has a small body of water at RL = 19.0 m with an area of 0.25 Ha and land sloping up from the low point.

This quarry in the future has the potential to store significant quantities of stormwater.

The final land shape for the quarry was unknown at the time of modelling. However in order to establish a quarry storage volume, the quarry was assumed to have a base area of 0.5 Ha at RL=19.0 m and for modelling purposes, an upper level of 3 Ha at an RL = 23.0 m.

Wilmers Quarry is more than 100 m from the CSM1 motorway and an easement is available to allow access and a swale to the drainage infrastructure under the motorway.

4.5 Soakage through the Base of the Ponds

At the commencement of modelling, the groundwater highs were unknown and dependent upon the groundwater modelling work being undertaken concurrently with the hydraulic modelling which informed this assessment. In order to assume a leakage rate through the pond base, a flow from each pond was modelled as an orifice flow.

Base areas of each of the ponds were taken and infiltration rates of 10 mm/hr taken and converted to a flow rate. A notional head of 600 mm was taken and a notional orifice size adopted with discharge to waste from the system, and entered into the model.

5. Model Findings

5.1 Model Runs

The critical 24 hour rain storm profile was adopted from the CCC Waterways, Wetlands and Drainage Guide (CCC, 2003).

The following runs were modelled:

- 10 year 24 hour storm;
- 100 year 24 hour storm;
- 100 year 24 hour storm with climate change;
- 100 year 48 hour storm;
- 100 year 60 hour storm.

The 100 year with climate change was modelled initially. However, the critical groundwater level rise was triggered by the smaller 100 year 24 hour storm without climate change. As such, the climate change scenario was not needed to be assessed further.

During the progress of the work, comparison with the groundwater model was undertaken. The results of the groundwater model showed that the peak groundwater level could be well above pond base level at its peak. As such, an intervention strategy is required to lower groundwater levels (and is proposed as part of the Project) in the general area such that the ground water would need to be at or below the pond base level at the beginning of a major storm event.

5.2 Model Outputs

5.2.1 HJR Pond

The catchment node upstream of the model has a storage component to simulate the catchment storage and attenuation. This consists of a 1050 mm dia. drainage pipe to the pond that (provides the hydraulic attenuation) and a storage component. At the peak of the storm the following volumes are impounded as shown in Table 4.

Table 4 HJR Catchment storage node

Storm	Peak Storage (m)	Peak Storage m³	Peak Outflow (m³/s) to HJR Pond
Q_{10} 24 hour	23.25	0	1.9
Q_{100} 24	24.24	6000	3.2
Q_{100} 48 hour	24	0	2.1

Q_{100} 60 hour	23.8	0	1.8

The outflow from the catchment storage flows to the recently enlarged HJR Pond. The existing inflow pipes to the pond are 1800 mm dia. pipes and butt jointed. The modelled pipes are 1050 mm dia. and have been sized to simulate the storage within the catchment and to account for a portion of surface ponding and overland flow.

From the modelling in all simulations, the peak water level in the pond does not reach the HJR Pond secondary spillway level for the Q10 or 10% AEP event.

However for the balance of events modelled there is spill over a 40 m wide weir crest. This flow would pass over HJR / Springs Road roundabout and flow either to the Mushroom Pond or Montgomery's Drain. Either way this flow will arrive at the Owaka Basin or to Wilmers Quarry.

The existing outlet control to the HJR pond is via a 525 mm dia. pipe which in turn feeds the 750 mm dia. pipes under the HJR / Springs Road roundabout. As part of detailed design it is possible to enlarge this 525 mm dia. conduit and allow more flow through the pipe system and hence less over the spillway.

The modelled peak flows are shown in Table 5 below.

Table 5 HJR Pond

Storm	Peak Storage Level (m)	Peak Inflow m³/s	Peak Spillway m³/s	Peak Piped Outflow (m³/s) to Owaka Basin
Q_{10} 24 hour	22.67	1.9	0	0.54
Q_{100} 24 hour	23.46	3.2	2.12	0.63
Q_{100} 48 hour	23.45	2.1	1.3	0.62
Q_{100} 60 hour	23.44	1.8	1.08	0.63

As the spillway flow is large we would recommend upgrades to the HJR Pond outlet to reduce spillway flows and to confirm this by modelling. It is noted that this pond is under the CCC control and is not part of the Project. On completing the assessment, it is noted that the catchment parameters are conservative, which has resulted in high flows entering the HJR Pond.

5.2.2 Owaka Basin

The current Owaka Basin has been constructed but has no external catchment and no inflow or outflow pipes or channels. The pond or basin has been set out in the SWAP and constructed by Fulton Hogan for the CCC. This work was carried out in parallel with the CSM1 project.

To integrate the Project stormwater infrastructure with the SWAP, the NZTA is proposing to carry out works to the CCC stormwater infrastructure that includes:

- Construction of a diversion of the Montgomery's Drain and divert to the Owaka Basin. This can be achieved with a single 1050 mm dia. pipe;

- Construction of a high level outlet from the Owaka Basin back to the realigned Montgomery's Drain;

Also proposed is the construction of an under drainage network for Owaka Basin. The scope of this network is indicative only. This is a CCC initiative outside of the Project scope.

Note: As a result of this study, the need for an under drainage system has been confirmed and is discussed in Section 6.1 below.

Table 6 Owaka Basin

Storm	Peak Storage Level (m)	Peak Inflow m^3/s
Q_{10} 24 hour	20.23	1.9
Q_{100} 24 hour	20.90	2.75
Q_{100} 48 hour	20.96	1.92
Q_{100} 60 hour	20.96	1.71

Based upon modelling outputs, the peak water level in the Owaka Basin is RL = 20.96 m. This gives a freeboard of only 220 mm to the adjacent carriageway on the CSM2 alignment.

In order to protect the pavement, a separate pavement drainage system is proposed to be installed at the time of construction. When activated, this system will naturally drain by gravity to the Upper Knights Stream.

The notional spill level in the model from the Owaka Basin to Montgomery's Drain is set at RL = 20.75 m and would need to be enlarged to lengthen the weir (currently set as a 1050 mm dia. and in the model as a 3 m long weir). During detailed design, the weir length should be increased to approximately 7 m, as this would lower the peak discharge level in the Owaka Basin. This adjustment is required to protect the CSM2 pavement.

5.2.3 Wilmers Quarry Basin

As stated above:

- The final slope and available volume is unknown;
- For the first round of modelling the catchment area to the Quarry was unknown and not included;
- Soakage to ground has been modelled at a rate of approx. 300 L/s.

Once spill for from Owaka Basin occurs then both the Owaka Basin and Wilmers Quarry will fill at a similar rate until spill occurs to Montgomery's Drain.

5.2.4 Maize Maze Pond and Basin

The preliminary design of the Maize Maze Pond is based upon motorway drainage from the low point in the CSM2 alignment approximately 250 m to the east of the Maize Maze Pond. The pond invert level of RL = 18.75 m is controlled by the vertical carriageway alignment.

In final detailed design, the pipe conveyance will allow the pipes to drain freely when the pond is empty. However when the pond is filling/full, the pipe size will need to reflect the reduced conveyance potential caused by the drowned outfall into the Maize Maze pond.

Based upon the assumption that groundwater does not compromise storage volume within the pond the pond will fill to the levels set out in Table 7 below.

Overflow to Montgomery's Drain does not occur for any of the modelled events.

Table 7 Maize Maze Pond

Event	Peak level inflow (m^3/s)	Peak level in Pond (m)	Peak Spill Rate
Q_{10} 24 hour	0.108	19.12	0
Q_{100} 24 hour	0.191	19.45	0
Q_{100} 48 hour	0.124	19.6	0
Q_{100} 60 hour	0.107	19.7	0

Groundwater and Maize Maze Pond

The assumption for the surface water model is that the groundwater is at least 1 m below the pond base level at the beginning of the storm. In this case it is also assumed that the pond water level will rise faster than the groundwater rise. Thus the pond volume will not be compromised by groundwater.

This assumption has been reviewed with the groundwater model which is being prepared by others. As a result of the potential for groundwater to be above pond invert levels at the commencement of the storm, a groundwater intervention strategy is proposed and this is discussed below in Section 6.1 below.

5.2.5 Ramp Pond and Basin

These ponds likewise have been modelled. The fill and spill table is set out in Table 8 below.

Table 8 Ramp Pond

Event	Peak level in Pond (m)	Peak Spill Rate
Q_{10} 24 hour	19.09	0
Q_{100} 24 hour	19.2	0
Q_{100} 48 hour	19.2	0
Q_{100} 60 hour	19.1	0

The base level of the Ramp Pond and Ramp Basin has been set at RL 18.5 m. This has been based upon a minimum carriageway level of RL = 19.87 m on the edge of seal on the outside lane. A target peak level in the pond of 19.27 m will give a 600 mm freeboard.

The same assumption that the groundwater will be at least 1 m below pond base level at the commencement of the storm has been used for this pond.

Thus for all events modelled in this exercise the peak pond level meets the pavement clearance envelope. However a separate pavement drainage system will be required to protect the pavement and drain to a different outlet. This is for consideration in detailed design.

5.2.6 Lee Basin

The Lee Basin was not modelled in this study. The catchment area to the basin will be reduced as part of the Project. There is also a small infilling of the batter along one edge of the pond as a result of the earthworks for the Project. It is expected that the basin will not be compromised by this reduction in volume.

5.2.7 Montgomery's Drain and Upper Knights Stream

Montgomery's Drain currently flows down the south-western side of HJR. The flows in the drain come from the overflow generated by the spilling from the HJR Pond plus any local impervious runoff from HJR downstream of the CSM1 roundabout. As the drain is relatively permeable, it is assumed that the runoff from the adjacent rural land is non-existent for almost all rain events.

Once the Project is complete, the Montgomery's Drain will receive flow from:

- Owaka Basin Overflow. This also includes spill resulting from flows from the Wilmers Quarry flowing back into the Owaka Basin;
- Spill from the Maize Maze Pond as well as controlled draining at the end of the storm event once there is capacity in the downstream Halswell River catchment. (Note: Zero overflow during flood event, based on the modelling runs carried out to date, for the critical 24 hour storm. However, for the 48 and 60 hour storms the ponds fills and creates modest overflows late in the storm event);
- Spill from the Ramp Pond as well as similar controlled draining at the end of the storm. (Note: Zero overflow base on the modelling runs carried out to date)

6. Modelling Discussion and Recommendations

6.1 Discussion

The hydrological and hydraulic modelling carried out has been done to enable prediction of effects on the surface water flow regime at the CCC and Project ponds in the vicinity of HJR. Further, to determine the effects of discharges to the Halswell River, via Montgomery's Drain and the Upper Knights Stream.

In parallel with this modelling, a groundwater model was built for the same area to predict the groundwater high levels as a result of existing conditions. This groundwater model required inputs from the surface water model to determine soakage rates and locations that will recharge groundwater during storm conditions.

A hydrological model was built of the contributing catchments to the ponds to determine inflow rates. The biggest catchment of 100 ha of mostly industrial zoned land had a limited piped network and also had little anecdotal evidence of flooding and overland flows.

We were also advised by the CCC that there is limited formal stormwater network in this catchment. By inference there is enough evidence to show that a portion of rainfall does not connect or directly connect to the reticulated network and discharge to ground occurs within the catchment.

A hydraulic surface water model was built using DHI software. This part of the model routed the catchment flow through the existing and proposed ponds, weirs and orifices, to simulate the physical environment post CSM2 construction.

6.2 Summary of Model Outputs

From the modelled results we can conclude:

- The flow from the CSM2 impervious areas to the Ramp and Maize Maze Ponds can be fully contained for the 100 year 24 and 60 hour storm events without spill to Montgomery's Drain. The 60 hour event is considered critical for the Halswell River catchment.
- The flows generated from the spill from the recently modified HJR Pond currently flow down Montgomery's Drain. Once the Project is complete, the flows from the HJR Pond will be routed through the Owaka Basin with some further attenuation potentially available in the Wilmers Quarry Basin. The modelling so far shows a significant peak flow reduction as shown in Section 6.4 below.
- The modelling does not include the recently supplied Wilmers Road catchment and as such the final reduction will be of a lesser magnitude. Further modelling will be required to determine the extent of reduction but this is beyond the scope of determining the effects from the Project, and is a matter for the CCC.
- The pond volumes have been based upon the assumption that the ponds will be notionally empty at the beginning of the storm event. The groundwater study has concluded that for significant periods, the groundwater without intervention would be significantly above the pond base and as

such the attenuation and storage offered by the ponds (as currently designed) would not be available.

There is a groundwater intervention strategy proposed to ensure the notionally empty assumption holds for the life of the Project. The details are summarised below.

6.3 Groundwater Modelling Result Summary

The full groundwater study is attached as Appendix C to Technical Report 3. In summary the groundwater level has been predicted to rise to 19.6 m following the effects of Central Plains Water (CPW) at the CSM2 and CCC ponds location adjacent to HJR.

Further, that within 2 – 4 years following implementation of CPW, the effects of CPW will be felt to 90% of the maximum predictions. Thus within a short period of time we predict that the groundwater level will rise above the pond base level and without intervention, groundwater would flow into the pond base and reduce the available storage volume in the ponds.

6.4 Groundwater Intervention Strategy

In order to reduce the effects of this groundwater level rise compromising pond volumes, an intervention strategy is proposed. The outline of the proposal is as follows:

- A pond under drainage system is proposed to intercept rising groundwater and maintain groundwater at or below pond base level;
- The under drainage system will generally consist of:
  - Infiltration trenches placed and excavated around the perimeter of the Project Ponds and similarly around the CCC Owaka Basin;
  - A network of shallow wells may be needed in addition to, or instead of a network of infiltration trenches. This will be subject to final design;
- The above networks will be connected by a trunk drainage system separated from surface water drainage. This system will collect groundwater interception flows (and CSM2 under channel drainage) and convey this flow to Upper Knights Steam;
- The proposed pipe to Upper Knights Stream will need to be of sufficient size to allow the predicted groundwater flow with minimal head loss to ensure upstream levels in the trunk drainage system maintain the groundwater level at RL=18.0 m (i.e. 0.5 m below the Ramp Pond base level), the indicative size is 600 mm dia.;
- The outlet to the trunk drain has been set at RL = 17.0 m within the Upper Knights Stream some 500 m downstream of the John Paterson Drive/ HJR intersection.
- The groundwater trunk system will also need to collect under channel drainage from critically low pavement levels to ensure pavements remained dry even when the adjacent ponds are full.

With the intervention strategy in place the assumption that the ponds would be notionally dry at the commencement of the next storm event would be satisfied. As such the effects of runoff from the Project can be adequately mitigated for.

6.5 Further Work

This assessment has modelled the effects of surface water flow that affects the CSM2 project ponds adjacent to HJR. The catchment for the Wilmers Quarry was not available at the time of the initial model runs.

It is recommended that additional model runs be carried out at the detailed design phase for the Project and/or separately by the CCC in relation to Council infrastructure, to:

- Include the Wilmers Quarry catchment area and refine the modelled Wilmers Quarry site to reflect the final land shape (CCC matter);
- Refine the potential storage available in the HJR pond catchment (CCC matter);
- Modify the HJR Pond spill structure to ensure more water is released sooner in the storm cycle to reduce the peak spillway flow rate (CCC matter);
- Modify the Owaka Basin outlet to Montgomery's Drain to allow for a longer weir. (i.e. enlarge the manhole from 1050 mm dia. to 1800 mm dia.). This will ensure a lower peak water level in the Owaka Basin and as such provide greater freeboard to the adjacent pavement carriageway (Project matter);
- Detailed design of the under drainage system for all ponds (CCC and Project matter); and
- Secure a drainage path through private land for outlet to the Upper Knights Stream. This land is currently under the ownership of Fulton Hogan and is proposed to be vested in the CCC as reserve land (CCC and Project matter).

7. Discussion on Flooding Effects

7.1 Existing Environment

Prior to the construction of CSM1, flows in excess of the capacity of the Wilmers Quarry basin did reach Montgomery's Drain (quantity unknown). Post CSM1, overflows will reach the recently constructed Owaka Basin. With no constructed outlet to the Owaka Basin, the flows are contained within the basin.

At present, there is the potential for large flows to Montgomery's Drain from the HJR Pond catchment. Over time, development will occur and the flows will increase. Without diversion of flows to the Owaka Basin (the CCC project), flows in Montgomery's Drain have the potential to exceed 3 m^3 /second.

7.2 With Project

Post CSM2, the connections for Owaka Basin and Wilmers Quarry are added to enable the attenuation of flows out of the HJR Ponds. This is set out in the SWAP, but enabled through the construction of the Project. As a result, the modelled outflow to Montgomery's Drain will reduce significantly to less than 1 m^3 /second. It is noted that these flows are all generated up catchment of the Project.

The new run off generated from the Project is captured and stored in two new ponds (Maize Maze and Ramp Ponds) and in the existing CSM1 Mushroom Pond. These ponds have been designed to drain to ground. However, the inflow rate exceeds the drainage rate, and the pond will fill during the course of any storm. For the critical Q_{100} storm:

- Q_{100} 24 hour – there is no spill to Montgomery's Drain;
- Q_{100} 48 hour – there is a small spill to Montgomery's Drain late in the storm event, which occurs when the overflow from Owaka Basin is significantly reduced; and
- Q_{100} 60 hour – there is a small spill to Montgomery's Drain late in the storm event, which occurs when the overflow from Owaka Basin is significantly reduced.

It is concluded that discharges to Montgomery's Drain and Upper Knights Stream are significantly smaller than prior to the implementation of the Project.

Based on these flows being smaller than the existing situation, it is concluded that effects of flooding in the Halswell River will also be reduced by this Project and by the works set out in the SWAP, which are enabled by the Project.

7.3 Groundwater Interference

Prior to CSM2 and the effects of CPW (i.e. the existing environment) groundwater levels are historically below the zone of influence for the Project. However, with the consenting of CPW, it is now part of the "existing environment" for planning purposes. Once constructed, 90% of the CPW effects will be felt within 2-4 years.

The surface water storage within the Project ponds and the CCC ponds will be affected by groundwater, as high groundwater would flow into the ponds and reduce pond capacity. Without intervention via groundwater lowering, the conclusions on flood effects above could not be made.

Irrespective of CPW, groundwater in the HJR area drains to the lower reaches of Upper Knights Stream and the Halswell River. As groundwater levels rise, groundwater will flow to the upper reaches of Upper Knights Stream. There may be a modest increase in base flow rates in this stream and river entirely due to CPW.

In order to mitigate for the effects of CPW groundwater lowering is proposed. The intervention consists of a network of under drainage slotted pipes and/or shallow wells connected to a truck pipe that discharges to Upper Knights Stream. This will begin to operate by gravity when the groundwater rises above RL = 17.5 m. The effects of this can be summarised as follows:

- Without intervention – an equilibrium will be formed with groundwater flowing to surface water;
- With intervention – a new equilibrium will be formed but the flow rates will be similar.

As such, the groundwater intervention will not increase flow rates in the Upper Knights Stream or to the Halswell River.

8. Conclusion

This assessment has determined that the effects of the surface water regime and the groundwater regime have a potential impact on the flow regime in the Montgomery's Drain leading to the Halswell River.

However with intervention to limit the impact of future groundwater level rises as a result of the CPW scheme, the effects can be mitigated for.

The intervention strategy needed for the Project will also be required by the CCC for their Owaka Basin. Thus a joint CCC and NZTA programme will be required to address the effect of rising groundwater levels.

Based on the modelled flows to Montgomery's Drain and Upper Knights Stream being less than the existing situation, it is concluded that effects of flooding in the Halswell River will also be reduced by this Project and by the works set out in the SWAP, which are enabled by the Project.

9. References

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Appendix E | Ground Water Wells

Well_No	Well_Sta_1	Locality	Road_Or_St	Well_Owner (When Drilled)	Depth	Dia.	Date_Drill
M36/4025	Active (Exist, Present)	Weedons	Main South Rd	C A & L K Odering	54.0	150	Fri, 3 Mar 1989
M36/4734	Active (Exist, Present)	Weedons	Main South Road	Mr P Brien	53.7	150	Sat, 5 Mar 1994
M36/4040	Active (Exist, Present)	Weedons	Main South Rd	Welbeloved .N.	54.0	150	Mon, 22 May 1989
M36/4709	Not Used	Rolleston	Lowes Road	Townley Kd & Me	42.3	150	Sun, 1 May 1994
M36/4217	Active (Exist, Present)	Weedons	Weedons Ross Rd	Williams Mj & Nj	29.3	100	Tue, 1 Jul 1969
M36/4083	Active (Exist, Present)	Weedons	Weedons Ross Rd	Williams Mj & Nj	40.0	150	Sat, 1 Jul 1989
M36/2683	Active (Exist, Present)	Weedons	Jones Rd	Timargo Holdings Limited	51.8	152	Thu, 1 Jul 1965
M36/3737	Active (Exist, Present)	Weedons	Weedons Ross Rd	Fletcher Wf & Fp	36.4	150	Wed, 29 Apr 1987
M36/4675	Not Used	Weedons	Main South Road	Weeden	17.4	76	Tue, 1 Jul 1862
M36/2978	Active (Exist, Present)	Rolleston	Weedons Ross Rd	Bussel A.R.	36.0	150	Fri, 1 Apr 1983
M36/0288	Not Used	Rolleston	Weedons Ross Road	Bussel A.R.	21.0	76	
M36/1339	Active (Exist, Present)	Weedons	Main South Road R.D.5 Christchurch	C A & L K Odering	101.8	150	Tue, 30 Oct 2001
M36/0298	Active (Exist, Present)	Weedons	Weedons Ross Road		18.3	0	
M36/0124	Not Used	Weedons		Phillips A.	21.3	76	Wed, 1 Jul 1925
M36/4043	Active (Exist, Present)	Weedons	Main South Rd	Canterbury Chicken Limited	51.6	150	Sun, 1 Jan 1989
M36/0089	Not Used	Weedons	Main South Road	Hannah	13.3	51	Wed, 23 Nov 1960
M36/3953	Active (Exist, Present)	Weedons	Main South Rd	Pugh Cw & Jm	36.0	125	Wed, 1 Jun 1988
M36/3954	Not Used	Weedons	Main South Rd	Pugh Cw & Jm	0.0	76	
M36/3569	Not Used	Weedons	Main South Rd	Brinks South Island Limited	29.6	152	Tue, 1 Apr 1975
M36/3570	Not Used	Weedons	Main South Rd	Palmer Holdings	24.4	64	
M36/2712	Active (Exist, Present)	Weedons	Main South Rd	Mr & Mrs G D & S A Mcneill	48.3	150	Mon, 3 Dec 1984
M36/2230	Active (Exist, Present)	Weedons	Main South Road	Wilson G.L.	17.0	89	
M36/0258	Active (Exist, Present)	Weedons	Main South Rd	Wilson G.J.	50.4	200	Fri, 17 Aug 1973
M36/1634	Active (Exist, Present)	Weedons	S.H.1.	Van De Ven, A	45.2	150	Fri, 18 Jul 1980
M36/2231	Active (Exist, Present)	Weedons	Main South Road	Mr C S Warren	39.6	150	Mon, 17 Jul 1978
M36/4306	Active (Exist, Present)	Templeton	Shands Road	Hinton, G.E	48.0	150	Mon, 1 Jul 1991
M36/0125	Active (Exist, Present)	Weedons	Opp. Berkett's Road	Davidson Pl & Mb	21.5	83	Wed, 1 Jul 1925
M36/2711	Not Used	Weedons	Main South Rd	Odering C.A.	25.6	150	Mon, 1 Apr 1968
M36/3326	Active (Exist, Present)	Weedons	Main North Rd	Blue Gum Hotels	32.6	150	Tue, 1 Oct 1985
M36/0120	Not Used	Weedons	Main South Rd	Blue Gum Motels	21.9	100	Mon, 1 Jul 1963
M36/0314	Not Used	Weedons	Main South Rd	Warren L.C	24.0	83	
M36/4100	Active (Exist, Present)	Weedons	Main South Road	Warren, J.	48.0	150	Thu, 24 Aug 1989
M36/3164	Not Used	Templeton	Waterholes Rd	Mwd	17.0	51	

M36/1330	Active (Exist, Present)	Templeton	Waterholes Road	M.W.D.	28.4	150	Tue, 1 Mar 1983
M36/1978	Active (Exist, Present)	Templeton	Waterholes Rd	Stanley .R.	23.5	150	
M36/0271	Active (Exist, Present)	Templeton	Waterholes Road	Heald R	25.0	150	
M36/3875	Active (Exist, Present)	Templeton	Main South Rd	Southern Woods Nursery Li mited	38.0	150	Mon, 11 Apr 1988
M36/3071	Active (Exist, Present)	Templeton	Main South Rd	Mr & Mrs I I & I C Kim	163.0	300	Tue, 30 Sep 1986
M36/3165	Active (Exist, Present)	Templeton	Waterholes & Sh1	Turkeyville	0.0	100	
M36/2405	Active (Exist, Present)	Templeton	108 Trents Rd	Mr & Mrs B G & V P Nyhan	177.0	250	Sat, 28 Sep 1985
M36/3718	Not Used	Templeton	Trents Rd	Nyhan D.G.	0.0	51	Wed, 1 Jul 1970
M36/0719	Not Used	Templeton	Globe Lodge,108 Trents Road	Nyhan D.G.	27.4	38	
M36/2695	Active (Exist, Present)	Templeton	Hamptons Rd	Moore D.D.	21.9	76	
M36/5122	Active (Exist, Present)	Templeton	Hamptons Road	Moore, D.D	30.0	150	Fri, 1 Mar 1996
M36/0326	Not Used	Prebbleton	Hornby	Central Canty Elect	19.8	150	
M36/4229	Active (Exist, Present)	Templeton	Marshs Rd	Woodhaven Builders	62.5	95	Thu, 27 Sep 1990
M36/4580	Active (Exist, Present)	Prebbleton	Springs Rd	Wells G.P	19.3	150	Mon, 11 Jan 1993
M36/0911	Active (Exist, Present)	Prebbleton	Marshs Road	Tegel Foods Limited, Hornb y	26.0	150	Mon, 1 Jul 1974
M36/2594	Not Used	Halswell	Halswell Junction Road	Mcvicar Timber Ind	18.2	64	
M36/1066	Not Drilled	Hornby			0.0	0	
M36/0924	Active (Exist, Present)	Hornby	Halswell Junction Road	King P.	14.0	0	
M36/2806	Active (Exist, Present)	Prebbleton	Springs Road	Lee, Paul G.	13.1	76	Tue, 10 Apr 1979
M36/5638	No Info Expired Boreconse nt	Hornby	Cnr Halswell Junction Road And Spring s Road	Mcvicar Timber Group Ltd	15.0	150	
M36/5640	Active (Exist, Present)	Hornby	Cnr Halswell Junction Road And Spring s Road	Mcvicar Timber Group Ltd	12.0	150	Wed, 10 Jun 1998
M36/5639	Active (Exist, Present)	Hornby	Cnr Halswell Junction Road And Spring s Road.	Mcvicar Timber Group Ltd	11.0	51	Wed, 10 Jun 1998
M36/4395	Active (Exist, Present)	Halswell	388 Halswell Junction Rd	Harkess	15.3	100	Thu, 6 Jun 1991
M36/4396	Not Used	Halswell	388 Halswell Rd	Harkess	12.0	32	
M36/4394	Active (Exist, Present)	Halswell	Halswell Junction Rd	Hayward R.P	20.0	150	
M36/6880	No Info Expired Boreconse nt	Templeton	Waterholes Road	M D Shearer	40.0	150	
M36/6881	No Info Expired Boreconse nt	Templeton	Waterholes Road	M D Shearer	40.0	150	
M36/7096	Active (Exist, Present)	Weedons	Main South Road	Kim, Sj	36.0	150	Fri, 16 Feb 1996
M36/7136	Not Drilled	Christchurch	Po Box 1482	Transit Nz - Neil Sheerin	35.0	100	
M36/7295	Active (Exist, Present)	Weedons	Main South Road	Mr K A Hannah	42.0	150	Fri, 23 Aug 2002

M36/7311	Active (Exist, Present)	Weedons	Main South Road	Mr C S Warren	56.0	150	Fri, 20 Sep 2002
M36/7374	Active (Exist, Present)	Hornby	Marshs Road	Tegel Foods	15.0	51	Sun, 14 Mar 2004
M36/7458	Active (Exist, Present)	Templeton	Blakes Road	Mr & Ms A M & S Williams	42.0	150	Wed, 21 Jan 2004
M36/7502	Active (Exist, Present)	Prebbleton	John Paterson Drive	Mr & Ms G H & D J Clarke	17.3	150	Mon, 17 Nov 2003
M36/7545	Active (Exist, Present)	Templeton	Blakes Road	Mr & Ms V G & B A Morrison & O'malley	33.0	150	Fri, 30 Apr 2004
M36/7707	Active (Exist, Present)	Leeston	Weedons Ross Road	Mr & Mrs Mf & Kk Southern	42.0	150	Sat, 14 May 2005
M36/7708	Active (Exist, Present)	Templeton	109 Trents Road	Mr & Mrs Gj & Kp O'connor	64.3	150	Mon, 23 Aug 2004
M36/7721	Active (Exist, Present)	Christchurch	95 Berketts Road	Mr Gl Bowden	42.0	150	Mon, 21 Jun 2004
M36/7809	Active (Exist, Present)	Weedons	Lowes Road	Mrs Wl Fletcher	38.0	150	Mon, 1 Nov 2004
M36/7996	Active (Exist, Present)	Templeton	Sh 1 & Dawsons Road	Evans Family Trust	48.0	150	Wed, 9 Nov 2005
M36/8070	Active (Exist, Present)	Halswell	5 John Paterson Drive	Mr & Mrs Rj & Cb Sissons	18.0	150	Thu, 16 Mar 2006
M36/8218	Active (Exist, Present)	Weedons	31 Weedons Ross Road	Mr & Mrs Mcdonald & Lee-Mcdonald	54.0	150	Wed, 27 Sep 2006
M36/8226	Active (Exist, Present)	Weedons	Main South Road	Mr Richard Steel	47.4	150	Tue, 30 May 2006
M36/8329	Active (Exist, Present)	Weedons	782 Weedons Road	Mr Ge & Mrs Ej Doyle	54.0	150	Wed, 21 Feb 2007
M36/20150	Active (Exist, Present)	Halswell	Halswell Junction & Mcteigue Roads	Nz Transport Agency (Chch)	15.0	50	Mon, 20 Oct 2008
M36/20347	Landparcel Proposed	Templeton	Corner Main South Road & Curraghs Road	Curragh Holdings Ltd	40.0	150	
M36/20482	Capped (Semi-Permanent)	Rolleston	Main South Road	New Zealand Transport Agency	20.0	250	Thu, 14 Oct 2010
M36/20493	Active (Exist, Present)	Rolleston	Hamptons Road	New Zealand Transport Agency	20.0	200	Fri, 29 Oct 2010
M36/20495	Active (Exist, Present)	Rolleston	Trents Road	New Zealand Transport Agency	20.0	200	Mon, 8 Nov 2010
M36/20498	Active (Exist, Present)	Rolleston	Shands Road	New Zealand Transport Agency	20.0	125	Mon, 8 Nov 2010
M36/20501	Active (Exist, Present)	Rolleston	Marshs Road	New Zealand Transport Agency	20.5	125	Wed, 27 Oct 2010
M36/20504	Active (Exist, Present)	Rolleston	Springs Road	New Zealand Transport Agency	20.5	200	Thu, 21 Oct 2010
M36/20542	Active (Exist, Present)	Templeton	Main South Road	Mr G J Cross	48.0	150	Mon, 10 Jan 2011
Bx23/0015	Active (Exist, Present)	Halswell	515 Halswell Junction Road	Fonterra Brands Ltd	11.5	50	Tue, 29 Mar 2011
Bx23/0016	Active (Exist, Present)	Halswell	515 Halswell Junction Road	Fonterra Brands Ltd	11.5	50	Wed, 30 Mar 2011

Appendix F | Contaminant Load Assessment

Aqua Terra International Ltd.
1454 Kaipara Coast Highway
RD 4
Warkworth 0984
0274 904 134
earl.s@xtra.co.nz
www.aquaterra.co.nz

Memorandum

To: Mary O'Callahan, Gary Payne, Craig Redmond, Tony Miller
From: Earl Shaver
Date: 11 October 2012
Re: CSM2 and MSRFL contaminant load discussion

The following items are provided to discuss:

- Assumptions made in assessing contaminant load generation,
- Estimated loads,
- Removal efficiency of vegetated swales,
- Groundwater discharge, and
- Potential impacts to groundwater.

Assumptions made in assessing contaminant load generation

- Length of new roadway (MSRFL and CSM2) is 12.9 km.
- Traffic load 55,000 vehicles/day for CSM2 and approximately 45,000 vehicles/day from the MSRFL.
- Contaminants considered include Total Suspended Solids (TSS), zinc, copper, Total Petroleum Hydrocarbons TPH, and pH.

Contaminant loads relating to oxygen demanding substances, nutrients, pathogens and pesticides are not being considered further as they are not generally associated with highway generated contaminants. Litter is also not considered as its control will be done by highway maintenance and should not be an issue for impacts to groundwater receiving systems.

Estimated loads

Contaminant loads could be considered using two methods, the Auckland Council's Contaminant Load Model, 2006 and by an approach recommended (Moores, Pattinson, Hyde, 2010 – hereafter called the Moore's study or calculation) that is specific to zinc and copper load generation from motorways. It is considered more accurate for these metals than the Auckland Council's version as the Auckland Council's version has contaminant loads for highways determined from central Auckland streets. These loads would be expected to be considerably higher relating to continual accelerating and braking, which would contribute greater loads than from the project being discussed.

As such the Moore's calculation will be used here for zinc and copper while the Auckland model will be used for TSS, which was not calculated in the Moore's approach. TPH was monitored but it was not detected in the majority of runoff samples so load predictions were not made for TPH. As a frame of reference, TPH loads are calculated by the Auckland model.

Using these approaches estimates for TSS, zinc, copper and TPH loads/year are the following:

- TSS – 92,880 kg/year
- Zinc (normal traffic) – 67.9 kg/year
- Zinc (congested traffic and intersections) – 150.3 kg/year
- Copper (normal traffic) – 11.2 kg/year
- Copper (congested traffic and intersections – 23.01 kg/year
- TPH – 5,832 kg/year

Assuming a mix of congested traffic (25%) and normal traffic (75%) the annual loads for the CSM2 and MSRFL project are (from the Moore's study) the following:

- Zinc – 89.12 kg/year
- Copper – 14.2 kg/year

Zinc and copper are used as surrogates for the spectrum of metals due to their ability to be attached to sediments and to be in a soluble form. Capture of these contaminants would mean that other metals are captured to a greater extent.

A key point regarding TPH is that it was monitored in the Moore's study but was not detected in the majority of runoff samples. One possible reason for this is the use of open graded asphalt as a pavement overlay for skid resistance. As a result TPH may well be trapped in the interstices.

pH has been measured on a number of projects internationally and while there can be variations from about 5.1 – 8 pH units, pH is generally a nearly neutral solution from highways. (Granato and Smith, 1999)(Pacific EcoRisk, 2007) so should not be a concern on this project.

Removal efficiency of vegetated swales

Swales have been the subject of numerous studies both in New Zealand and internationally. Again using the recent Auckland region highway study (Moore's study) they specifically looked at swales receiving road runoff and conducted site monitoring to determine removal rates for total copper and zinc. Contaminant removal of TSS by swales was done using the Auckland Council's TP 10, which is 75%. The NZTA stormwater treatment standard uses a similar design approach and should achieve the same treatment expectations.

The following table 1 provides removal expectations of swales for the contaminants listed above.

Table 1
Contaminant Removal Estimates for CSM2 swales

Contaminant	Load (kg/year)	Load Reduction Factor	Load potentially exported (kg/year)
TSS	92,880	0.75	23,220
Zinc	89.12	0.8	17.8
Copper	14.2	0.8	2.8
TPH	5,832	0.57 (AC contaminant model)	2,506

It should be noted that the table values are for the entire alignment that is 12.9 km long. A more reasonable estimate would be to consider unit loadings per hectare (paved area covers approximately 28.4 ha) since swales are designed to accommodate small catchment areas. There will not be one surface point where all the flow is taken to so consideration of load/hectare is reasonable. The load per hectare is given in the following table 2.

Table 2
Contaminant Removal Estimates per hectare for CSM2 swales

Contaminant	Load (kg/year)	Load Reduction Factor	Load potentially exported (kg/year)
TSS	3,270	0.75	818
Zinc	3.1	0.8	0.62
Copper	0.5	0.8	0.1
TPH	205	0.57 (AC contaminant model)	88

As can be seen these values are small from a metals context. There is a variation between performance expectations provided in the Moore's report and the groundwater contaminant modelling done by BECA on the MSRFL and CSM2 project (3 October 2012). BECA conservatively assumes for the purpose of the groundwater monitoring that the swales will treat 50% of the inflow copper concentration in the runoff and 70% of the zinc concentration. I feel that the Moore's study is more reflective of performance than their recommendations. The organic material in swales is very effective at removal of metals (particulate and soluble).

Regarding the BECA report in conjunction with this one gives good correlation. Their approach was based on the use of a simple groundwater model. This report is based on generating contaminant loads from a landuse and treatment perspective. Their groundwater study used contaminant concentrations while this report generally uses annual loads. The end result is the same. Using both approaches groundwater related impacts are expected to be less than minor. The studies complement each other well.

Issues related to TSS and TPH will be discussed in the next section as ground is the receiving environment.

Groundwater discharge

Consideration of discharge of contaminants to ground depends on the particulate associated or dissolved contaminants. The potential for contamination is of greatest concern in areas with well-drained soils, typically sand with low organic content. In association with soils another potential problem is where the water table is shallow.

The TSS load exported is not an important issue as sediment will be effectively trapped in the soil matrix. This results in a maintenance issue rather than a groundwater discharge issue.

The prediction of groundwater contamination potential can be very complex and it depends on the concentration and form of the contaminant, the characteristics of the soil, and the rate that water moves through the soil. Swale function depends on vegetation and organic soils and in situations where water discharges to ground the organic soils play a very important role.

Pretreatment of contaminants by the swales will reduce particulate and soluble fractions of contaminants, which increases the duration that the vadose zone soils can capture or sorb filterable contaminants by ion exchange or other processes. The organic matter in swales is reactive to passing contaminants by having a high cation exchange capacity and provides good pretreatment prior to water infiltration. Cation exchange capacity can decrease over time and the ability of swales to continually generate organic matter assists in maintenance of the cation exchange capacity.

Potential impacts to groundwater

Consideration of international studies related to migration of metals in swales overlying sandy soils indicates that substantial accumulation of zinc, lead and copper occurred in the top 100 mm of organic matter. Major characteristics that affect potential groundwater contamination are the following:

- High mobility (low sorption potential) in the vadose zone,
- High abundance (high concentrations) of contaminants in stormwater, and
- High soluble fractions.

Mobility is based on the partitioning coefficients, k_D values due to soils having a range of values for their chemical properties. Zinc and copper are classed as having low mobility, even in sandy soils (Clark, Pitt, 2009). That mobility is especially low when pretreatment is provided by swales.

In the same study (Moores study) estimates of concentrations of zinc and copper were found. While there was some variation from site to site, the following values for runoff treated by swales were found.

- Copper – 7.5 mg/m^3
- Zinc – 20 mg/m^3

Using the drinking water standards as a guide, the limit for drinking water for zinc is 1,500 mg/m^3 so the impacts on any groundwater supply should be less than minor.

In a similar fashion, the maximum acceptable value for copper in drinking water is 2,000 mg/m^3 which is much higher than anticipated copper concentrations.

In addition, infiltration processes also reduce contaminant potential to groundwater. Another point from a study already mentioned (Clark, Pitt, May, 2009) provided median influent and effluent concentrations related to infiltration practices. Using zinc as a surrogate, they found influent concentrations of 69 mg/l and effluent concentrations of 32 mg/l. These results indicate additional contaminant capture is provided by the soil matrix.

TPH has hundreds of compounds and they generally are not monitored individually in stormwater monitoring so compliance with drinking water standards are not provided here. Monitoring of TPH in general does show significant reduction of TPH as the compounds flow through a coarse media such as sand. One sand filter study (Shaver, 1994) monitored TPH removal from a sand filter above 90%. Given Christchurch climate it can be expected that volatilisation will be an important process in TPH reduction.

Overall, groundwater contamination issues should be less than minor from this project.

References

Clark, S.E., Pitt, R., May 2009, Development of a Guidance Document for Developing Strategies for Managing Municipal Stormwater: Trade-offs Between

Infiltration and Discharge to Surface Waters, Water Environment Research Foundation.

Granato, G.E., Smith, K.P., 1999, Estimating Concentrations of Road-Salt Constituents in Highway Runoff from Measurements of Specific Conductance, Water Resources Investigation Report 99-4077, U.S. Department of Interior, U.S. Geological Survey.

Moores, J., Pattinson, P., Hyde, C., March 2010, Enhancing the control of contaminants from New Zealand's Roads: Results of a road runoff sampling programme, National Institute of Water and Atmospheric Research Ltd., New Zealand Transport Agency research report 395.

Pacific EcoRisk, 2007, Potential Effects of Highway Runoff on Priority Fish Species in Western Washington, Washington State Department of Transportation.

Shaver, E., (1994), Sand Filter Design for Water Quality Treatment, Urban Runoff and Receiving Systems, (Herricks, E. ed.), ASCE, New York

Appendix G | Contaminant Modelling Report

NZ TRANSPORT AGENCY
WAKA KOTAHI

Appendix G, Technical Report No 3

Christchurch Southern Motorway Stage 2 and Main South Road Four Laning

Assessment of Groundwater Quality Effects

November 2012

Quality Assurance Statement			
	Prepared by:	Terry Widdowson	November 2012
	Reviewed by:	Mark Utting	November 2012
	Approved by:	Graham Levy	November 2012

Contents

Appendices

Abbreviation	Description
MSRFL	Main South Road Four Laning
CSM2	Christchurch Southern Motorway – Stage 2
Beca	Beca Infrastructure Ltd
CPW	Central Plains Water Enhancement Scheme
CSM1	Christchurch Southern Motorway – Stage 1
CSM2	Christchurch Southern Motorway – Stage 2
ECan	Environment Canterbury the publicity name of Canterbury Regional Council
EMC	Event Mean Concentrations
GHD	GHD Limited
MfE	Ministry for the Environment
NZTA	NZ Transport Agency
MSRFL	Main South Road Four Laning
PAH	Polycyclic aromatic hydrocarbons

Executive Summary

An assessment of effects of stormwater discharge on groundwater quality has been undertaken to supplement the assessment of groundwater effects associated with changes in groundwater level during operation of the Christchurch Southern Motorway 2 (CSM2) and Main South Road Four Laning (MSRFL) project. This assessment has relied on information sourced and discussed in parallel geotechnical (Technical Report No. 11) and hydrogeological investigations (Appendix C to Technical Report No. 3) and assessments undertaken for the project.

This assessment has modelled the effect on groundwater quality of stormwater contaminants produced from road runoff during operation of the project. The road runoff is to be collected in swales which run along the 8.4km length of the CSM2 alignment and 4.5km length of the MSRFL. The swales are designed to treat the stormwater by removing and reducing contaminants before the stormwater either soaks into the sub soil beneath the alignment or discharges to collection ponds.

The geology beneath the proposed alignment can be summarised as a sequence of sands and gravels with varying amounts of silt and clay. The upper part of the sequence comprises a relatively permeable unconfined aquifer with thickness in the order of 20+m. Groundwater depth below the proposed alignment varies between 3m and 17m below ground level. Groundwater is flowing in a general south-easterly direction with a shallow gradient. The groundwater is abstracted locally for use as drinking water and irrigation. Most drinking water abstractions are from depths greater than 20m but there are a few that are shallower.

The model used to assess the effects comprises a series of Microsoft Excel worksheets developed by the UK Environment Agency. These worksheets allow contaminants to be modelled as they migrate from the soil source zone to groundwater and then within groundwater to a selected point where the groundwater is utilised or discharges into a sensitive environment. The contaminants modelled were copper, zinc and the polycyclic aromatic hydrocarbons (PAH), pyrene and fluoranthene. These contaminants have been selected based on a literature review of typical road runoff contaminants.

The estimated concentrations of copper and zinc in stormwater are less than their NZ Drinking Water Standard values. Therefore, copper and zinc in stormwater discharged from the proposed alignment pose low risk to groundwater used for potable supply.

Risk assessment of pyrene and fluoranthene has indicated that when dilution in groundwater beneath the alignment and attenuation along the groundwater flow path is considered, these contaminants pose low risk to groundwater used for potable supply. This is valid for wells that are located 30m or more from the designation boundary. According to information supplied by Environment Canterbury, there are 17 wells within 30m of the designation boundary that may be affected by stormwater discharge.

1. Introduction

The NZ Transport Agency (NZTA) has engaged GHD Ltd (GHD) and its sub-consultant, Beca Infrastructure Ltd (Beca) to undertake an Assessment of Environmental Effects of the proposed Christchurch Southern Motorway Stage 2 (CSM2). The proposed CSM2 is for a new four lane motorway extending from CSM1 at Halswell Junction Road for approximately 8 km to join Main South Road (SH1) to be "four laned" (MSRFL) near Robinsons Road. The combined CSM2 and MSRFL are collectively referred to as "the Project."

The construction of the project has the potential to affect the quality of groundwater used for potable supply. This report identifies the possible effects that could occur and discusses the results of quantitative risk assessment of those effects. This report should be read in conjunction with the Assessment of Groundwater Effects – Appendix C to Technical Report 3, and with Technical Report 3: Assessment of Stormwater Disposal and Water Quality Environmental Effects, which describes the stormwater management system.

2. Existing Environment

2.1 Geology

Technical report 11 (Geotech Engineering and Geo-Hazard Report) presents an overview of the geology of the project site and should be consulted for details. In summary, the upper 20+ m of the project site is underlain by alluvial gravels, sands, silts and clays of the Springston Formation. Glacial outwash deposits of gravels, sands, silts and clays of the Burnham Formation lie directly beneath. The geology relevant to this assessment is described in more detail below.

The Yaldhurst Member of the Springston Formation underlies much of the alignment. Containing significant percentages of silts, sandy silts and clay, this finer-grained member forms the surficial layer over much of the project area, especially the eastern end, and is generally about 0.1m to 2.2m thick.

The Halkett Member of the Springston Formation underlies the entire project area, directly beneath the Yaldhurst Member (where present) or from ground surface downward where the Yaldhurst Member is absent. In the vicinity of the project, the Halkett Member is coarser than the Yaldhurst Member and consists of sandy gravel to sandy gravel with cobbles with varying silt content to depths of greater than 21.5m below ground level, as indicated by investigation boreholes along the proposed project alignment.

2.2 Hydrogeology

A general overview of the hydrogeology of the project area is given in Technical Report 3 (Assessment of Stormwater Disposal and Water Quality Environmental Effects (GHD, 2012). In summary, groundwater beneath the project area flows through a series of unconfined, semi-confined and confined aquifers consisting of permeable sands and gravels (with cobbly zones) separated by leaky aquitards consisting of silts, clays and fine sands. The Yaldhurst Member of the Springston Formation with its generally lower permeability tends to form a confining layer to the east of the CSM2 project area. The Halkett Member with its generally higher permeability forms the shallow, unconfined aquifer beneath the project area ("Aquifer 1" as designated by ECan). The Halkett Member (Aquifer 1) becomes locally confined, where the groundwater level rises above the fine-grained base of the overlying Yaldhurst Member.

The depth to the water table within the uppermost aquifer beneath the alignment varies from 3m to 17m below ground level depending on the year, season and position (east to west) along the alignment. Analysis of groundwater flow directions, hydraulic properties of the aquifer materials and water levels is provided below.

2.2.1 Groundwater Flow Direction and Gradients

Shallow groundwater beneath the proposed CSM2 alignment flows generally from the west-northwest toward the east-southeast as indicated by both regional and project-specific analyses. Groundwater gradients have been estimated at about 0.002 beneath the western end of CSM2 (near Robinsons Road) and about 0.001 beneath the eastern end (near Halswell Junction Road).

2.2.2 Hydraulic Properties of the Subsurface Materials

The Halkett Member of the Springston Formation has been adopted for modelling purposes as it forms the main unconfined aquifer beneath the project. The hydraulic properties of the Halkett Member are discussed in Beca report Assessment of Groundwater Effects – Christchurch Southern Motorway Stage 2 (CSM2), 2012. The aquifer properties relevant to this assessment are given in Table 1.

Table 1 Hydrogeological Properties Used in the Assessment

Property	Value	Comment
Hydraulic conductivity	1×10^{-3} m/s	As calibrated in the groundwater effects report
Hydraulic gradient	0.0015	The mean from the range in the groundwater effects report
Effective porosity	0.2	For unconfined aquifer, equal to specific yield

3. Potential Groundwater Issues

The potential effect on groundwater being assessed in this report is the introduction of contaminants to the groundwater system from infiltrated stormwater discharge. The stormwater discharge is collected in swales running the length of the alignment on both sides of the road. The swales are designed to allow stormwater to infiltrate into the underlying soils.

There are numerous wells within 100m of the alignment used to abstract groundwater for irrigation and potable water supply. The majority of the wells used to abstract drinking water are screened below 20m. However there are a number of wells which are abstracting groundwater from less than 20m. In addition, there are wells for which there is no information on water abstraction depth.

These wells are shown on the plan in Appendix A. This assessment considers the risks to potable water supplies only.

4. Contaminant Concentrations in Road Runoff

4.1 Estimating Contaminant Concentrations in Stormwater

A literature review of both national and international studies was undertaken to determine representative concentrations of contaminants likely to be discharging from the project roadway. The results of the literature review are shown in Table 2. In the table, average contaminant concentrations are given as either EMCs (Event Mean Concentrations), means or median values. The EMC is defined as the total mass discharged during an event per unit total volume of stormwater discharged. It is therefore a flow weighted average of contaminant concentrations across all storm events.

The most significant contaminants in road runoff are copper, zinc and hydrocarbon compounds. Traffic volumes are often cited as having the greatest influence on the contaminant concentrations in road runoff. However, it is not necessarily the best, or the only, measure of contaminant concentrations (Moores et al., 2009). Other factors include vehicle movement patterns such as braking, rainfall characteristics, antecedent dry periods, road surface permeability and design. Roads with intersections, roundabouts, or more congestion exhibit traffic behaviour that includes higher braking rates and tyre wear. Contaminant concentrations in road runoff therefore vary widely as shown in Table 2.

To derive representative concentrations of copper and zinc (before stormwater treatment), the median values of those reported in Table 2 were used. Calculating the median instead of the mean allows for outliers to be considered without overly skewing the result. It is also useful for populations with a large range. The median values derived were 0.034 mg/l for copper and 0.19 mg/l for zinc.

For hydrocarbon contaminants there is limited data. For this assessment, the highest concentrations of dissolved pyrene (0.00012 mg/l) and fluoranthene (0.00018 mg/l) were used from Table 2 to represent hydrocarbon contaminants. Pyrene and fluoranthene are polycyclic aromatic hydrocarbons (PAH) and are associated with diesel fuel and motor oils.

Table 2 Average Concentrations of Contaminants in Road Runoff from Literature Review

Source		Total Copper (mg/l)	Total Zinc (mg/l)	Dissolved PAH (mg/l)	Reference
SH1 Motorway	Porirua	0.080	0.060	0.000308	Sheriff 1998
SH1 Northern Motorway	Silverdale	0.060	0.19	–	Larcombe 2003
SH1 Corridor	Otahuhu	0.053	0.159	0.00002	Kennedy, 2003
Urban highway	Texas	0.037	0.222	–	Barrett *et al.*, 1998

Source		Total Copper (mg/l)	Total Zinc (mg/l)	Dissolved PAH (mg/l)	Reference
Two-lane arterial road	SH18 @ Westgate	0.025	0.125	-	Moores *et al.*, 2009
Northern Motorway	SH1 @ Northcote	0.0153	0.034	-	Moores *et al.*, 2009
Two-lane rural highway	SH1 @ Huapai	0.0183	0.076	-	Moores *et al.*, 2009
Four-lane rural motorway	SH1 @ Redvale	0.0158	0.066	-	Moores et al., 2009
Highway Runoff	California	0.048	0.208	-	Caltrans, 2002
Free Flowing Highway	California	0.0383	0.222	-	Caltrans, 2002
US Highways	US	0.021	0.111	-	Kayhanian *et al.*, 2007
Roads, Genoa	Genoa	0.019	0.081	-	Gnecco *et al.*, 2005
Round about	Hamilton	0.027	0.284	Pyrene (0.00012) Fluoranthene (0.00018)	Pandey *et al.*, 2005
Intersection	Cambridge	0.012	0.195	Pyrene (0.00004) Fluoranthene (0.00014)	Pandey *et al.*, 2005
Roundabout	Tauranga	0.096	0.718	-	Taylor *et al.*, 2004
Highways	Osaka	0.066	0.648	-	Shinya *et al.*, 2000
Freeways	US	0.0347	0.200	-	Pitt *et al.*, 2004
Median	-	**0.034**	**0.19**	-	

4.2 Stormwater Treatment by Swales

Runoff from the project road area will discharge to vegetated swales constructed on either side of the roadway. Swales are grass or vegetated channels used to treat stormwater runoff. As runoff passes along the swales, contaminants are removed by filtration, infiltration, absorption and biological uptake. The vegetation also reduces the velocity of the flow allowing particulates to settle out.

In general, studies have found that swales are effective at removing solids, oils and heavy organics from stormwater runoff. However they are not as effective at removing soluble metal species, nutrients and bacteria. There are large variations between studies as to the reported effectiveness of swales as stormwater treatment devices. This is likely to reflect not only differences in the design and maintenance of these systems, but also the experimental design of the studies reviewed (Moore et al., 2009).

Further information on stormwater treatment is provided in the contaminant load discussion in Appendix F of Technical Report 3.

5. Water Quality Guidelines

The water quality guidelines for the stormwater contaminants identified in Section 4 are detailed in Table 3 below.

Table 3 Water Quality Guidelines for Stormwater Contaminants

Contaminant	Water Quality Standard	Value (mg/l)
Copper	Drinking Water Standards for New Zealand 2005 (revised 2008), Ministry of Health (Guideline Value)	1.0
Zinc	Drinking Water Standards for New Zealand 2005 (revised 2008), Ministry of Health (Guideline Value)	1.5
Pyrene	CCME (Canadian Councils for Ministers of the Environment), 2002, Canadian Environmental Quality Guidelines	0.000025
Fluoranthene	CCME (Canadian Councils for Ministers of the Environment), 2002, Canadian Environmental Quality Guidelines	0.00004

The guidelines were selected in accordance with the Ministry for the Environment (MfE) Contaminated Land Management Guidelines No. 2, Hierarchy and Application in New Zealand of Environmental Guideline Values, 2011.

6. Risk Assessment Methodology

6.1 How the Model Works

The groundwater risk assessment has been undertaken using a model developed by the UK Environment Agency. The model is described in Remedial Targets Methodology: Hydrogeological Risk Assessment for Land Contamination, 2006. It operates in Microsoft Excel as a series of worksheets.

The model uses a staged approach to determine risk-based remedial targets for soil and groundwater. A remedial target is derived at each stage (referred to as Levels) but this target is likely to be less stringent at the next Level as additional processes such as dilution and attenuation, which affect contaminant concentrations along the pathway from source to receptor, are considered. The staged approach for soils can be summarised as follows:

Level 1 – considers whether concentrations of contaminants in pore water (water in the unsaturated zone above the water table) are sufficient to impact the receptor.

Level 2 – considers whether dilution by groundwater flow beneath the source area is sufficient to reduce contaminant concentrations to acceptable levels.

Level 3 – considers whether attenuation in the aquifer down gradient of the source area is sufficient to reduce contaminant concentrations to acceptable levels.

The model works by calculating the concentration of a contaminant, the Remedial Target, that is acceptable at the source in order to achieve the Target Concentration at the Compliance Point. Simply put, it works backwards from the receptor (the compliance point) to define the acceptable concentration of a contaminant at the source which will meet the environmental standard at the receptor. It should be noted that the term 'Remedial Target' does not imply that remediation is or has taken place it is just the terminology used in the model.

The target concentration is the applicable environmental standard for the contaminant of concern. The compliance point is the point (location) where the environmental standard is applied. The compliance point changes as the assessment moves through the Levels: for Level 1 it is the soil zone where the contaminant is leached by pore water, for Level 2 it is groundwater beneath the source; for Level 3 it is a selected location down hydraulic gradient of the source, such as a groundwater abstraction well or spring discharge.

At each Level of assessment, the remedial target is compared to the source concentration which, in this case, is the estimated stormwater concentration. If the remedial target is greater than the source concentration, then compliance is achieved and no further action is required. This is because the remedial target is the concentration at the source which will achieve the environmental standard at the receptor, so if the source concentration is below the remedial target then the environmental effect can be stated as 'less than minor'.

If compliance is not achieved, and it is appropriate to do so, then the assessment moves to the next Level.

6.2 Model Assumptions

When conceptualising the contaminant source-pathway-receptor relationships, certain assumptions have to be made. These assumptions dictate how the model will be configured and how specific calculations will be performed. The assumptions used are detailed below:

- Background groundwater concentrations of modelled contaminants are zero. It is standard practice to set background concentrations for hydrocarbon contaminants at zero. Background groundwater concentrations of copper and zinc were not established and any inputs from stormwater discharge would be in addition to background concentrations.
- There is no allowance for biodegradation of pyrene and fluoranthene in the subsurface even though these compounds are known to biodegrade. The main reason for not considering biodegradation is the relatively rapid groundwater velocities expected to occur in the subsurface. This adds some conservatism to the assessment.
- There is no allowance for attenuation of contaminants within the unsaturated zone (from the base of the swale to the water table). The reasons for this are that (a) migration of infiltrated stormwater is likely to be relatively rapid with minimal fines (clays and silts) to adsorb contaminants and (b) the field data required for the calculations was not available. This adds some conservatism to the assessment.
- Swales are assumed to run along both sides of the alignment (soakage pits are at the design spacing – 30m length every 300m) with fixed width of 2m either side.
- The Level 3 assessment calculates the cumulative effect of stormwater discharge at the down hydraulic gradient end of the 12.9km route, as the route is essentially aligned in the direction of groundwater flow. This means that for compliance points located along and perpendicular to the route, the assessment is conservative.
- The infiltration rate is based on the annual average rainfall value of 680mm/yr and does not account for flood events. Such events would add considerable dilution to the stormwater discharge and result in rapid transport of stormwater to the overflow ponds.
- Values for dispersivity are based on empirical equations within the model.

6.3 Model Input Parameters

Details of the parameters used in the risk assessment are given in Table 4 for the Level 2 assessment and Table 5 for the Level 3 assessment.

Table 4 Parameter Values used in the Level 2 Assessment

Parameter	Unit	Value	Source
Hydraulic conductivity	m/d	86	Groundwater effects report
Hydraulic gradient	-	0.0015	Groundwater effects report
Saturated aquifer thickness	m	20	Groundwater effects report
Mixing zone thickness	m	10	Assumed half of aquifer thickness
Length of contaminant source parallel to groundwater flow	m	720	Calculated from stormwater report
Width of contaminant source perpendicular to groundwater flow	m	4	Stormwater report
Infiltration	m/d	0.0019	Groundwater effects report

Table 5 Parameter Values used in the Level 3 Assessment

Parameter	Unit	Value	Source
Hydraulic conductivity	m/d	86	Groundwater effects report
Hydraulic gradient		0.0015	Groundwater effects report
Width of contaminant source perpendicular to groundwater flow	m	4	Stormwater report
Thickness of contaminant source	m	10	Equal to mixing zone thickness
Bulk density	g/cm^3	2.2	Literature[1]
Effective porosity	fraction	0.2	Groundwater effects report
Fraction of organic carbon	fraction	0.001	Estimated (conservative)
Distance to compliance point	m	20	Project data
Longitudinal dispersivity	m	2	10% of distance to compliance point
Transverse dispersivity	m	0.2	1% of distance to compliance point

[1] Dann et al, 2009. Characterisation and Estimation of Hydraulic Properties in an Alluvial Gravel Vadose Zone, Vadose Zone Journal 8:651-653.

Parameter	Unit	Value	Source
Vertical dispersivity	m	0.02	0.1% of distance to compliance point
Organic carbon partition coefficient			
Pyrene	l/kg	68000	website
Fluoranthene	l/kg	49000	website

The estimated contaminant concentrations within the stormwater discharge and the relevant guideline values are summarised in Table 6.

Table 6 Contaminant Concentrations and Guideline Values

Contaminant	Concentration in Stormwater (mg/l)	Guideline Value (mg/l)
Copper	0.034	1.0
Zinc	0.19	1.5
Pyrene	0.00012	0.000025
Fluoranthene	0.00018	0.00004

7. Assessment of Environmental Effects

7.1 Level 1 Assessment

All contaminants were subject to Level 1 assessment on the basis that the concentrations defined in Table 6 were equivalent to pore water concentrations (this is conservative as the concentrations used here are pre-treatment within the swales and swales are designed to remove particulates from the stormwater discharge). This is essentially the same as the leached component from a contaminant source within the soil.

Concentrations of copper (0.034 mg/l) and zinc (0.19 mg/l) within stormwater discharge are below their drinking water quality guideline values of 1 mg/l and 1.5 mg/l respectively. Therefore copper and zinc within stormwater discharge pose low risk to groundwater used for potable supply and they are not considered further within the assessment.

Concentrations of pyrene (0.00012 mg/l) and fluoranthene (0.00018 mg/l) within stormwater discharge are both greater than their adopted drinking water quality guideline values of 0.000025 mg/l and 0.00004 mg/l respectively. These contaminants were therefore subject to Level 2 assessment and the results of the assessment are discussed below.

The results of the Level 1 assessment for each contaminant are shown on the worksheets in Appendix B.

7.2 Level 2 Assessment

The results of the Level 2 assessment are summarised in Table 7, the remedial targets for both pyrene and fluoranthene are less than their stormwater concentrations. This means that dilution of the stormwater contaminants in groundwater beneath the swales is not reducing the concentrations to acceptable levels. Compliance was therefore not achieved and the contaminants were subject to Level 3 assessment.

It was considered appropriate to move on to the next Level of assessment on the basis that Level 2 assesses compliance with groundwater directly beneath the swales. However, groundwater is being abstracted for potable supply at distances down hydraulic gradient of the swale discharge.

Table 7 Determination of Level 2 Remedial Target

Contaminant	Remedial Target (mg/l)	Stormwater Concentration (mg/l)
Pyrene	0.000049	0.00012
Fluoranthene	0.000078	0.00018

Note: If Remedial Target > Stormwater Concentration then compliance is achieved

The results of the Level 2 assessment for each contaminant are shown on the worksheets in Appendix C.

7.3 Level 3 Assessment

The results of the Level 3 assessment are summarised in Table 8, the remedial targets for both pyrene and fluoranthene are greater than their stormwater concentrations. This means that attenuation of the contaminants along the groundwater flow path is reducing the concentrations to acceptable levels. Reviewing the Level 3 assessment outputs indicates that the main mechanism for attenuation is dispersion along the groundwater flow path. Dispersion is the spreading of the contaminant plume as it moves through the aquifer, causing it to 'thin' and thereby reduce contaminant concentrations. The spreading occurs in three dimensions.

Compliance is therefore achieved for groundwater abstraction wells at a minimum distance of 30m from the designation boundary.

Table 8 Determination of Level 3 Remedial Target

Contaminant	Remedial Target (mg/l)	Stormwater Concentration (mg/l)
Pyrene	0.00013	0.00012
Fluoranthene	0.00021	0.00018

Note: If Remedial Target > Stormwater Concentration then compliance is achieved

The results of the Level 3 assessment for each contaminant are shown on the worksheets in Appendix D.

8. Conclusions

This groundwater risk assessment has assessed the effects of stormwater discharge along the alignment on groundwater abstraction wells within the vicinity of the alignment.

The assessment has calculated that the effects of stormwater discharge on potable groundwater quality are low at a distance greater than 30m from the designation boundary. This means that existing groundwater abstraction wells for potable supply should not be adversely affected by the operation of the project provided they are at least 30m distance from the designation boundary.

According to information provided from ECan's wells database, there are 17 wells within 30m of the designation boundary which are potentially used for drinking water supply. These wells are shown on the plan in Appendix A and the table in Appendix E.

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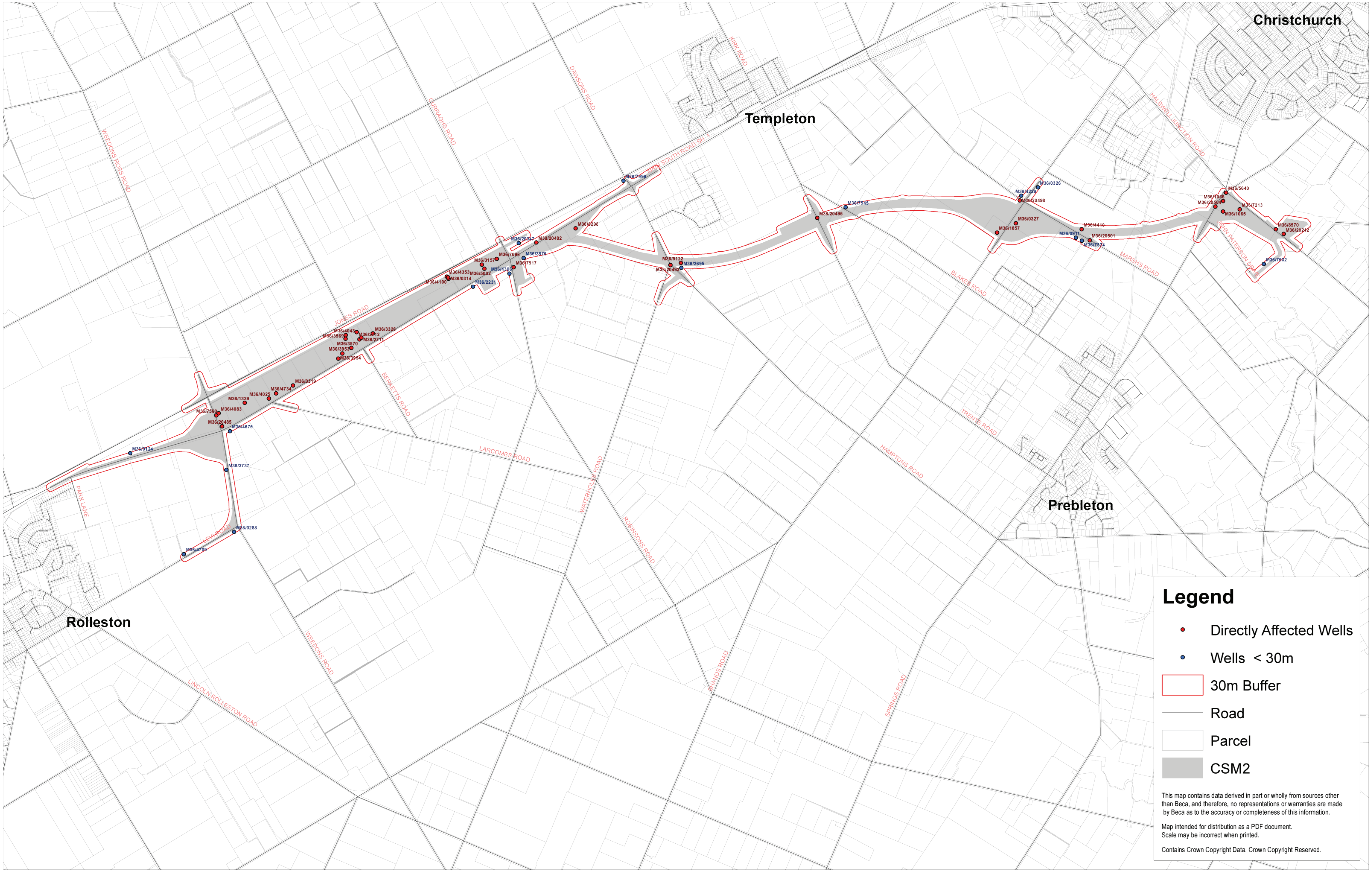
Appendix A

Location of Groundwater Abstraction Wells Along the CSM2 Alignment



GIS@beca.com

File: P:\3390\3390691\TGI\55_Workspaces\01_mxd\3390691-1-3.mxd Author: Beca GIS Date: 12/10/2012

Map Scale @ A1: 1:16,000

0 0.25 0.5 0.75 1 Kilometres

N

Revision	Author	Verified	Approved	Date
0.1	BGP	BAP	AH	12/10/2012

Title:

Wells Potentially Affected by Stormwater Discharge

Client: NZTA

Project: Christchurch Southern Motorway 2 & 3

Beca

Discipline: GIS

Drawing No: GIS-3390691-1-3

Appendix B

Worksheets for Level 1 Risk Assessment

Remedial Targets Worksheet , Release 3.1

Level 1 - Soil

Environment Agency

Select the method of calculating the soil water Partition Co-efficient by using the pull down menu below

User specified value for partition coefficient

Contaminant		**Pyrene**	
Target concentration	C_T	**0.000025**	**mg/l**

Input Parameters	**Variable**	**Value**	**Unit**	**Source of parameter value**
Standard entry				
Water filled soil porosity	θ_W	0.00E+00	fraction	
Air filled soil porosity	θa	0.00E+00	fraction	
Bulk density of soil zone material	ρ	0.00E+00	g/cm^3	
Henry's Law constant	H	0.00E+00	dimensionless	
Entry if specify partition coefficient (option)				
Soil water partition coefficient	Kd	0.00E+00	l/kg	
Entry for non-polar organic chemicals (option)				
Fraction of organic carbon (in soil)	foc		fraction	
Organic carbon partition coefficient	Koc		l/kg	
Entry for ionic organic chemicals (option)				
Sorption coefficient for neutral species	$K_{oc,n}$		l/kg	
Sorption coefficient for ionised species	$K_{oc,i}$		l/kg	
pH value	pH		pH units	
Acid dissociation constant	pKa			
Fraction of organic carbon (in soil)	foc		fraction	
Soil water partition coefficient used in Level Assessment	**Kd**	**0.00E+00**	**l/kg**	*Specified value*

This sheet calculates the Level 1 remedial target for soils(mg/kg) based on a selected target concentration and theoretical calculation of soil water partitioning. Three options are included for determining the partition coefficient.

The measured soil concentration as mg/kg should be compared with the Level 1 remedial target to determine the need for further action.

Level 1 Remedial Target

Level 1 Remedial Target	**#DIV/0!**	**mg/kg**	**(for comparison with soil analyses)**
	or		
	0.000025	**mg/l**	**(for comparison with leachate test results)**

Site being assessed:	CSM2
Completed by:	TW
Date:	12-Oct-12
Version:	3.1

Remedial Targets Worksheet , Release 3.1

Level 1 - Soil

Select the method of calculating the soil water Partition Co-efficient by using the pull down menu below

User specified value for partition coefficient

Contaminant		**Fluoranthene**	
Target concentration	C_T	**0.00004**	**mg/l**

Input Parameters	**Variable**	**Value**	**Unit**	**Source of parameter value**
Standard entry				
Water filled soil porosity	θ_W	0.00E+00	fraction	
Air filled soil porosity	θa	0.00E+00	fraction	
Bulk density of soil zone material	ρ	0.00E+00	g/cm^3	
Henry's Law constant	H	0.00E+00	dimensionless	
Entry if specify partition coefficient (option)				
Soil water partition coefficient	Kd	0.00E+00	l/kg	
Entry for non-polar organic chemicals (option)				
Fraction of organic carbon (in soil)	foc		fraction	
Organic carbon partition coefficient	Koc		l/kg	
Entry for ionic organic chemicals (option)				
Sorption coefficient for neutral species	$K_{oc,n}$		l/kg	
Sorption coefficient for ionised species	$K_{oc,i}$		l/kg	
pH value	pH		pH units	
Acid dissociation constant	pKa			
Fraction of organic carbon (in soil)	foc		fraction	
Soil water partition coefficient used in Level Assessment	**Kd**	**0.00E+00**	**l/kg**	*Specified value*

This sheet calculates the Level 1 remedial target for soils(mg/kg) based on a selected target concentration and theoretical calculation of soil water partitioning. Three options are included for determining the partition coefficient.

The measured soil concentration as mg/kg should be compared with the Level 1 remedial target to determine the need for further action.

Level 1 Remedial Target

Level 1 Remedial Target	**#DIV/0!**	**mg/kg**	**(for comparison with soil analyses)**
	or		
	0.00004	**mg/l**	**(for comparison with leachate test results)**

Site being assessed:	CSM2
Completed by:	TW
Date:	12-Oct-12
Version:	3.1

Appendix C

Worksheets for Level 2 Risk Assessment

Remedial Targets Worksheet , Release 3.1

Level 2 - Soil

Contaminant		**Pyrene**		from Level 1
Target concentration	C_T	**0.000025**	**mg/l**	from Level 1

This sheet calculates the Level 2 remedial target for soils (mg/kg) or for pore water (mg/l).

The measured soil concentration as mg/kg or pore water concentration should be compared with the Level 2 remedial target to determine the need for further action. Equations presented in 'Hydrogeological risk assessment for land contamiantion' (Environment Agency 2006)

Input Parameters	**Variable**	**Value**	**Unit**	**Source of parameter value**	
Standard entry					
Infiltration	Inf	1.90E-03	m/d		
Area of contaminant source	A	1.44E+03	m^2		Not used in calculation
Entry for groundwater flow below site					
Length of contaminant source in direction of groundwater flow	L	7.20E+02	m		
Saturated aquifer thickness	da	2.00E+01	m		
Hydraulic Conductivity of aquifer in which dilution occurs	K	8.60E+01	m/d		
Hydraulic gradient of water table	i	1.50E-03	fraction		
Width of contaminant source perpendicular to groundwater flow	w	4.00E+00	m		Not used in calculation
Background concentration of contaminant in groundwater beneath site	Cu	0.00E+00	mg/l		
Define mixing zone depth by specifying or calculating depth (using pull down list)		**Specify**			
Enter mixing zone thickness	Mz	1.00E+01	m		
Calculated mixing zone thickness	Mz		m		

Calculated Parameters

Dilution Factor	**DF**	**1.94E+00**		
Level 2 Remedial Target		**4.86E-05**	**mg/l**	**For comparison with measured pore water concentration. This assumes Level 1 Remedial Target is based on Target Concentra**
		or		
		#DIV/0!	**mg/kg**	**For comparison with measured soil concentration. This assumes Level 1 Remedial Target calculated from soil-water**

Additional option

Calculation of impact on receptor

Concentration of contaminant in contaminated discharge (entering receptor)	Cc	1.20E-04	mg/l	
Calculated concentration within receptor (dilution only)		6.18E-05	mg/l	0

Site being assessed:	CSM2
Completed by:	TW
Date:	12-Oct-12
Version:	3.1

Environment Agency

Remedial Targets Worksheet , Release 3.1

Level 2 - Soil

Contaminant		**Fluoranthene**		from Level 1
Target concentration	C_T	**0.00004**	**mg/l**	from Level 1

This sheet calculates the Level 2 remedial target for soils (mg/kg) or for pore water (mg/l).

The measured soil concentration as mg/kg or pore water concentration should be compared with the Level 2 remedial target to determine the need for further action. Equations presented in 'Hydrogeological risk assessment for land contamiantion' (Environment Agency 2006)

Input Parameters	**Variable**	**Value**	**Unit**	**Source of parameter value**	
Standard entry					
Infiltration	Inf	1.90E-03	m/d		
Area of contaminant source	A	1.44E+03	m^2		Not used in calculation
Entry for groundwater flow below site					
Length of contaminant source in direction of groundwater flow	L	7.20E+02	m		
Saturated aquifer thickness	da	2.00E+01	m		
Hydraulic Conductivity of aquifer in which dilution occurs	K	8.60E+01	m/d		
Hydraulic gradient of water table	i	1.50E-03	fraction		
Width of contaminant source perpendicular to groundwater flow	w	4.00E+00	m		Not used in calculation
Background concentration of contaminant in groundwater beneath site	Cu	0.00E+00	mg/l		
Define mixing zone depth by specifying or calculating depth (using pull down list)		**Specify**			
Enter mixing zone thickness	Mz	1.00E+01	m		
Calculated mixing zone thickness	Mz		m		

Calculated Parameters

Dilution Factor	**DF**	**1.94E+00**		
Level 2 Remedial Target		**7.77E-05**	**mg/l**	**For comparison with measured pore water concentration. This assumes Level 1 Remedial Target is based on Target Concentra**
		or		
		#DIV/0!	**mg/kg**	**For comparison with measured soil concentration. This assumes Level 1 Remedial Target calculated from soil-water**

Additional option

Calculation of impact on receptor

Concentration of contaminant in contaminated discharge (entering receptor)	Cc	1.80E-04	mg/l	
Calculated concentration within receptor (dilution only)		9.26E-05	mg/l	0

Site being assessed:	CSM2
Completed by:	TW
Date:	12-Oct-12
Version:	3.1

Appendix D

Worksheets for Level 3 Risk Assessment

Remedial Targets Worksheet , Release 3.1

Level 3 - Soil

See Note

Environment Agency

Input Parameters

Input Parameters	Variable	Value	Unit	Source
Contaminant		Pyrene		from Level 1
Target Concentration	C_T	0.000025	mg/l	from Level 1
Dilution Factor	DF	1.94E+00		from Level 2

Select analytical solution (click on brown cell below, then on pull-down menu)

Ogata Banks — Equations in HRA publication

Select nature of decay rate (click on brown cell below, then on pull-down menu)

Approach for simulating degradation of pollutants: **Apply degradation rate to dissolved pollutants only**

	Variable	Value	Unit	Source of parameter value
Enter source concentration		**Soil leachate concentration as mg/l**		
Enter soil leachate concentration		**0.00012**	mg/l	
Half life for degradation of contaminant in water	$t_{1/2}$	9.00E+99	days	
Calculated decay rate	λ	7.70E-101	$days^{-1}$	calculated
Width of plume in aquifer at source	Sz	4.00E+00	m	from Level 2
Plume thickness in aquifer at source	Sy	1.00E+01	m	from Level 2
Bulk density of aquifer materials	ρ	2.20E+00	g/cm^3	
Effective porosity of aquifer	n	2.00E-01	fraction	
Hydraulic gradient	i	3.09E-03	fraction	from Level 2 (adjusted)
Hydraulic conductivity of saturated aquifer	K	8.60E+01	m/d	from Level 2
Distance to compliance point	x	3.00E+01	m	
Distance (lateral) to compliance point perpendicular to flow direction	z		m	
Distance (depth) to compliance point perpendicular to flow direction	y		m	
Time since pollutant entered groundwater	t	1.00E+99	days	*time variant options only*
Parameters values determined from options				
Partition coefficient	Kd	6.80E+01	l/kg	*see options*
Longitudinal dispersivity	ax	3.000	m	*see options*
Transverse dispersivity	az	0.300	m	*see options*
Vertical dispersivity	ay	0.030	m	*see options*

Parameter values should be checked against Level 1 and 2

Calculated Parameters

Calculated Parameters	Variable		
Groundwater flow velocity	v	1.33E+00	m/d
Retardation factor	Rf	7.49E+02	fraction
Decay rate used	λ	1.03E-103	d^{-1}
Hydraulic gradient used in aquifer flow down-gradient	i	3.09E-03	fraction
Rate of contaminant flow due to retardation	u	1.77E-03	m/d
Ratio of Compliance Point to Source Concentration	C_{ED}/C_0	3.63E-01	fraction
Attenuation factor (C_O/C_{ED})	**AF**	**2.76E+00**	**fraction**
Soil leachate concentration	Co	1.20E-04	

Remedial Targets

Level 3 Remedial Target	**1.34E-04**	**mg/l**	For comparison with measured pore water concentration.
Ogata Banks	**or**		This assumes Level 1 Remedial Target is based on Target Concentration.
	#DIV/0!	**mg/kg**	For comparison with measured soil concentration. This
Distance to compliance point	**30**	**m**	assumes Level 1 Remedial Target calculated from soil-water partitioning equation.
Ratio of Compliance Point to Source Concentration C_{ED}/C_0	**3.63E-01**	**fraction**	Ogata Banks

Care should be used when calculating remedial targets using the time variant options as this may result in an overestimate of the remedial target.
The recommended value for time when calculating the remedial target is 9.9E+99

Enter method of defining partition co-efficient (using pull down list)

Calculate for non-polar organic chemicals

Entry if specify partition coefficient (option)

Soil water partition coefficient	Kd		l/kg

Entry for non-polar organic chemicals (option)

Fraction of organic carbon in aquifer	foc	1.00E-03	fraction
Organic carbon partition coefficient	Koc	6.80E+04	l/kg

Entry for ionic organic chemicals (option)

Sorption coefficient for related species	$K_{oc,n}$		l/kg
Sorption coefficient for ionised species	$K_{oc,i}$		l/kg
pH value	pH		
Acid dissociation constant	pKa		
Fraction of organic carbon in aquifer	foc		fraction

Soil water partition coefficient	Kd	6.80E+01	l/kg

Define dispersivity (click brown cell and use pull down list)

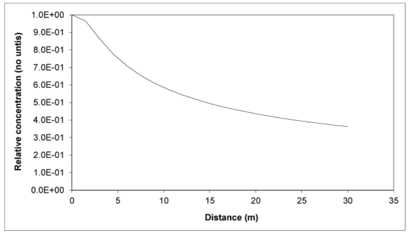
Dispersivities 10%, 1%, 0.1% of pathway length

		Enter value	Calc value	Xu & Eckstein	
Longitudinal dispersivity	**ax**	0.00E+00	3.00E+00	2.13E+00	m
Transverse dispersivity	**az**	0.00E+00	3.00E-01	2.13E-01	m
Vertical dispersivity	**ay**	0.00E+00	3.00E-02	2.13E-02	m

Note values of dispersivity must be > 0

Xu & Eckstein (1995) report ax = $0.83(\log_{10}x)^{2.414}$; az = ax/10, ay = ax/100 are assumed

Note
This worksheet should be used if pollutant transport and degradation is best described by a first order reaction. If degradation is best desribed by an electron limited degradation such as oxidation by O_2, NO_3, SO_4 etc than an alternative solution should be used



Note: 'Relative concentration' is the ratio of calculated concentation at a given position compared to the source concentration. The calculations assume plume disperses from the top of the aquifer. An alternative solution assuming the centre of the plume is located at the mid-depth of the aquifer is presented in the calculation sheets.

This sheet calculates the Level 3 remedial target for soils(mg/kg) or for pore water (mg/l), based on the distance to the receptor or compliance located down hydraulic gradient of the source Three solution methods are included, the preferred option is Ogata Banks.By setting a long travel time (e.g. 9E99) it will give the steady state solution, which should always be used when calculating remedial targets.

The measured soil concentration as mg/kg or pore water concentration should be compared with the Level 3 remedial target to determine the need for further action.

Note if contaminant is not subject to first order degradation, then set half life as 9.9E+99.

Site being assessed:	CSM2
Completed by:	TW
Date:	########
Version:	3.1

Calculated (relative) concentrations for distance-concentration graph

Ogata Banks
From calculation sheet

Distance	**Relative concentration (No units)**	**Concentration mg/l**
0	1.0E+00	6.18E-05
1.5	9.65E-01	5.96E-05
3.0	8.64E-01	5.34E-05
4.5	7.76E-01	4.80E-05
6.0	7.08E-01	4.37E-05
7.5	6.54E-01	4.04E-05
9.0	6.11E-01	3.77E-05
10.5	5.74E-01	3.55E-05
12.0	5.44E-01	3.36E-05
13.5	5.18E-01	3.20E-05
15.0	4.95E-01	3.06E-05
16.5	4.75E-01	2.93E-05
18.0	4.57E-01	2.82E-05
19.5	4.41E-01	2.73E-05
21.0	4.27E-01	2.64E-05
22.5	4.14E-01	2.56E-05
24.0	4.02E-01	2.48E-05
25.5	3.91E-01	2.41E-05
27.0	3.81E-01	2.35E-05
28.5	3.71E-01	2.29E-05
30.0	3.63E-01	2.24E-05

Remedial Targets Worksheet , Release 3.1

Level 3 - Soil

See Note

Environment Agency

Input Parameters	Variable	Value	Unit	Source
Contaminant		Fluoranthene		from Level 1
Target Concentration	C_T	0.00004	mg/l	from Level 1
Dilution Factor	DF	1.94E+00		from Level 2

Select analytical solution (click on brown cell below, then on pull-down menu)

Ogata Banks — Equations in HRA publication

Select nature of decay rate (click on brown cell below, then on pull-down menu)

Approach for simulating degradation of pollutants: Apply degradation rate to dissolved pollutants only

	Variable	Value	Unit	Source of parameter value
Enter source concentration		Soil leachate concentration as mg/l		
Enter soil leachate concentration		0.00018	mg/l	
Half life for degradation of contaminant in water	$t_{1/2}$	9.00E+99	days	
Calculated decay rate	λ	7.70E-101	$days^{-1}$	calculated
Width of plume in aquifer at source	Sz	4.00E+00	m	from Level 2
Plume thickness in aquifer at source	Sy	1.00E+01	m	from Level 2
Bulk density of aquifer materials	ρ	2.20E+00	g/cm^3	
Effective porosity of aquifer	n	2.00E-01	fraction	
Hydraulic gradient	i	3.09E-03	fraction	from Level 2 (adjusted)
Hydraulic conductivity of saturated aquifer	K	8.60E+01	m/d	from Level 2
Distance to compliance point	x	3.00E+01	m	
Distance (lateral) to compliance point perpendicular to flow direction	z		m	
Distance (depth) to compliance point perpendicular to flow direction	y		m	
Time since pollutant entered groundwater	t	1.00E+99	days	*time variant options only*
Parameters values determined from options				
Partition coefficient	Kd	4.90E+01	l/kg	*see options*
Longitudinal dispersivity	ax	3.000	m	*see options*
Transverse dispersivity	az	0.300	m	*see options*
Vertical dispersivity	ay	0.030	m	*see options*

Parameter values should be checked against Level 1 and 2

Calculated Parameters	Variable		
Groundwater flow velocity	v	1.33E+00	m/d
Retardation factor	Rf	5.40E+02	fraction
Decay rate used	λ	1.43E-103	d^{-1}
Hydraulic gradient used in aquifer flow down-gradient	i	3.09E-03	fraction
Rate of contaminant flow due to retardation	u	2.46E-03	m/d
Ratio of Compliance Point to Source Concentration	C_{ED}/C_0	3.63E-01	fraction
Attenuation factor (C_O/C_{ED})	**AF**	**2.76E+00**	**fraction**
Soil leachate concentration	Co	1.80E-04	

Remedial Targets

Level 3 Remedial Target	**2.14E-04**	**mg/l**	For comparison with measured pore water concentration.
Ogata Banks	**or**		This assumes Level 1 Remedial Target is based on Target Concentration.
	#DIV/0!	**mg/kg**	For comparison with measured soil concentration. This
Distance to compliance point	**30**	**m**	assumes Level 1 Remedial Target calculated from soil-water partitioning equation.
Ratio of Compliance Point to Source Concentration C_{ED}/C_0	**3.63E-01**	**fraction**	Ogata Banks

Care should be used when calculating remedial targets using the time variant options as this may result in an overestimate of the remedial target.
The recommended value for time when calculating the remedial target is 9.9E+99

Enter method of defining partition co-efficient (using pull down list)

Calculate for non-polar organic chemicals

Entry if specify partition coefficient (option)

Soil water partition coefficient	Kd		l/kg

Entry for non-polar organic chemicals (option)

Fraction of organic carbon in aquifer	foc	1.00E-03	fraction
Organic carbon partition coefficient	Koc	4.90E+04	l/kg

Entry for ionic organic chemicals (option)

Sorption coefficient for related species	$K_{oc,n}$		l/kg
Sorption coefficient for ionised species	$K_{oc,i}$		l/kg
pH value	pH		
Acid dissociation constant	pKa		
Fraction of organic carbon in aquifer	foc		fraction
Soil water partition coefficient	Kd	4.90E+01	l/kg

Define dispersivity (click brown cell and use pull down list)

Dispersivities 10%, 1%, 0.1% of pathway length

		Enter value	Calc value	Xu & Eckstein	
Longitudinal dispersivity	**ax**	0.00E+00	3.00E+00	2.13E+00	m
Transverse dispersivity	**az**	0.00E+00	3.00E-01	2.13E-01	m
Vertical dispersivity	**ay**	0.00E+00	3.00E-02	2.13E-02	m

Note values of dispersivity must be > 0

Xu & Eckstein (1995) report ax = $0.83(\log_{10}x)^{2.414}$; az = ax/10, ay = ax/100 are assumed

Note
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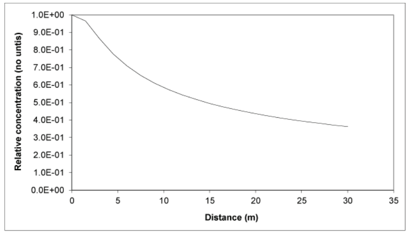

Note: 'Relative concentration' is the ratio of calculated concentation at a given position compared to the source concentration. The calculations assume plume disperses from the top of the aquifer. An alternative solution assuming the centre of the plume is located at the mid-depth of the aquifer is presented in the calculation sheets.

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Note if contaminant is not subject to first order degradation, then set half life as 9.9E+99.

Site being assessed:	CSM2
Completed by:	TW
Date:	########
Version:	3.1

Calculated (relative) concentrations for distance-concentration graph

Ogata Banks
From calculation sheet

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6.0	7.08E-01	6.56E-05
7.5	6.54E-01	6.06E-05
9.0	6.11E-01	5.66E-05
10.5	5.74E-01	5.32E-05
12.0	5.44E-01	5.04E-05
13.5	5.18E-01	4.80E-05
15.0	4.95E-01	4.59E-05
16.5	4.75E-01	4.40E-05
18.0	4.57E-01	4.24E-05
19.5	4.41E-01	4.09E-05
21.0	4.27E-01	3.95E-05
22.5	4.14E-01	3.83E-05
24.0	4.02E-01	3.72E-05
25.5	3.91E-01	3.62E-05
27.0	3.81E-01	3.53E-05
28.5	3.71E-01	3.44E-05
30.0	3.63E-01	3.36E-05

Appendix E

Wells Within 30m of the Alignment

CSM2 and MSRFL: Groundwater Abstraction Well Located Within 30m of the Alignment

Well No.	Well Status	Well Depth (m)	Well Use	Top of Well Screen (m)	Base of Well Screen (m)	NZTM_X	NZTM_Y	Well Owner	Well Address	Distance from Alignment (m)
M36/0124	Not Used	21.0	Unknown	Unknown	Unknown	1551507	5174189	Elizabeth Ann Veix, John Maurin Veix	1528 Main South Rd,Rolleston	22.0
M36/20347	Unknown	40.0	Domestic and Stockwater	Unknown	Unknown	1555205	5176188	Curraghs Holdings Limited	10 Curraghs Rd, West Melton	27.0
M36/3737	Active	36.4	Domestic and Stockwater	Unknown	Unknown	1552422	5174030	Fay Patricia Fletcher, William Frederick Fletcher	755 Weedons Rd, Weedons-Lincoln College	4.5
M36/4229	Active	62.5	Domestic Supply	61.5	62.5	1559983	5176639	Deirdre Elena Groube, Kevin Charles Groube	191 Marshs Rd, Prebbleton	28.0
M36/7545	Active	33.0	Domestic and Stockwater	31.5	33.0	1558314	5176529	Belinda Ann Morrison, Clio Trustee Services Limited, Vaughan Graeme Morrison	304 Blakes Rd, Prebbleton	3.5
M36/0288	Not Used	21.0	Unknown	Unknown	Unknown	1552496	5173439	Denis John Bussell, Helen Isabel Bussell, John Francis Butchard	693 Weedons Rd, Weedons-Lincoln College	19.4
M36/0326	Not Used	19.8	Unknown	14.3	15.8	1560143	5176719	Calder Stewart Industries Limited	201 Marshs Rd, Prebbleton	14.8
M36/0911	Active	26.0	Commercial	Unknown	Unknown	1560503	5176239	Tegel Foods Limited	262 Marshs Rd, Prebbleton	8.4
M36/2231	Active	39.6	Irrigation	30.9	37.0	1554770	5175775	Christopher Selwyn Warren	1181 Main South Rd, Rolleston	28.0
M36/2695	Active	21.9	Domestic Supply	Unknown	Unknown	1556751	5175952	Emma Joy Steel, Michael Joseph Sweeney	Unknown	21.0
M36/3875	Active	38.0	Domestic Supply	36.0	38.0	1555250	5176046	Jennifer Joy Flett, Murray John Mannall, Susanne Madeline Mannall	1/1133 Main South Rd,Rolleston	9.6
M36/4306	Active	48.0	Domestic Supply	Unknown	Unknown	1555115	5175899	Godfried Maria Louise Van Tulder, Philip Haunui Royal, Sandra Kay Van Tulder	Unknown	22.0
M36/4675	Not Used	17.4	Domestic Supply	Unknown	Unknown	1552456	5174399	Elizabeth Jean Doyle, Gary Edward Doyle	768 Weedons Rd,Weedons-Lincoln College	25.0
M36/4709	Not Used	42.3	Unknown	40.0	42.0	1552018	5173229	Fay Lynette Taylor, Grant Murray Taylor, LS Trustees (No.7) Limited	58 Levi Rd, Rolleston	20.8
M36/7374	Active	15.0	Groundwater Quality	11.0	15.0	1560561	5176211	Tegel Foods Limited	262 Marshs Rd, Prebbleton	9.3
M36/7502	Active	17.3	Domestic and Stockwater	15.8	17.3	1562292	5175989	Aiko Harcourt, Martin Richard Harcourt, Peter Ian Cullen	19 John Paterson Drive, Halswell	14.6
M36/7996	Active	48.0	Domestic and Stockwater	46.5	48.0	1556199	5176782	Unknown	Unknown	25.0

Appendix H | Applicable WQL6 Rules and Commentary

Rule	Commentary
Condition 1 (a)	In general, the stormwater system is designed to a 100 year ARI rainfall event which exceeds the standard specified in Condition 1. (a). However, some flooding on the upstream property owner will be required to convey overland flows to the soak pit locations for disposal (where siphons are impractical). Ponding surrounding the soak pits may also induce flooding, due to the flat gradients of the existing ground. This will be limited to increases in flood level of 250 mm for the 100 year ARI event and no additional floor level flooding in events up to the 50 Year ARI 24 hour event. The area around the inlets to the overland flow siphons will be lowered to construct a settlement area (to reduce the volume of silt entering the system) and to limit the elevation of the inlet
Condition 1 (b)	This condition has been interpreted to mean ponding of stormwater for no greater than 48 hours after the cessation of rainfall. The disposal system has been designed to dispose of the collected stormwater within 48 hours. Given the potential for high groundwater levels the design disposal rate from these ponds has been significantly reduced, increasing the disposal infrastructure, however, the duration of storage may exceed 48 hours (depending on the rainfall event).
Condition 1 (c)	The nature of the topography and the geometrics of the Project limit the possibilities for erosion to occur. Grass cover on the overpass embankments will avoid erosion.
Condition 1 (d)	Compliance with the 1 m rule will be possible for the vast majority of the Project. Some collection and treatment detention basins, disposal fields and siphons will be within one metre of groundwater, for example the ponds in Halswell Junction Road area, the east facing off-ramps at the CSM1 connection, overland flow siphons about Springs Road and the Robinsons Road overpass.
Condition 1 (e)	There is one property on the CSM2 corridor which is registered on the ECan Listed Land Use Register (LLUR) (former landfill on the NW corner of Robinsons Rd). This site is referred to as the old Currahs Road Landfill. There is a second property listed on the LLUR located to the west of the roundabout with Springs Road and

	Halswell Junction Road. At Robinsons Road a bund is proposed to prevent runoff from the Robinsons Road site entering the Project drainage system. There may be discharge onto and from these properties At Springs Road no work is proposed as part of the Project. No Project runoff will enter the old quarry pit as the Pit is upstream of the Project.
Condition 3 (a)	The corridor is well clear of any community drinking water supply zone. In general, the discharge system is designed to meet the treatment objectives listed in Condition 3. (c). Generally, the flows will be through an organic filter media, excepting the swales (where topsoil will be used) and the bases of the overland flow siphons, which will only rarely activate. These aspects of the design will not meet this treatment standard set out in the NRRP, however, the effects of these discharges will be less than minor. It is proposed to use specific soil characteristics to characterise the organic filter media in the treatment devices in preference to using the percolation rates specified in the rule. The application of the soil characteristics were agreed by ECan in a recent consent variation for the CSM1 Project. This will allow the percolation rates to more closely match the percolation rate of the receiving environment.
Condition 3 (d)	The size of the disturbed area will be much greater than two hectares during the construction of the Project. This condition will not be met.

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Appendix I | Applicable NRRP Rules and Commentary

Land use consents (s9)	Commentary
Rule WQL36 Excavation of land in the Coastal Confined Gravel Aquifer System, or over an unconfined or semi-confined aquifer	
The use of land to excavate more than 100 cubic metres of material in any 12 month period from land: (a) Over an unconfined or semi-confined aquifer and the depth of excavation: i. exceeds five metres or ii. is deeper than the highest groundwater level which can reasonably be expected to occur at the site, based upon the relevant and available groundwater data or (b) In the Coastal Confined Gravel Aquifer System where there is less than one metre of undisturbed material between the base of the excavation and Aquifer 1 Is: 1. **A restricted discretionary activity** if such use complies with all of the conditions of this Rule 2. **A discretionary activity** if such use is within Christchurch Groundwater Protection Zone 1B or complies with conditions 1(a), 1(b) or 1(c) 3. **A non-complying activity** if such use does not comply with any one or more of Conditions 1(a), 1(b), 1(c) or 1(d). **Conditions:** 1. The use of land shall not occur within: (a) 50 metres of the bed of any permanently or intermittently flowing river, or a lake or (b) 50 metres of a wetland boundary or (c) A Community Drinking Water Supply Protection Zone for a well listed in Schedule WQL2 or (d) Christchurch Groundwater Protection Zone 1, 1A, 1C, 1D or Zone 2, as shown on the Map Volume Part 1 – Planning Maps.	The extent of the unconfined aquifer includes Robinsons Road Overpass and the Maize Maze Pond which may include excavations for foundations deeper than 5 m. The Robinsons Road Overpass and Maize Maze are located over an unconfined or semi-confined aquifer, however it will not occur within the land identified in conditions 1a) – d).
Rule WQL37 Deposition of more than fifty cubic metres of material into excavated land over an unconfined or semi-confined aquifer	
Except where it is authorised as a **permitted activity** under Rule WQL22 the use of land for the deposition of more than 50 cubic metres of material in any consecutive 12 month period where	Foundation construction will be required at the base of the excavations for the Robinsons

the land into which the material is deposited: (a) is excavated to a depth in excess of five metres below the natural land surface; and (b) is located over an unconfined or semi confined aquifer, where the highest level of groundwater which can reasonably be expected to occur at the site based upon the relevant and available groundwater data, is less than 30 metres below the natural land surface; is – 1. **A controlled activity** if such use complies with all of the conditions of this Rule 2. **A discretionary activity** if such use does not comply with any one or more of conditions of this Rule. **Conditions:** 1. The material shall only consist of clean fill. 2. The volume of vegetative matter in any cubic metre of material deposited shall not exceed three per cent. 3. The material shall not be deposited into groundwater. 4. Any cured asphalt deposited shall be placed in the land at least one metre above the highest groundwater level expected at the site. **5.** A management plan shall be prepared in accordance with Section 8.1 and **Appendix B** of "*A Guide to the Management of Cleanfills*", Ministry for the Environment, January 2002.	Road Overpass and the Maize Maze Pond, both of which are located over the unconfined aquifer. The percentage of organic material required in the filter layers (5% – 10%) in the base of the treatment devices will exceed the 3% threshold specified in condition 2.
Rule WQL31 Construction of a groundwater bore or a water infiltration gallery	
The use of land to construct a bore or to excavate land for a water infiltration gallery, for the purpose of taking, investigating or monitoring groundwater is – 1. **A restricted discretionary activity** if such use complies with all of the conditions of this Rule 2. **A non-complying activity** if such use does not comply with any one or more of conditions of this Rule. **Conditions:** 1. The activity shall comply with Schedule WQL4 *Standards and Terms for the construction of bores and water infiltration*	The groundwater collection field associated with the intermittent pumping (diversion) of water from the Robinsons Road overpass requires consent under this rule. Construction of replacement wells for severed land parcels and affected wells will require consent under this rule.

<table>
<tr><td>galleries.
2. The information recorded as a requirement of Section 3 “Record Keeping” of Schedule WQL4 Standards and Terms for the construction of bores and water infiltration galleries, shall be forwarded to Environment Canterbury within one month of completion of the work.</td><td></td></tr>
<tr><td>Discharge permits (s15)</td><td></td></tr>
<tr><td>Rule WQL6 Discharge of stormwater onto or into land</td><td rowspan="2">This rule is discussed in detail in Appendix F above.</td></tr>
<tr><td>The discharge of stormwater onto or into land where contaminants may enter groundwater

is -

1. A permitted activity if the discharge:
(a) was lawfully established at 4 July 2004 or
(b) is solely from a roof and complies with Conditions 1 and 2 or
(c) is from any other source, including a road, and complies with Conditions 1 and 3
2. A discretionary activity if the discharge is:
(a) solely from a roof and does not comply with Conditions 1 or 2 or
(b) from any other source, including a road, and does not comply with any one or more of Conditions 1, 3(b), 3(c) or 3(d)
3. Unless another person, who has applied for, or been granted, a discharge permit under Rule WQL8 provides written authority for the activity to be carried out under their permit.
4. A non-complying activity if the discharge does not comply with Condition 3(a) unless another person, who has applied for, or been granted, a discharge permit under Rule WQL8, provides written authority for the activity to be carried out under their permit.

Conditions:

Discharge from any source

1.
(a) The discharge shall not cause stormwater from up to and including a 24 hour duration 2% exceedance probability rainfall event to enter any other property beyond the boundary of the property or area in which the discharge occurs, unless written authorisation from the affected landowner is obtained</td></tr>
</table>

(b) The discharge shall not result in the ponding of stormwater on the ground for more than 48 hours (c) The discharge shall not cause erosion of soil (d) The discharge system shall be located at least one metre above the highest groundwater level that can be reasonably inferred for the site at or about the time the system is constructed and (e) The discharge shall not be onto or from a property that has been registered by the Environment Canterbury on its Listed Land Use Register as a site that is 'not investigated', 'below guideline values for', 'managed for', 'partially investigated', 'significant adverse environmental effects' or 'contaminated for'. **Discharge solely from a roof** 2. (a) The discharge system shall be sealed to prevent any other contaminants entering the system. **Discharge from any source other than a roof** 3. (a) The discharge shall not be within a Community Drinking Water Supply Protection Zone for a well listed in Schedule WQL2 if: (1) the discharge was not lawfully established before the date this rule became operative and (2) the discharge is from that part of a road, including a State highway that has four lanes for motor vehicles. (b) The discharge shall not be from a property where: (1) an activity or industry specified in Schedule WQL9 is occurring or (2) the quantity of hazardous substances stored or handled exceeds the thresholds in Schedule WQL9 and the hazardous substances may become entrained in stormwater. (c) A discharge that is: (1) solely from a sealed road or (2) from a combination of sources and is located in an area where the depth to unconfined or semi-confined groundwater is less than six metres as indicated in Map Volume – Part 2 Indicative Maps, shall either be via a	

<table>
<tr><td>fully vegetated soil treatment system with the following characteristics:
1. a minimum depth of 200 millimetres of soil, and
2. an infiltration rate between 20 and 50 millimetres per hour, and
3. at least 5 per cent clay content in the soil, and
4. be designed to capture and infiltrate all contributing stormwater for rainfall events up to and including a 24 hour duration ten per cent annual exceedance probability
or via an alternative stormwater treatment system that is certified in writing by a suitably qualified and competent person as providing at least equivalent stormwater treatment. A copy of that certification, design plans for the system and appropriate technical documentation that demonstrates the technical basis for the certification shall be provided to the Environment Canterbury at least 20 working days prior to installation.
(d) Unless the discharge from a combination of sources was legally established before the date of this rule became operative, or the discharge into a stormwater collection system for an unauthorized stormwater discharge, the discharge shall not be from an area of disturbed land greater than:
(1) 1000 square meters within Zone BP in Map Volume – Part 1 Planning Maps or
(2) Two hectares in any other location.</td><td></td></tr>
<tr><td>Rule WQL2 Discharge of land drainage, site dewatering, aquifer test or bore development water into a river, lake or artificial watercourse, or onto land which may result in water or a contaminant entering a river, lake or artificial watercourse</td><td rowspan="2">Some site dewatering may be required, depending on seasonal groundwater levels, in order to construct the foundations for the road and stormwater pond</td></tr>
<tr><td>The discharge of land drainage water, site dewatering water, aquifer test or bore development water:
(a) into a river, lake or artificial watercourse or
(b) onto land which may result in a contaminant or water entering a river, lake or artificial watercourse that is not classified by Rules WQL1, WQL4, WQL7 or WQL8
is –</td></tr>
</table>

1. A **permitted activity** if the discharge is: (a) land drainage water and the discharge complies with all of Conditions 1 to 9 of this Rule or (b) aquifer test, bore development or site dewatering water and the discharge complies with all of Conditions 1 to 8 of this Rule or	land drainage system. The discharges are likely to be to Montgomery's Drain, an artificial watercourse.
2. Where the discharge does not comply with any one or more of Conditions 1 to 8 and, in addition, where the discharge of land drainage water does not comply with Condition 9 of this Rule the activity is classified by Rule WQL48.	Given that Montgomery's Drain and Upper Knights Stream are notionally dry condition 2 cannot be met.
Conditions: **General conditions for all discharges** 1. The specific conductance (conductivity measured at 25 degrees Celsius) of the discharge shall not exceed 40 millisiemens per metre. 2. The rate of flow in the river or artificial watercourse at the point and time of discharge to surface water shall be at least five times the rate of the discharge. 3. The rate of discharge to a lake shall not exceed five litres per second. 4. The concentration of: (a) total suspended solids in a discharge to water shall not exceed 25 grams per cubic metre or (b) un-ionised hydrogen sulphide in a discharge to water shall not exceed 0.005 grams per cubic metre.	Given the relative scales of the Halswell River Catchment to the Montgomery's Drain catchment the likelihood of flooding being instigated is small. Approvals from the CCC and ECan officers will be sought prior to discharge as part of the erosion and sediment control plan. The erosion and sediment control plan is discussed in greater detail in the CEMP.
5. The discharge shall not result in: (a) flooding of a dwelling or land owned or occupied by another person, other than with the express permission of that person or (b) erosion of the bed or banks of the receiving water body.	Given that the receiving waterways are notionally dry condition 6 is not applicable. Notwithstanding this, given that the discharges will be of groundwater it is unlikely that the condition would be breached.
6. The discharge shall not, outside of the Mixing Zone: (a) change the colour of the receiving water by more than five Munsell units (b) change the clarity of the receiving water by more than 20 per cent (c) change the pH of the receiving water by more than 0.5 pH unit (d) change the temperature of the receiving water of a river or artificial watercourse by more than two degrees Celsius (e) change the temperature of the receiving water of a lake by more than two degree Celsius	Discharges to land are considered to be permitted under this rule.

(f) produce conspicuous oil or grease films, scums, foams, floatable or suspended materials (g) produce any objectionable odour (h) render freshwater unsuitable for consumption by farm animals or (i) cause the concentration of *Escherichia coli* to exceed 550 E. coli per 100 millilitres. 7. The discharge shall not reduce the quality of the receiving water within: (a) 500 metres upstream on a river or artificial watercourse or (b) 500 metres on a lake from an intake for a community drinking water supply listed in Schedule WQL2. 8. The discharge shall not contain any hazardous substance, hazardous waste or added radioactive isotope. **Additional condition for land drainage water only** 9. A discharge of land drainage water shall: (a) only be from a drainage system which existed at 3 July 2004 and (b) not be from a wetland unless the drainage of the wetland is authorised as a permitted activity by the rules of Chapter 7 of the NRRP or by a resource consent and (c) flow by gravity only.	
Rule WQL7 Discharge of stormwater into a river, lake or artificial watercourse	
The discharge of stormwater into: (a) a river, lake or artificial watercourse or (b) onto land where it may enter a river, lake or artificial watercourse is – 1. A **permitted activity** if the discharge (a) was lawfully established at 4 July 2004 or (b) complies with all of the conditions of this Rule. 2. Where the discharge does not comply with any one or more of Conditions 1 to 10 of this Rule the activity is classified by Rule WQL48 unless another person, who has applied for, or been granted, a discharge permit under Rule WQL8 provides written authority for the activity to be carried out under their permit.	There are two scenarios for discharges to surface water from the highway drainage system: overflows from the Maize Maze Pond and the Ramp Pond during events greater than a 100 year ARI (or combinations of extreme groundwater and lesser rainfall events), and drawing down of the pond during extreme groundwater events. Both scenarios will discharge into Montgomery's Drain and/or the stormwater network connecting Montgomery's Drain

3. a **non-complying activity** if the discharge does not comply with Condition 11 of this Rule unless another person, who has applied for, or been granted, a discharge permit under Rule WQL8 provides written authority for the activity to be carried out under their permit. **Conditions:** 1. There is no stormwater collection system available for the collection of the stormwater. For the purpose of this condition, "available" means: (a) a stormwater collection system passes within 50 m of the discharge location and (b) the stormwater can flow into the collection system under gravity and (c) the stormwater collection system operator will accept the discharge. 2. The discharge shall not be from a property where: (a) an activity or industry specified in Schedule WQL9 is occurring or (b) the quantity of hazardous substances stored or handled exceeds the thresholds in Schedule WQL9 and the hazardous substances may become entrained in stormwater. 3. The discharge shall not be onto or from a property that has been registered by the Environment Canterbury on its Listed Land Use Register as a site that is 'not investigated', 'below guideline values for', 'managed for', 'partially investigated', 'significant adverse environmental effects' or 'contaminated for'. 4. The discharge shall not be into: (a) a water race, as defined in Section 5 of the Local Government Act 2002 or (b) a wetland, unless the wetland is part of a lawfully established stormwater or wastewater treatment system. 5. The discharge shall not result in an increase in the flow in the receiving water body at the point of discharge of more than one per cent of a flood event with an Annual Exceedance Probability of 20 per cent (five year ARI event). 6. Unless the discharge was lawfully established before the date this rule became operative, the discharge shall not be	to Upper Knights Stream. The ponds have been sized for a 100 year total storm detention, therefore condition 5 will be in the overflow scenario. The draw down scenario will occur after the recession of the peak in the prior rainfall event. Given that the discharges will be significantly diluted (by post-first flush runoff in the overflow scenario and potentially groundwater in the drawdown scenario) and from the downstream end of a treatment system the water quality aspects are expected to be met without difficulty. The only condition which may be breached in this rule is 6 (b), whereby discharges may occur if an extreme rainfall event occurs during construction. The relevant rule in this situation is Rule WQL48, addressed below.

from an area of disturbed land of greater than: (a) 1000 square metres located in Zone BP in Map Volume - Part 1 Planning Maps or (b) one hectare in any other location. 7. Where the discharge is from a roof with no other stormwater, it shall be via a system that prevents any other contaminants from entering the stormwater system. 8. The concentration of total suspended solids in the discharge shall not exceed: (a) 50 grams per cubic metre, where the discharge is to any Spring-fed river, Banks Peninsula river, or to a lake or (b) 100 grams per cubic metre where the discharge is to any other river or to an artificial watercourse. 9. The discharge of stormwater from an electricity substation area, where oil filled equipment is located, shall only be made to surface water, where: (a) a connection to a sewerage network is not available, and (b) the electricity substation area is enclosed within an impervious bunded area, or designed to contain all spillages, or is encircled by interceptor drains, and drains to an oil interceptor of a type and size which gives a concentration of oil and grease not exceeding 15 grams per cubic metre in the discharge as measured by American Society for Testing and Materials (ASTM) Method D4281, or American Public Health Association (APHA) 5520B, and can retain the capacity of the largest container of oil on the site plus 10 per cent of that volume and (c) a copy of all maintenance records for the stormwater and oil containment systems shall be made available to Environment Canterbury upon request. 10. The discharge shall not be within 500 m upstream on a river, or an artificial watercourse, or within 500 m on a lake, from an intake for a community drinking water supply listed in Schedule WQL2. 11. Unless the discharge was lawfully established before the date this rule became operative, the discharge shall not be	

to any water body that is Class NATURAL. For the purposes of this rule: (a) **'Stormwater collection system'** means any system specifically made or formed to collect or direct stormwater and includes, but is not limited to kerb and channel, swales, pipes, drains, ponds and sumps. (b) In Condition 3 '**not investigated**' means the present or past site land-use history has been confirmed as one that appears on the Ministry for the Environment Hazardous Activities and Industries List (HAIL). '**below guideline values for**' means the site has been investigated. The results demonstrate that hazardous substances are present, but indicate that, under the current land use, the adverse effects on the environment or risks to people are acceptable. '**managed for**' means the site has been investigated. The results demonstrate that there are hazardous substances present at the site, but indicate that any adverse effects or risks to people and the environment are managed. '**partially investigated**' means the site has been partially investigated. The results demonstrate that there are hazardous substances present at the site, but there is insufficient information to quantify adverse effects or risks to people and the environment. '**significant adverse environmental effects**' means the site has been investigated. The results demonstrate that sediment, groundwater or surface water has hazardous substances in or on it that: (i) have significant adverse effects on the environment or (ii) are reasonably likely to have significant adverse effects on the environment. '**contaminated for**' means the site has been investigated. The results demonstrate it is land of one of the following kinds: (i) if there is an applicable national environmental standard on contaminants in soil, the land is more contaminated than the standard allows or (ii) if there is no applicable national environmental standard on contaminants in soil, the land has a hazardous substance in or	

<table>
<tr>
<td>on it that –

(1) has significant adverse effects on the environment or

(2) is reasonably likely to have significant adverse effects on the environment.</td>
<td></td>
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<tr>
<td>Rule WQL48 Discharge of water or a contaminant into a river, lake or an artificial watercourse</td>
<td rowspan="2">Consent is being sought to allow discharges to an artificial watercourse (Montgomery's Drain) under this rule, in the situation that an extreme rainfall event occurs during construction and in the instance that site de-watering discharges into this drain.

Where the discharge is a result of the overspill into the artificial watercourse, it is considered that the discharge will not exceed the specified concentrations as the discharge will be significantly diluted after the stormwater has been treated, or the discharge may only be groundwater.

Where the discharge is of water from site de-watering these conditions shall be met as the discharge will be groundwater.</td>
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<tr>
<td>The discharge of water, or a contaminant, into a river, lake or artificial watercourse that is not classified by Rules WQL5, WQL7, WQL8, WQL15, WQL16, WQL17, WQL18, WQL19, WQL21 or WQL41

is –

1. A discretionary activity if the discharge complies with all of the conditions of this Rule
2. A non-complying activity if the discharge does not comply with any one or more of the conditions of this Rule.

Conditions:

1. The concentration of the total suspended solids in the discharge shall not exceed the concentrations in the following table:
<table>
<tr><td></td><td>Stormwater discharge</td><td colspan="2">Other Discharge</td></tr>
<tr><td>Water Quality Management Unit</td><td></td><td>Minimum ratio of receiving water flow to discharge flow at any time is greater than 3:1</td><td>Minimum ratio of receiving water flow to discharge flow at any time is less than or equal to 3:1</td></tr>
<tr><td></td><td colspan="3">Total suspended solids maximum (grams per cubic metre)</td></tr>
<tr><td>Banks Peninsula or Spring-fed rivers</td><td>100</td><td>100</td><td>50</td></tr>
<tr><td>All other rivers</td><td>250</td><td>250</td><td>100</td></tr>
</table>
2. The discharge shall not, outside of the Mixing Zone</td>
</tr>
</table>

calculated in accordance with Part 2 of Schedule WQL1, meet the relevant water quality: (a) standards in Schedule WQL1 for that water quality class specified on the Map Volume Part 1 – Planning Maps and (b) provisions and standards in any applicable water conservation order. 3. The relevant water quality standards in Schedule WQL1 shall be met at the point of discharge and there shall be no Mixing Zone within 500 metres upstream in a river or artificial watercourse, or within 500 metres in a lake, from an intake for a community drinking water supply listed in Schedule WQL2.	