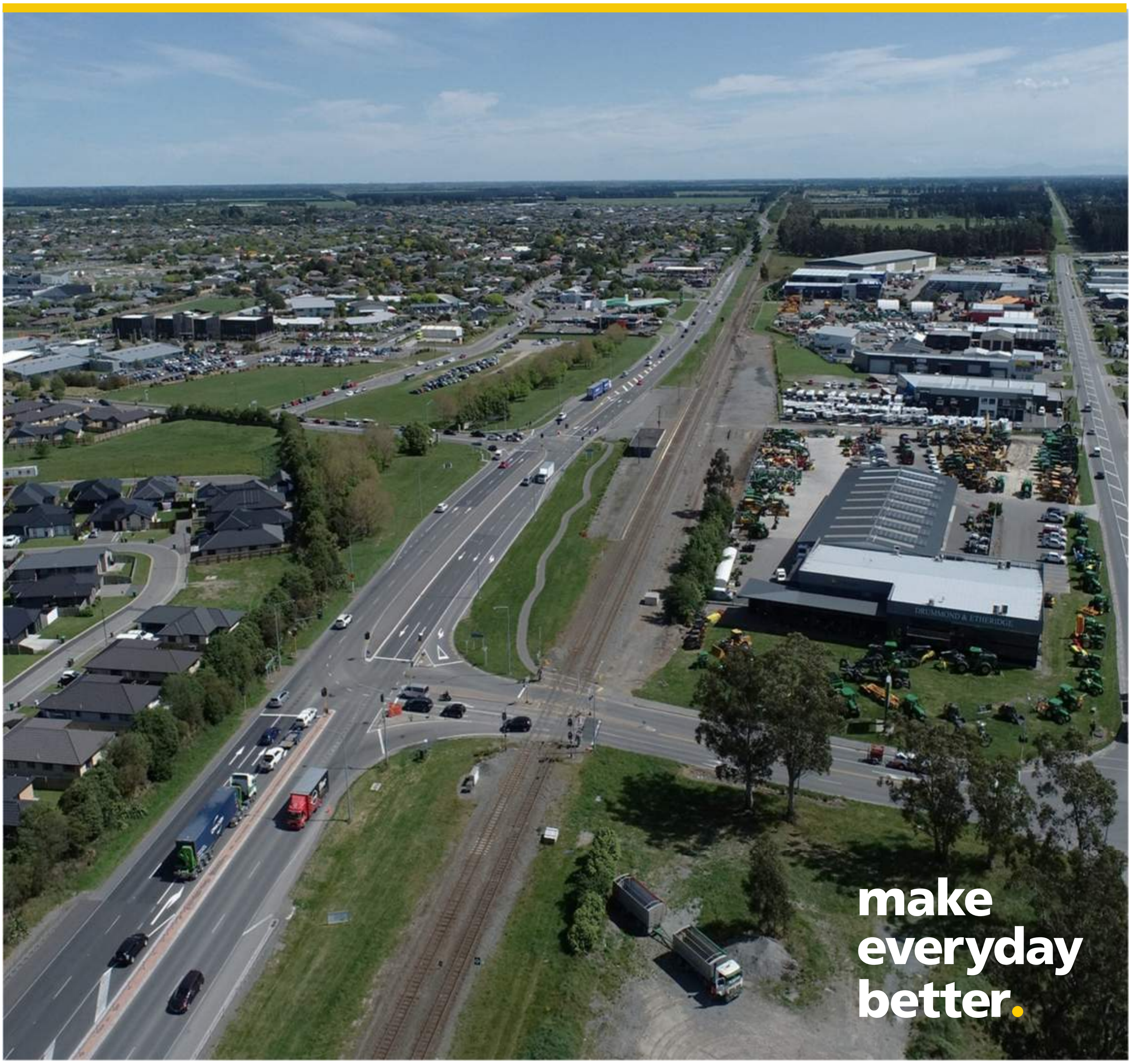
Sensitivity: General

SH1 Rolleston Access Improvements

Stormwater Management Report (Package 2)

Prepared for New Zealand Transport Agency Waka Kotahi
Prepared by Beca Limited

5 February 2025



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Revision History

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Action	Name	Signed	Date
Prepared by	Steve Harvey	S. Harvey	05/02/2025
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on behalf of	Beca Limited		

Executive Summary

This Stormwater Management Report has been prepared by Beca Limited (Beca) to inform the Assessment of Effects on the Environment (AEE) for the Notice of Requirement (NoR 1) and the s15 resource consent being sought by New Zealand Transport Agency Waka Kotahi (NZTA) for Package 2 of the overall SH1 Rolleston Access Improvements (the Project).

The Project includes a number of safety improvements to intersections along SH1 through Rolleston to reduce deaths, serious injuries and better manage the forecast future growth in traffic volumes. The Project includes two packages:

- Package 1 - SH1 / Dunns Crossing Road Roundabout and associated works.
- Package 2 - Overpass and balance of the works.

This report sets out the stormwater management approach for Package 2 only, which is currently in the Preliminary Design phase. The stormwater management approach considers both the road corridor catchments and the overland flow paths draining toward and across the road corridor.

Package 2 involves the construction of:

- An overpass connecting the residential and industrial areas of Rolleston, north and south of SH1 respectively, including facilities for pedestrians and cyclists.
- Safety improvements to intersections along SH1 through Rolleston, including ‘left out only’ at Hoskyns Road, and a ‘left in/left out only’ at Tennyson Street, Brookside Road and Rolleston Drive South.
- A service lane from the southbound off-ramp from Rolleston Drive (north) to provide left-in only access to McDonalds, BP service station, and Tennyson Street.
- Removal of the southbound merge lane at the end of the Christchurch Southern Motorway on SH1, north of Rolleston. This will include extension of two lanes from the Weedons Ross Road interchange to south of the new overpass.

This report described the proposed Package 2 stormwater management design and considers both the potential stormwater effects and expected stormwater effects (i.e. mitigated by the proposed stormwater management) for the construction and operational phases of Package 2.

Runoff from road corridor catchments is generally treated, attenuated and discharged to ground in accordance with NZTA’s guidelines and local Selwyn District Council standards. Where practicable, runoff from the road corridor catchments will be conveyed to stormwater basins which will provide first flush treatment, attenuation and discharge to ground. First flush treatment for the major catchments will be provided by first flush basins (volume based treatment) with infiltration through a designed sand media, prior to discharge to ground. Runoff in excess of the first flush volume will be conveyed to soakage basins, where stormwater up to the 1% AEP will discharge to ground. The first flush treatment within the first flush basins will be designed for the runoff from the first 25mm rainfall depth from a rainfall event. In catchments where connection to stormwater basins is not feasible but treatment is still proposed, first flush treatment will be provided in proprietary devices. Proprietary treatment systems will designed for the first flush flow (rather than first flush volume in the basins), being the flow rate from a 10mm/hr intensity rainfall.

Attenuation (or buffer storage) will be provided in the soakage basins (or soak pits), where stormwater will be stored when the rate of runoff into the soakage area exceeds the discharge rate to ground. The soakage with attenuation will discharge the runoff from the additional impervious area to ground, mitigating the potential impact of the increase in impervious area.

There are small sections of new impervious area at the extremities of the Package 2 footprint that are not able to be conveyed to the first flush basins. This is because the proposed highway vertical alignment and

the distance from the proposed basins means that stormwater from these areas is unable to drain to the basins. Stormwater from the minor catchments will be managed in a way that matches the existing network in each catchment, generally with collection via catchpits and soakage to ground, with additional soakage allowance to cater for the increase in impervious areas where appropriate. For most of these minor catchments no treatment will be provided. The exception to this is the Rolleston Drive catchment, where an increase in contaminant loads is expected, and first flush treatment will be provided by proprietary devices. Overall, the stormwater system will manage, treat and discharge to ground, runoff from a larger impervious area than the additional impervious area created by the Package 2 works.

The Package 2 stormwater management system mitigates the stormwater effects of the proposed works by providing first flush treatment, attenuation and discharge to ground up to the 1% AEP event for an area greater than the increase in impervious area of the project.

Where existing CSM2 SH1 stormwater network is to be removed to facilitate the new road corridor, the design allows for the treatment, attenuation and disposal to ground of stormwater for the existing catchment area of that asset.

In other areas where the existing stormwater network is affected by the Package 2 works (e.g. the stormwater assets will be removed or stormwater catchments are modified) the stormwater system will be modified to match the existing system design.

From SDC's flood modelling, there are existing overland flow paths across the Package 2 extents of SH1. It is not proposed to raise the level of the state highway as part of Package 2, and therefore these existing overflow paths will continue to flow across SH1 (i.e. no cross-culverts are proposed in the Package 2 design).

Where proposed raised centre islands would block existing overland flow paths, then options to mitigate changes to overland flow paths will be considered and confirmed during detailed design (e.g. cut-downs to allow flow through the island, or changing the extents of the islands).

A set of preliminary design stormwater drawings is included in Appendix A – RM Approval Drawings.

Construction-related discharge of sediment will be mitigated by the preparation and implementation of an Erosion and Sediment Control Plan.

1 Introduction

1.1 Purpose

This Stormwater Management Report has been prepared by Beca Limited (Beca) to inform the Assessment of Effects on the Environment (AEE) for the Notice of Requirement (NoR 1) and the s15 resource consent being sought by New Zealand Transport Agency Waka Kotahi (NZTA) for Package 2 of the overall SH1 Rolleston Access Improvements (the Project).

1.2 Project Description

Rolleston is one of the fastest growing towns in New Zealand and is experiencing transport pressures to keep the community connected and state highway intersections safe. In addition, there are increasing potential conflicts at road/rail crossings.

The urgent need for investment in the Rolleston transport network has been recognised as a Road of Regional (ROR) significance, with the Project part of the 'Canterbury Package'.

The project includes a number of safety and efficiency improvements (reduced deaths and serious injuries, greater travel choices and reduced travel times) on State Highway 1 and adjacent local roads in Rolleston.

The Project is being delivered in two packages:

- Package 1 - SH1 / Dunns Crossing Road Roundabout and associated works.
- Package 2 - Overpass and balance of the works.

This report discusses Package 2 only.

Package 2 involves the construction of:

- An overpass connecting the residential and industrial areas of Rolleston, north and south of SH1 respectively, including facilities for pedestrians and cyclists.
- Safety improvements to intersections along SH1 through Rolleston, including 'left out only' at Hoskyns Road, and a 'left in/left out only' at Tennyson Street, Brookside Road and Rolleston Drive South.
- A service lane from the southbound off-ramp from Rolleston Drive (north) to provide left-in only access to McDonalds, BP service station, and Tennyson Street.
- Removal of the southbound merge lane at the end of the Christchurch Southern Motorway on SH1, north of Rolleston. This will include extension of two lanes from the Weedons Ross Road interchange to south of the new overpass.

The improvements will provide for a safe crossing of the State Highway via the overpass between Jones Road and Rolleston Drive, along with improved safety of vehicles accessing and egressing the state highway.

The Package 2 footprint is shown in Figure 1-1.

Figure 1-1: Package 2 Footprint (approximate and indicative only)

1.3 Scope of Report

The scope of this report is to set out the stormwater management approach for Package 2, i.e. the overpass and balance of the works. This includes the capture, conveyance, treatment, attenuation and discharge of runoff from the state highway and local authority roads, as well as conveyance of existing overland flows across the state highway. It also includes the construction phase stormwater management, and the operation and maintenance of the stormwater system.

This report will form part of the NoR and resource consent application to Environment Canterbury.

This report should be read alongside the AEE, which contains further details on the history and context of the Project. The AEE also contains a detailed description of works to be authorised under the NoR and applied for under the regional consents, and the typical construction methodologies that will be used to implement this work.

The AEE also assesses consistency with the regional policies and planning frameworks relevant to the Project.

2 Existing Environment

2.1 General Environment

The topography of Package 2 and the surrounding area forms part of the Canterbury Plains and is therefore generally flat, with a gentle slope from northwest to southeast. Package 2 is entirely within Selwyn District.

Package 2 is in Rolleston, approximately 22km southwest of Christchurch. SH1 extends approximately 4km through Rolleston in a northeast to southwest orientation. The rail corridor separates SH1 from the land to the northwest. The formed corridors of SH1 and the South Island Main Trunk (SIMT) railway run parallel to each other (separated by approximately 30m). The immediate area surrounding Package 2 consists of residential housing (south of SH1) and commercial/industrial properties (north of SH1). Some rural grassland is present adjacent to the north-east of the Hoskyns Road section.

2.2 General Ground Conditions

The ground conditions for Package 2 are generally a mixture of sands, silts and gravel. Typically, the nearer surface material contains a larger proportion of silts and organics, with underlying sandy gravels that appear to have better drainage properties. The composition of, and depth to the sandy gravel material varies across site. Fill material was encountered at the northern end of the proposed overpass between SH1 and Jones Road, consisting of sandy gravel up to the ground surface.

The ground investigations were commenced on 17 June 2024 and was completed by 9 August 2024.

The physical works for the ground investigations were undertaken by Corde Ltd, and the full description of the investigations is reported in the Geotechnical Interpretive Report which has been included in Appendix B.

2.3 Contaminated Land Detailed Site Investigation

The Rolleston Access Improvements (Package 2) Detailed Site Investigation (DSI) (Contamination) Report is included in Appendix D.

Soil sampling for contamination was undertaken as part of the ground investigations from 14 machine excavated test pits, 18 pavement pits and 7 infiltration pits, 2 boreholes, and 3 utility services trenches. The investigation targeted the various areas of proposed development within the site, and HAIL[1] activities/potential sources of contamination identified in the project Preliminary Site Investigation (Stantec 2023). Samples were analysed for at least one of the following contaminants of concern: heavy metals (antimony, arsenic, cadmium, copper, chromium, nickel, mercury, tin and zinc), polycyclic aromatic hydrocarbons (PAH), total petroleum hydrocarbons (TPH), semi-volatile organic compounds (SVOC), and asbestos.

An extract of the testing locations from the DSI is shown in Figure 2-1.

[1] Ministry for the Environment Hazardous Activities and Industries List (HAIL)

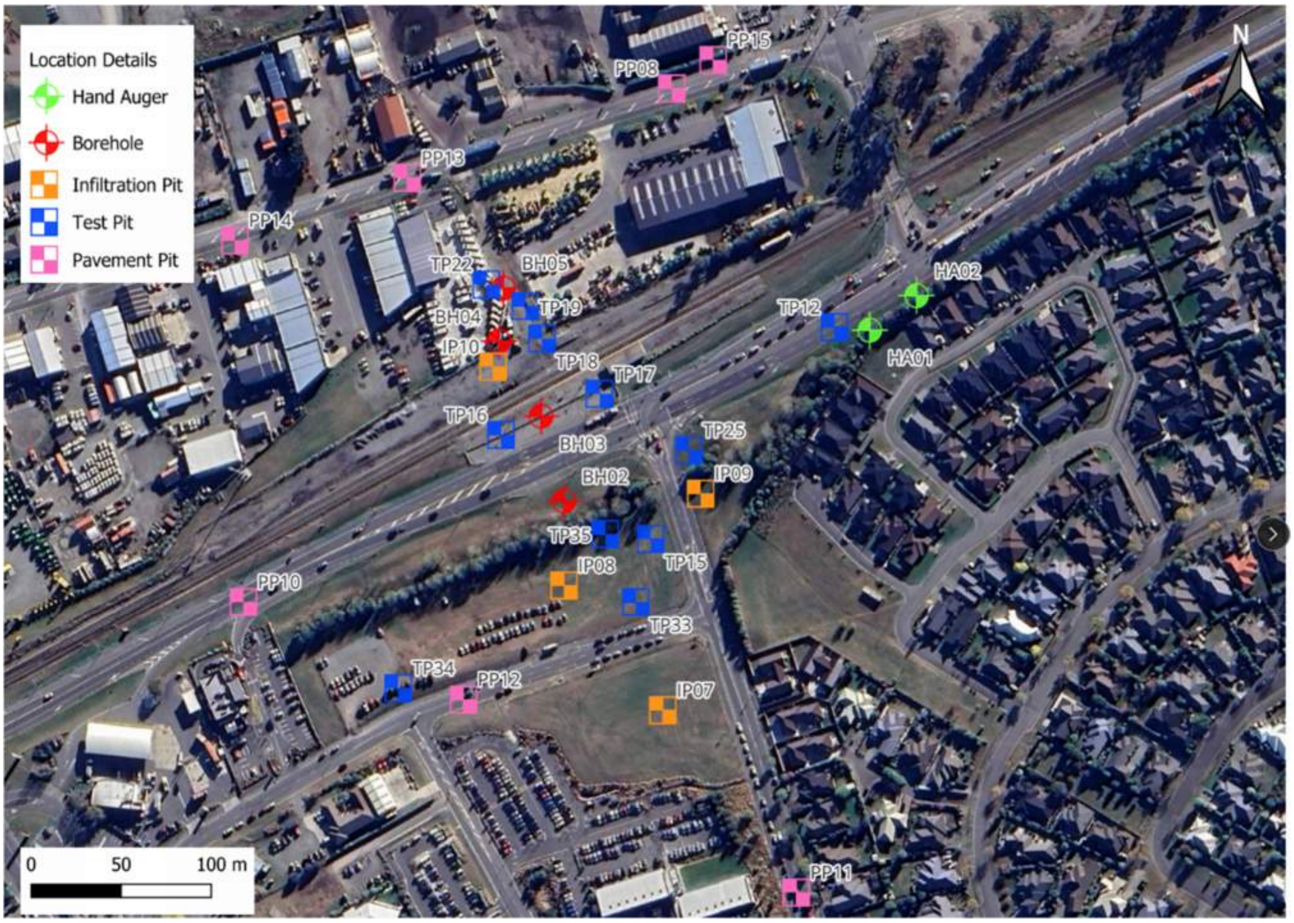

Figure 2-1: Site Investigation Plan

Uncontrolled waste material was encountered in pavement pit PP10, and the stained and odorous material in shallow soils around TP18, TP22, IP10 and BH05. If this stained and odorous material is encountered in areas where stormwater soakage or swales are proposed, the DSI recommended removing this material and replacing with cleanfill (as required).

Deeper soils (greater than 0.7 m below ground level (bgl)) within the IP10, and TP18, TP22, IP10 and BH05 (greater than 0.5 m bgl) were around or below published background concentrations for the analytes tested, and were considered suitable for stormwater discharges.

Testing was also proposed at the location of the second of the proposed stormwater basins within the overpass north catchment (as the area has been identified as having HAIL activity/ies on it), however due to land ownership and access limitations this has not yet been carried out.

Testing at TP20 and TP21 (refer Figure 2-2) will be carried out prior to construction of the basins and will inform whether mitigation measures are required. If the material is not considered suitable for reuse on site, it would require disposal to a facility licensed to accept the concentrations observed.

Further investigation and/or management during construction is also required for PP10 to confirm the extents of the uncontrolled waste observed during the DSI in proximity to the swales, and to determine whether any mitigation measures are required. If the material is removed, it is not considered suitable for reuse on site and would require disposal to a facility licensed to accept the contamination levels observed.

For the remainder of the site, based on the observed results, soils were considered suitable for reuse across the site, including within swales and basins.

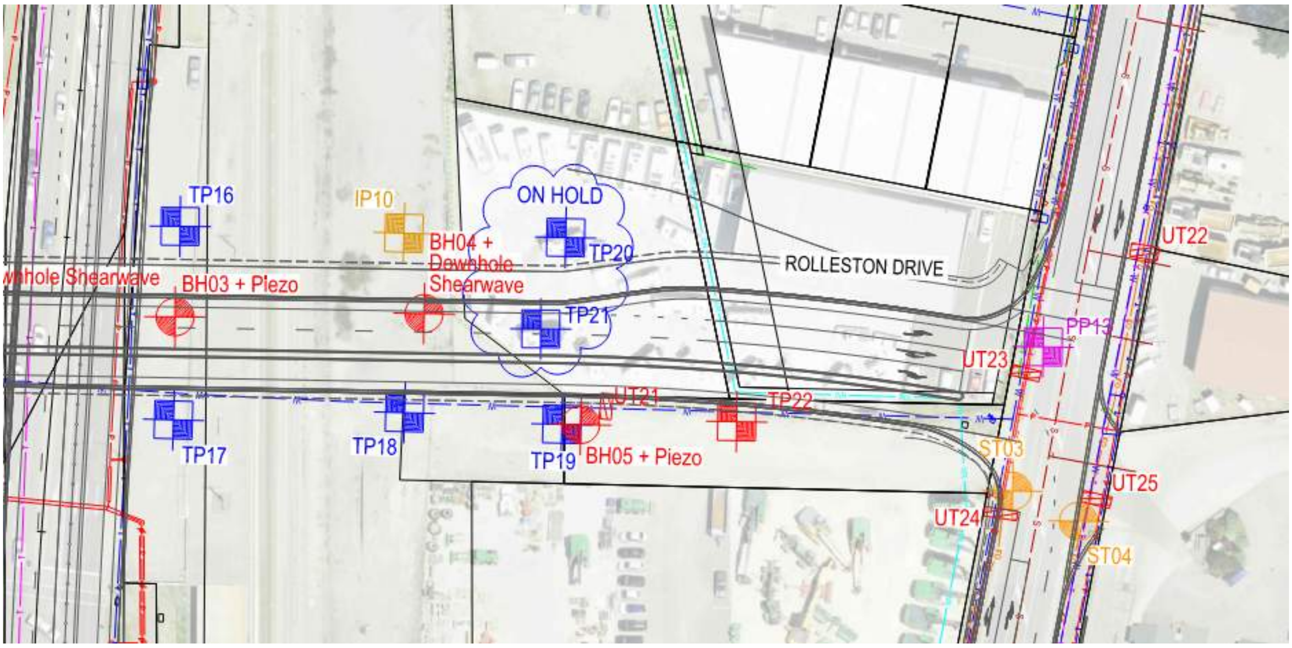

Figure 2-2: Test Pits TP20 & TP21 Location Plan

2.4 Groundwater

As part of the geotechnical investigations, several piezometers were installed to investigate groundwater levels. BH03 and BH05 are located within the Package 2 footprint, near to the proposed first flush and soakage basins (refer to Section 4). The depth to groundwater was measured in the piezometers on 21 August 2024 at 11.98m below ground level (bgl) in BH03 and 11.91m bgl in BH05.

Nearby bores listed in the ECan wells database indicate that shallow groundwater levels are generally found at 10-20m bgl in the area, with the regional piezometric contours indicating approximately 15m bgl, with regional groundwater flowing towards the southeast. These features are shown in Figure 2-3 and Figure 2-4.

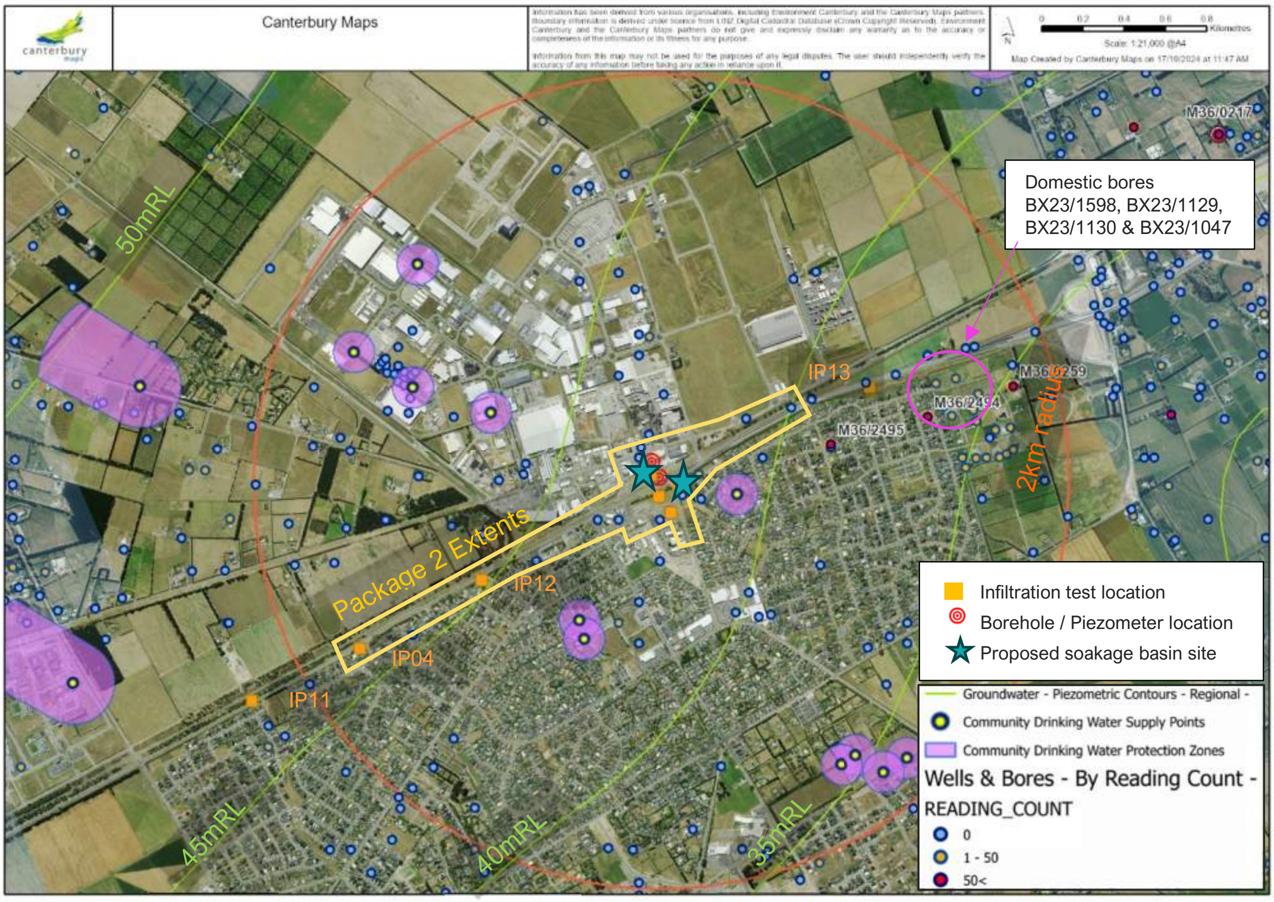

Figure 2-3: Package 2 area showing local ground investigation information and groundwater bores within 2km (adapted from Canterbury Maps on 16/10/2024).

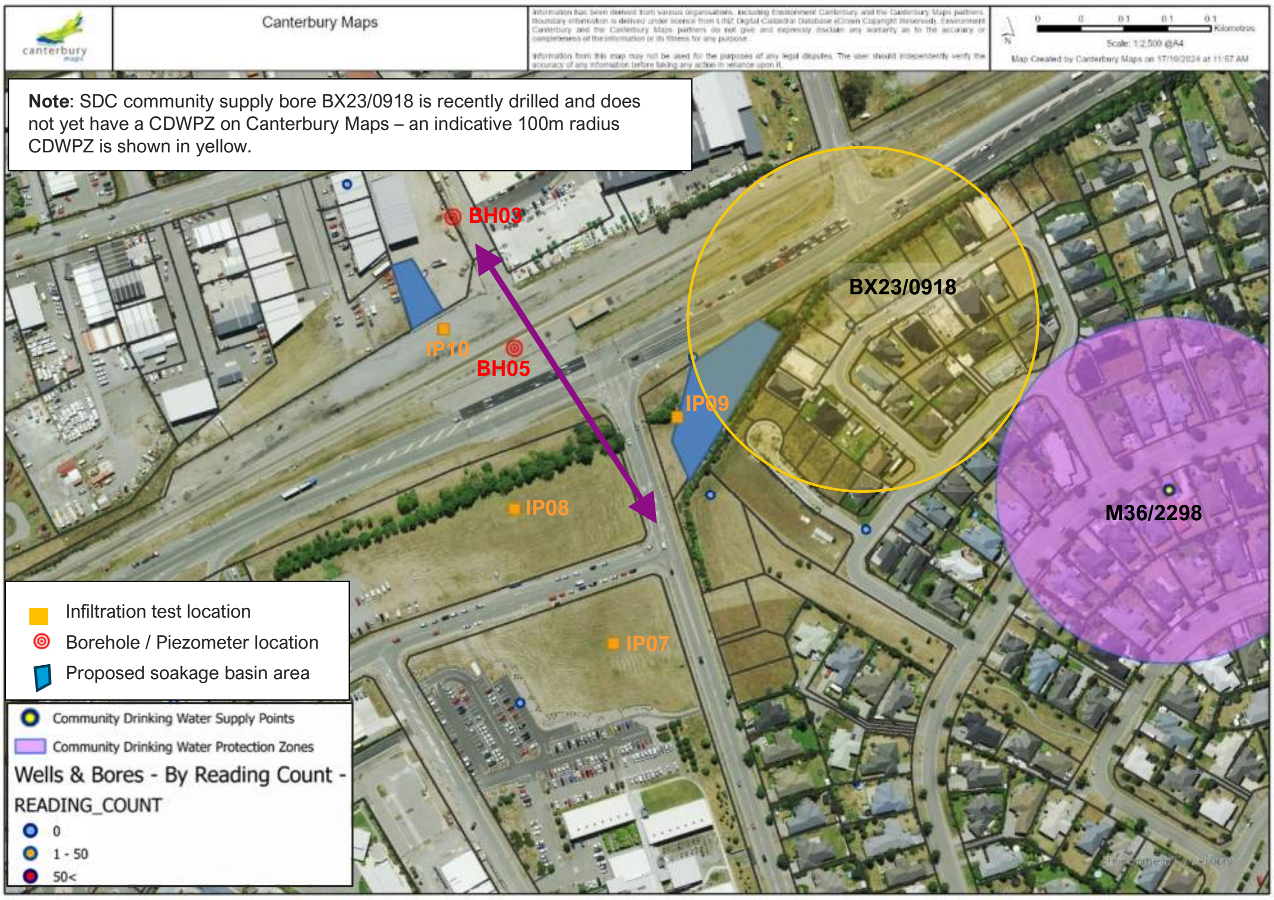

Figure 2-4: Groundwater investigation bores near the Overpass location (approximated by purple arrow) (accessed 16/10/2024).

Our review of the ECan Wells database indicates that there are 92 bores within 2km of the overpass. The reported bore use types are summarised by the following:

- 15x domestic & stockwater
- 38x domestic
- 15x irrigation
- 5x groundwater level observation or geotechnical investigation
- 2x commercial/ industrial
- 1x stock supply
- 14x community water supply
- 2x other or not stated.

Of these 92 bores, 5 are located within the Package 2 extent shown in Figure 2-4. Four of these are geotechnical investigation bores and therefore do not need to be considered with regard to potential effects. The fifth bore is a domestic supply bore with status ‘buried, unlikely still exists’, and therefore can also be disregarded.

An additional 18 bores are located within approximately 200m downgradient of the Package 2 site area. Of these, 10 are sealed, buried, or not used, and the remaining 8 have ‘active’ status. Five of these active bores are domestic supply bores, two are community water supply bores and one is used for water level observation. The domestic bore M36/0009 is unlikely to exist as the land party owner (BP Limited) advised ECan in 2013 that the bore was not found at its given location.

The four remaining domestic bores (BX23/1598, BX23/1129, BX23/1130 & BX23/1047) are screened at approximately 45-48m depth and are located near Haymakers Crescent (shown in Figure 2-4). The land parcels these bores are sited on are all connected to the Rolleston Town Supply, and the actual bore usage may need to be confirmed (should potential effects be identified at these bores – see Section 8).

The two nearby community drinking water supply bores are BX23/0918 and M36/2298, which are deep (screened at 93.5-99.5m in M36/2298 and 182.7 – 190.2m bgl in BX23/0918), high yielding bores that supply Rolleston with drinking water. The take and use of water from M36/2298 is consented under CRC193859 for 52.8L/s, whereas BX23/0918 does not appear to have a consent to take and use water. These bores are shown in Figure 2-3 and Figure 2-4.

The likely future source protection zone for BX23/0918 has been estimated based on Schedule 1 of the Land and Water Regional Plan (LWRP) (i.e. 100m radius zone as shown in Figure 2-4). This CDWPZ zone overlaps the site of a proposed first flush and soakage basin. However SDC has advised that it is not going to use this bore(email received 25 October 2024).

The nearest ECan monitoring bores (Not shown on the above figures) are M36/5248, 2.5km west of the overpass site, and M36/4126, 4.1km north of the site. These bores indicate that the local groundwater quality is reasonably typical of Canterbury Plains groundwater. There is no available ECan groundwater quality data directly downgradient of the site.

2.5 Existing Overland Flow Paths

Based on the Selwyn District Council's 200 year ARI flood maps (refer excerpt shown in Figure 2-5), it is understood that there are secondary flow paths from the northwest of Package 2, heading southeast across SH1.

Due to the relatively flat topography of the road, the flow paths are generally spread out along the length of the Package 2 extents. There is however a concentration of flows across SH1 east of Hoskyns Road. These flows cross SH1 before being intercepted by the earth bund along the south of the road corridor. This bund diverts some of the flow east away from the site extents and some of the flow west to Rolleston Drive.

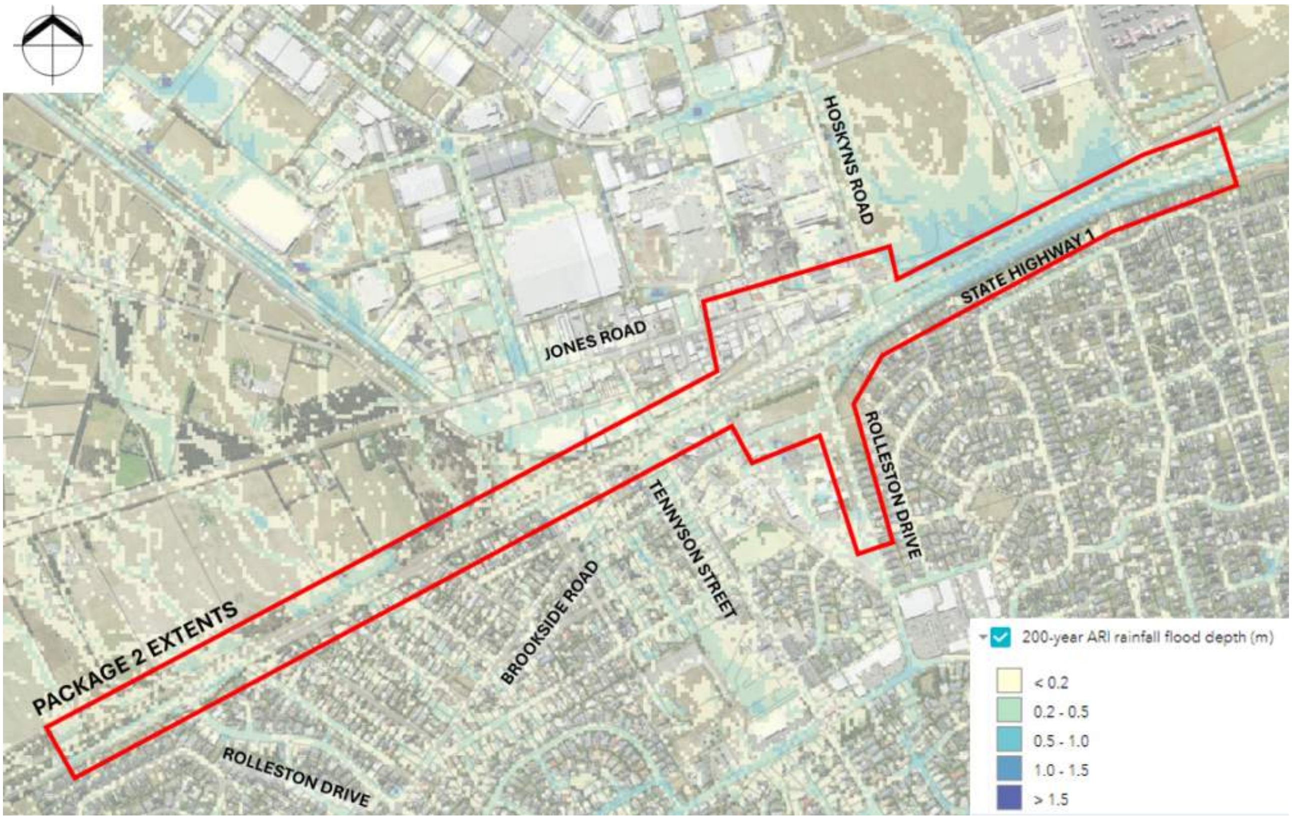

Figure 2-5: SDC 200 Year ARI Flood Map (accessed 04/10/2024)

2.6 Existing Stormwater System

2.6.1 Overview

According to the data available in Canterbury Maps Viewer, consultation with NZTA and SDC, and a site inspection, most of the existing stormwater infrastructure within the Package 2 footprint is located within the SDC road network.

Figure 2-6 illustrates the existing stormwater infrastructure shown in Canterbury Maps Viewer relative to the boundaries of Package 2. Stormwater nodes (catchpits and soak pits) are marked as green dots, while stormwater pipes are indicated by green lines. Additionally, there is a water race that crosses the site, depicted by the blue lines (covered section) and yellow lines (open section).

While no state highway assets are shown in the Canterbury Maps Viewer, NZTA has provided as-built information detailing the CSM2 highway upgrade works and associated drainage starting just east of Hoskyns Road, heading towards Christchurch.

Given the presence of near-surface gravels and high soakage rates, the local drainage system mainly consists of catchpits directly connected to soak pits. There are also brief segments of stormwater piped reticulation networks that seem to connect to soak pits. The CSM2 system includes catchpits on both sides of SH1, connected by pipes draining to a bunded swale on the south side of SH1 providing treatment, attenuation and discharge to ground.

A site walkover conducted by Beca on 5 August 2024 confirmed the existence of the assets identified on Jones Road, Hoskyns Road, Rolleston Drive, and Tennyson Street as seen in Canterbury Maps Viewer.

All known and identified existing stormwater drainage features have been included in the resource consent stormwater drawings found in Appendix A.

The existing stormwater infrastructure is shown in Figure 2-6 and is described in more detail by area in the following sections.

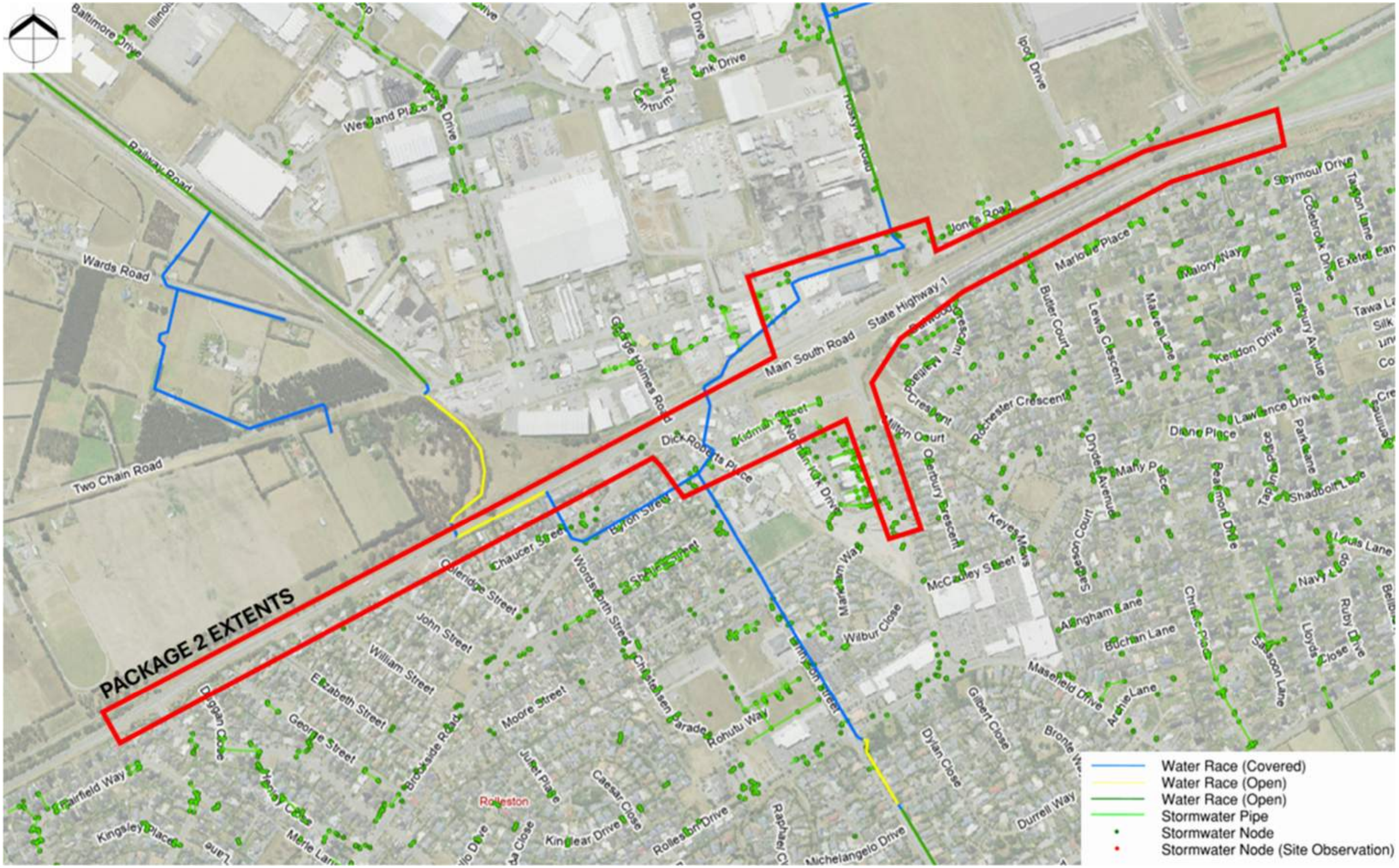

Figure 2-6: Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

2.6.2 State Highway 1

There is limited existing drainage infrastructure along the state highway within the Package 2 extents, with a small number of catchpits and soak pits being identified from Google Street View and from the site walkover. The areas which could be accessed during the site walkover were limited due to safe access, so it wasn't possible to inspect any of the drains and soak pits along the state highway.

Between Hoskyns Road and the eastern extents of the Package 2 works, the as-built information for the CSM2 project provided by NZTA shows an existing stormwater network consisting of catchpits on both sides of SH1, connected by pipes draining to a bunded swale on the south side of SH1 providing treatment, attenuation and discharge to ground through the base of the swale. The existing swale is shown along the south side of SH1 (see Figure 2-7), but the piped network is not shown and can be seen on the as-built drawings.

Figure 2-7: Hoskyns Road to Scheme Extents Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

A selection of the CSM2 as-built information has been provided in Appendix C and the network has also been included on the resource consent stormwater drawings in Appendix A.

Apart from the infrastructure constructed as part of the CSM2 project, there is very limited existing stormwater infrastructure along SH1. It is assumed that generally stormwater runoff from the highway in low intensity events generally discharges to land, soaking to ground in the berm areas of the state highway (i.e. informal discharge to ground), except where collected by the small number of catchpits and soak pits. During more intense periods of rainfall it is assumed that stormwater runoff from SH1 discharges south of the state highway following the natural contours of the land, discharging either to land or to the SDC network.

2.6.3 Jones Road & Hoskyns Road

Existing SDC stormwater assets on Jones Road and Hoskyns Road consist of catchpits with connections to soak pits (see Figure 2-8)

SDC has advised that the only recent report of flooding from the area was in relation to a kerb breakout on the corner of Hoskyns Road and State Highway 1 (identified by the red dot in Figure 2-8. Following a site inspection on 5 August 2024 it was noted that the kerb breakout inlet in this location was filled with debris and the vegetation behind the inlet was overgrown. It has been noted that since the site inspection, maintenance works have been carried out in the area, with the kerb breakout and vegetation having been cleared.

Figure 2-8: Jones Road & Hoskyns Road Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

2.6.4 Tennyson Street

There is limited existing stormwater infrastructure on Tennyson Street, both within and in proximity to the Package 2 extents (refer Figure 2-9). There is a single catchpit on the corner of Tennyson Street and Dick Roberts Place (shown by the green dot in Figure 2-9), along with a single catchpit on SH1 outside of the BP service station (shown by a red dot). It is assumed that they both discharge to ground via soak pits.

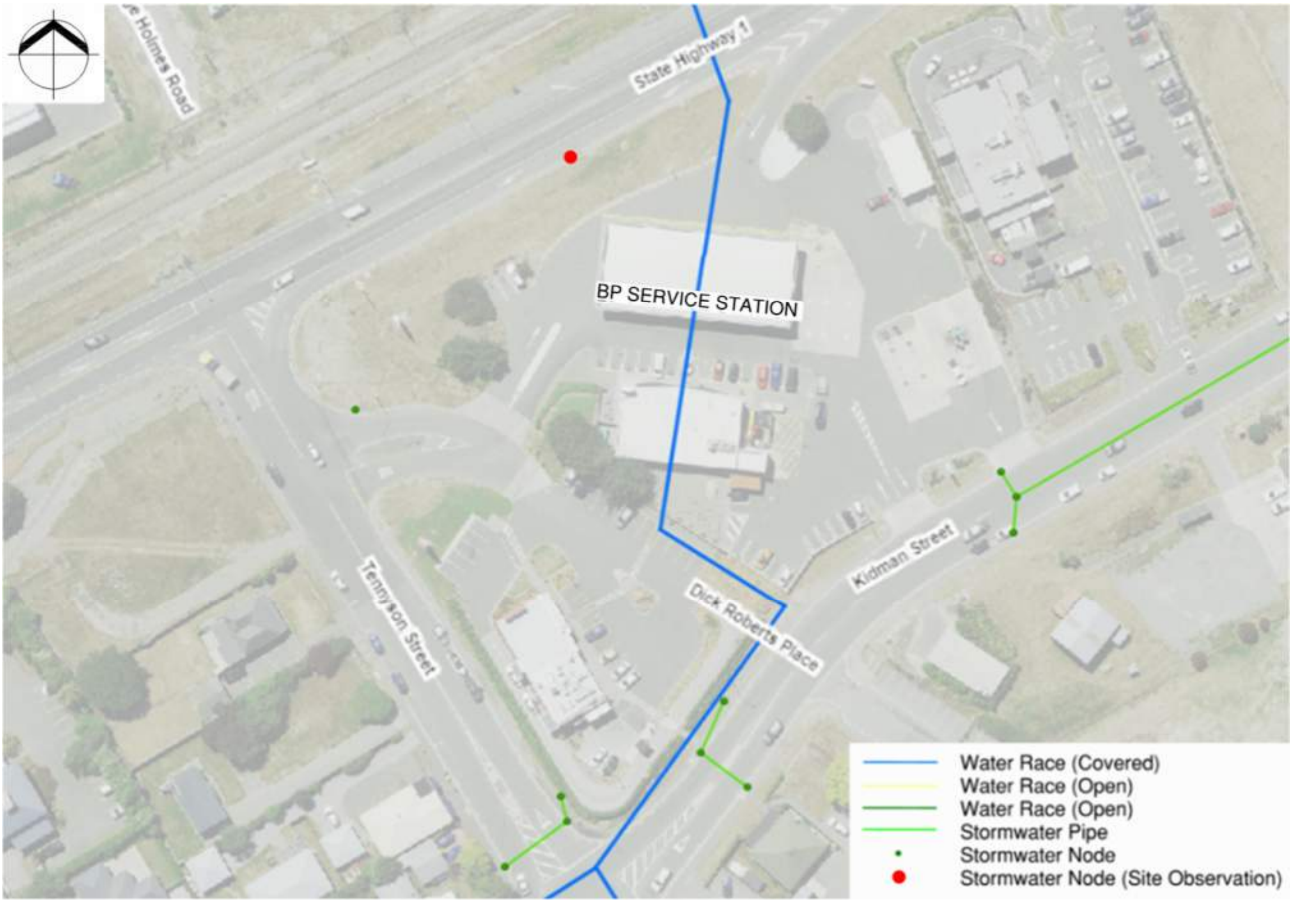

Figure 2-9: Tennyson Street Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

2.6.5 Rolleston Drive (North)

There are several catchpits and soak pits along Rolleston Drive (North), south of the Kidman Drive junction, within the Package 2 extents. It is assumed that all the soak pits discharge to ground via soakage.

Figure 2-10: Rolleston Drive (north) Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

2.6.6 Kidman Street

There are several catchpits and soak pits along Kidman Street, west of the Rolleston Drive (North) junction, within the Package 2 extents. The catchpits in this area appear to connect a small stormwater pipe network that discharges to a stormwater management area within the SDC park and ride area. It is assumed that this network discharges to ground via soak pit.

Figure 2-11: Kidman Street Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

2.6.7 Rolleston Drive (South)

There is limited drainage on Rolleston Drive (south), both within and in proximity to the Package 2 extents. There are two single catchpits (shown by the green dots) shown south of the state highway junction. They appear to be connected by a short section of stormwater pipe network and it is assumed that they both discharge to ground via soakage.

There are two single catchpits (shown by the red dots) along the state highway. Either side of each catchpit is a short section of swale. It is assumed that both of these catchpits discharge stormwater runoff from the state highway to ground via soakage. It is also possible that these catchpits are connected by a section of pipe and act as a bubble for stormwater to flow across the junction that may have block a previous flow path when constructed.

Figure 2-12: Rolleston Drive (South) Existing SDC Stormwater Infrastructure from Canterbury Maps (accessed 10/10/2024)

3 Design Philosophy

3.1 Design Standards and Guidelines

A number of design standards and guidelines have been used in the development of the stormwater design for Package 2. These include, but are not limited to:

- NZTA Waka Kotahi, P46 Stormwater Specification (P46), 2016
- NZTA Waka Kotahi, Stormwater Treatment Standard for State Highway Infrastructure, 2010
- NZTA Waka Kotahi, Bridge Manual, Third edition, Amendment 4, 2022
- TNZ Highway Surface Drainage, A Design Guide for Highways with a Positive Collection, 1977
- NZTA TM-2502 Preferred method for calculating road surface water run-off in New Zealand, 2021
- Selwyn District Council (SDC), Engineering Code of Practice (ECOP), 2012
- Christchurch City Council (CCC), Waterways Wetlands and Drainage Guide (WWDG), Part A, 2003
- Christchurch City Council (CCC), Waterways Wetlands and Drainage Guide (WWDG), Part B, 2012
- Ministry for the Environment (MfE), National Policy Statement for Freshwater Management 2020 (NPS-FM), Amended January 2024
- CIRIA, CIRIA C753 The SuDS Manual 2015, version 5 2016
- CRC for Water Sensitive Cities, Adoption guidelines for stormwater biofiltration systems: Cities as water supply catchments – sustainable technologies, 2015
- KiwiRail, Track Standard, Level Crossings (T-ST-AM-5360), Issue 2, 2018

3.2 Design Assumptions

The key design assumptions are:

- Treatment and attenuation of stormwater is required in general accordance with NZTA's guidelines and local Selwyn District Council standards. This includes first flush treatment for water quality, attenuation and discharge to ground of runoff from the road corridor up to the 1% AEP event, where practicable.
- As a minimum, the design will include first flush treatment, attenuation and disposal to ground up to the 1% AEP event for an impervious area equal to the additional impervious area created by the project.
- The Package 2 works do not increase flood levels in the surrounding catchments or increase flood risk to private properties up to the 1% AEP event. This is managed by retaining existing site levels in critical locations so that existing overland flow paths are not altered by the Package 2 works (i.e. flow paths stay the same as the existing flow paths, with no new cross-drainage provided).

3.3 Design Constraints

The following constraints have been identified for the Package 2 stormwater design:

- Existing ground levels
- Management of road corridor catchments and secondary flow paths
- Management of overland flow paths
- Geotechnical conditions
- Contaminated land

3.4 Key Design Parameters

3.4.1 Rainfall

The design rainfall event intensities for the Package 2 shall be in accordance with Table 3-1. These rainfall intensities have been adopted from HIRDS Version 4, with climate change allowance for scenario RCP 8.5 (2081-2100).

Table 3-1: Rainfall intensities (mm/hr) from HIRDS V4 RCP8.5 for the period 2081-2100 (Accessed 04/04/2024)

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h
1.58	0.633	32.1	21.4	17.1	11.9	8.26	4.53	3.02	1.96	1.21	0.887
2	0.5	36.3	24.1	19.3	13.4	9.32	5.09	3.39	2.18	1.35	0.991
5	0.2	51.6	34.1	27.2	18.8	13	7.05	4.66	2.98	1.83	1.35
10	0.1	63.9	42.1	33.5	23	15.9	8.57	5.65	3.6	2.2	1.62
20	0.05	77.4	50.8	40.3	27.6	19	10.2	6.7	4.24	2.59	1.9
30	0.033	85.9	56.3	44.6	30.5	20.9	11.2	7.35	4.64	2.83	2.07
40	0.025	92.1	60.2	47.7	32.6	22.3	12	7.82	4.94	3	2.19
50	0.02	97.3	63.5	50.3	34.3	23.5	12.5	8.19	5.16	3.14	2.29
60	0.017	101	66.2	52.4	35.7	24.4	13	8.51	5.36	3.25	2.38
80	0.013	108	70.7	55.9	38	25.9	13.8	9	5.66	3.44	2.5
100	0.01	114	74.1	58.5	39.8	27.1	14.4	9.41	5.91	3.58	2.6
250	0.004	137	89	70.1	47.4	32.2	17	11	6.9	4.16	3.02

The first flush rainfall depth is 25mm.

3.4.2 Runoff Coefficients

The runoff coefficients used for peak flow, volume and first flush are shown in Table 3-2 and Table 3-3.

First flush volumes have been determined using NZTA P46, with the remaining peak flow and volume coefficients determined using CCC WWDG.

Table 3-2: Runoff Coefficients Peak Flow

Events	C Pervious	C Impervious	C Industrial / Business	C Residential	C Rural
First Flush	0.15	0.95	-	-	-
10% AEP	0.30	0.85	0.77	0.42	0.30
1% AEP	0.35	0.99	0.82	0.47	0.35

Table 3-3: Runoff Coefficients Volume

Events	C Pervious	C Impervious	C Industrial / Business	C Residential	C Rural
First Flush	0.15	0.95	-	-	-
10% AEP	0.35	0.98	0.89	0.44	0.35
1% AEP	0.35	0.99	0.89	0.44	0.35

3.4.3 Catchments

Package 2 catchments are defined according to the geometric design of the road and the surrounding ground levels (from topographical survey and LiDAR).

Only road corridor catchments have been defined as part of the Package 2 design. Existing overland flow paths are to be maintained by retaining the existing road levels where-ever possible or providing cut-downs in median islands to facilitate flows across the road corridor.

3.4.4 First Flush Volumes

For volume based treatment devices, such as first flush basins, first flush volumes will be calculated using a first flush rainfall depth of 25mm, with a 0.95 runoff coefficient for the impervious area and 0.15 runoff coefficient for the pervious area.

3.4.5 First Flush Flows

For flow based treatment devices (e.g. flow-through proprietary devices such as StormFilters), first flush flow rates will be calculated using a first flush rainfall intensity of 10mm/hr, with a 0.95 runoff coefficient for the impervious area and 0.15 runoff coefficient for the pervious area.

3.4.6 Ground Soakage Rates

The geotechnical investigations included a number of soakage tests to investigate the existing subsurface capacity for stormwater soakage. The full details of the soakage testing are reported in the Memorandum on Infiltration (Soakage) Testing for Rolleston Access Improvement Project. This memo forms part of the Geotechnical Interpretive Report which has been included in Appendix B.

The measured soakage test rates were factored to arrive a design soakage rate, with the factor of safety used based on the potential consequence of failure in accordance with Table 3-4, taken from Table 25.2, SuDS, CIRIA 2015.

Table 3-4: Suggested Factors of Safety for Hydraulic Design of Soakage Systems (CIRIA, 2015)

Size of Area to be Drained (m^2)	Consequence of Failure		
	No Damage or Inconvenience	Minor Damage to External Areas or Inconvenience	Damage to Buildings or Structures, or Major Inconvenience
<100	1.5	2	10
100-1000	1.5	3	10
>1000	1.5	5	10

Based on the catchment areas and the likely consequences of failure, a factor of safety of 5 has been applied to the measured soakage rates to derive the factored soakage rates, as shown in Table 3-5 below. Factored soakage rates were limited to a maximum of 300mm/hr due to the uncertainty of long-term performance.

Table 3-5: Summary of Measured and Design Soakage Rates for Package 2

Infiltration Pit ID	Location	Measured Soakage Rate (mm/hr)	Design (Factored) Soakage Rate (mm/hr)
IP07	South West of Rolleston Drive / Kidman Street Junction	389	77
IP08	North West of Rolleston Drive / Kidman Street Junction	243	48
IP09	South East of Rolleston Drive / State Highway 1 Junction	2,325	300*
IP10	KiwiRail Land Below Overpass	497	99

Note(s):

* Design soakage rate limited to a maximum of 300mm/hr due to the uncertainty of long-term performance

The design soakage rate used in the design of each soakage facility is based on the nearest test location. Further soakage testing is required at construction to confirm the design soakage rates used.

3.4.7 Roughness Coefficients

Manning's roughness coefficients will be as summarised in Table 3-6.

Table 3-6: Roughness Coefficients

Description	Manning's n
Swales (flow along)	0.035 – 0.05
Concrete pipes	0.012
Concrete culverts	0.012
Kerb & channel and pavement	0.013

3.4.8 Survey Datum

All levels are in Christchurch Drainage Datum (CDD), which is the drainage datum for Christchurch. This datum is 9.043m below Lyttelton Vertical Datum 1937 (mean sea level, MSL), or in other words in Christchurch Drainage Datum (CDD), MSL=9.043m.

3.5 Proposed Design Departures

Proposed departures are outlined in Table 3-7.

Table 3-7: Proposed Design Departures

Topic	Description
Minimum Pipe Cover	Reduce pipe cover from 1.2m to 0.5m while meeting NZS 3725 or AS/NZS2566.1 to enable shallower stormwater systems (including connections to swales and basins).
Minimum Pipe Velocities	Due to the flat grade of the Canterbury Plains and to enable shallow stormwater systems (including connections to swales and basins), flatter pipe gradients may be required which may result in pipe velocities less than the minimum pipe velocities in the first sections of the pipe.
Swale Underdrainage System	Where a swale is at a grade of less than 2%, an underdrainage system (subsoil drain) is usually required as per NZTA P46. Due to the relatively flat grades of the alignment and existing ground, subsoils would likely be required under all swales. Due to the high soakage rates expected within the project extents, no subsoil drains under swales are proposed.
Overland Flow Paths	Existing overland flow paths are to be maintained by keeping the existing road levels and flow paths or where there are new central islands providing cut-downs through the islands to facilitate flows across the road corridor in the existing locations. P46 states that this is acceptable by NZTA on an exemption basis.

4 Long-Term Stormwater Management

4.1 Introduction

Unless stated otherwise, the stormwater management approach discussed in this report relates to the long-term stormwater management for Package 2. The short term or construction stormwater management is discussed in Section 6 of this report.

4.2 Operation and Maintenance

A Stormwater Operation and Maintenance Manual will be prepared as part of the overall Operation and Maintenance Manual for the Project. This will include requirements and schedules for inspection, operation and maintenance of all stormwater management system including catchpits, pipes, culverts, basins, and planting. Refer to Section 7 of this report for further information.

4.3 Overview of Stormwater Design Approach

4.3.1 General Overview

The stormwater management system for Package 2 considers the two main stormwater contributors:

- Road corridor catchments
- Overland flow paths

The design of the stormwater management system has been carried out using the design philosophy set out in Section 3 of this report.

The design of the stormwater system by catchment is set out in Section 5 of this report.

The stormwater design for the Package 2 road corridor runoff manages stormwater through various elements such as collection and conveyance, treatment, discharge to ground and attenuation. Key features include the use of kerb and channel, catchpits, manholes, swales, and pipework to effectively collect and convey stormwater. The system also uses first flush basins for treatment of major catchments, mitigating contaminants coming from the road corridor during the early part of a storm event. Proprietary treatment devices are also proposed in areas where space is restricted, and treatment of minor catchments is required. Infiltration and soakage methods are used for discharging stormwater to ground, with attenuation basins providing storage.

As a minimum the design includes the treatment, attenuation and disposal to ground of stormwater runoff from the road corridor for an equivalent area to the additional impervious area created by Package 2. The new impervious areas are identified in Section 4.3.2.1.

Where existing CSM2 SH1 stormwater network is to be removed to facilitate the new road corridor, the design allows for the treatment, attenuation and disposal to ground of stormwater for the existing catchment area of that asset.

In other areas where the existing stormwater network is affected by the Package 2 works (e.g. the stormwater assets will be removed or stormwater catchments are modified) the stormwater system will be modified to match the existing system design. These catchment areas are identified in Section 4.3.2.2.

The geometric design of Package 2 will not affect the existing overland flow paths across the state highway (refer Section 2.5). Therefore, no cross-drainage is proposed and there will be no effect on the flood levels in the surrounding catchments or flood risk to private properties.

Where proposed raised centre islands would block existing overland flow paths, then options to mitigate changes to overland flow paths will be considered and confirmed during detailed design (e.g. cut-downs to allow flow through the island, or changing the extents of the islands).

4.3.2 Road Corridor Catchments

4.3.2.1 Additional Impervious and Pervious Areas

Figure 4-1 below shows the changes to impervious and pervious areas as a result of the proposed works within Package 2:

- Additional impervious area (i.e. existing pervious areas that will become impervious) are shown in blue; and
- Additional pervious areas (i.e. existing impervious areas that will become pervious) are shown in green.

Table 4-1: Proposed additional impervious and pervious areas and overall increase in impervious area

Description	Area (m²)
Additional impervious area	17,400
Additional pervious area	5,100
Overall increase in impervious area	12,300

There is an overall increase in impervious area within Package 2 of approximately 12,300m².

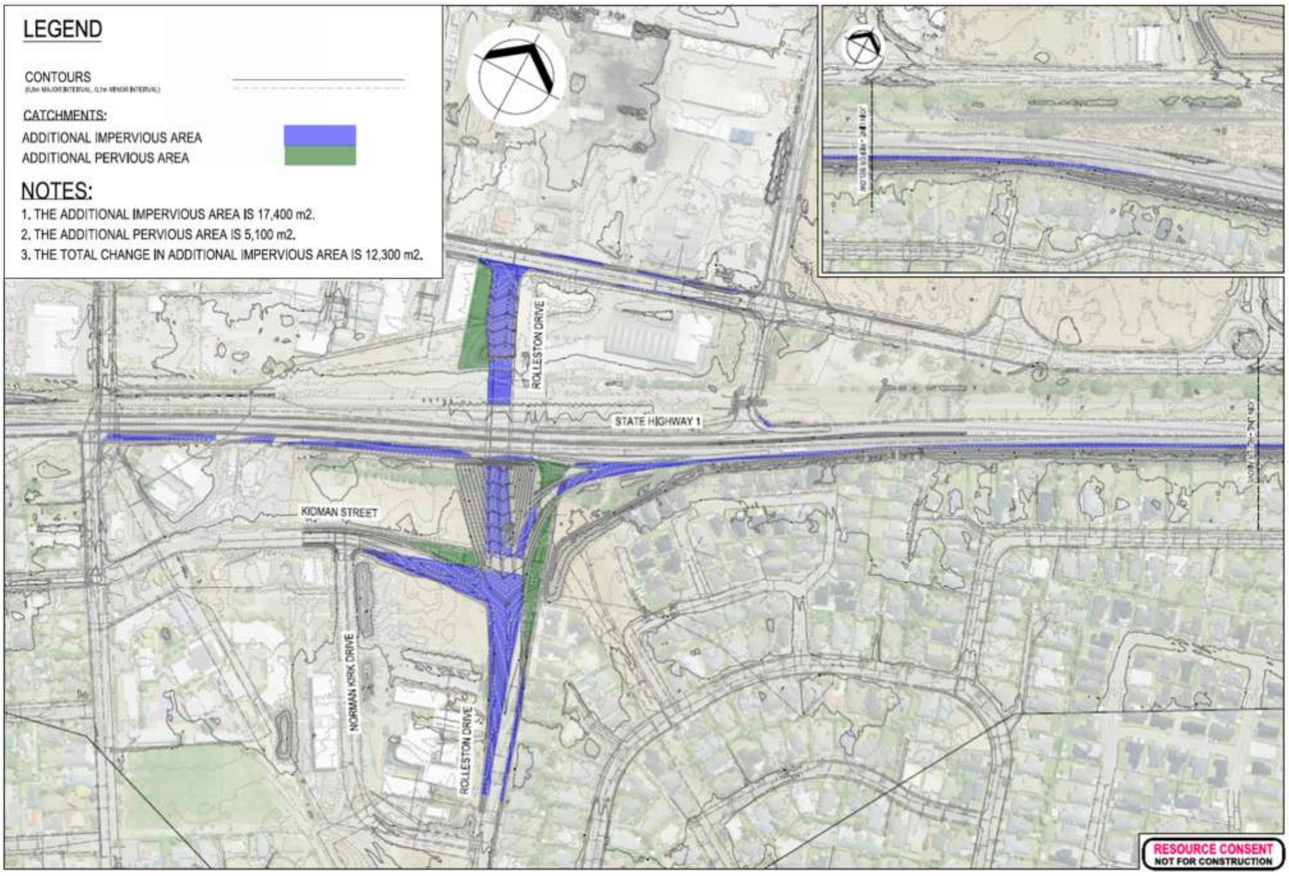

Figure 4-1: Impervious and Pervious Areas Plan for Package 2

Where the new overbridge crosses SH1, this has not been included as an additional impervious area. This is because there is already runoff from the existing impervious footprint (i.e. SH1 underneath) and therefore the overbridge does not increase the runoff.

4.3.2.2 Design Catchments

The Package 2 road corridor catchments are defined according to the geometric design of the road and the surrounding ground levels. Based on this, Figure 4-2 below shows the proposed impervious and pervious areas as a result of the geometric design, forming the road corridor catchments.

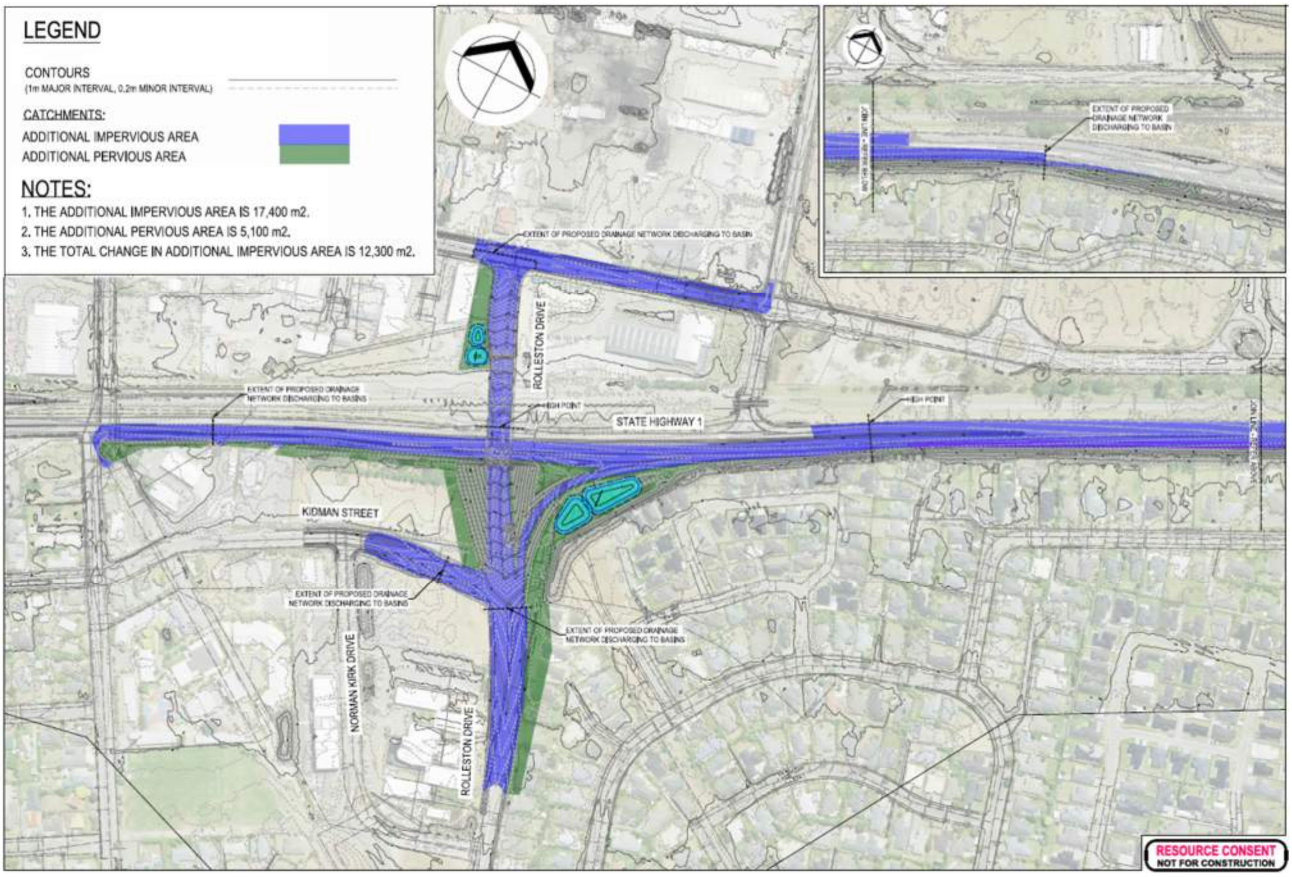

Figure 4-2: Catchment Plan for Package 2

4.3.3 Cross-Drainage

As noted in Section 4.3.1, existing overland flow paths will not be modified by the Package 2 works, and therefore no cross-drainage is proposed.

4.4 Key Design Elements

The key elements of the long-term stormwater management design are:

- Collection and conveyance
- Treatment of first flush
- Discharge to ground
- Attenuation

The following sections of this report break down the approach taken for each of these key design elements.

4.4.1 Collection and Conveyance

The design of the collection and conveyance network for Package 2 includes a number of different elements, including;

- Kerb and channel
- Catchpits
- Manholes
- Swales
- Pipework

Stormwater runoff from Package 2 will generally be conveyed using pipes due to the number of road crossings and space constraints. Swales will be used where space is available or where required to capture runoff from SH1.

All collection and conveyance will be designed using the Rational Method (with the runoff coefficients and rainfalls value described in Section 3.4) to calculate runoff and Manning's equation (with a roughness value as stated in Section 3.4) to calculate conveyance capacity.

4.4.1.1 Kerb and Channel

Channel flow widths within the highway will generally be designed in accordance with P46, which includes:

- During a 10% AEP 10 minute duration event
  - shoulder flows shall not encroach on the trafficable lanes (including shoulder priority lanes)
- During a 1% AEP 10 minute duration event
  - For two lane state highways, ramps and local roads, at least 3m of live traffic lane must be free of stormwater with a maximum depth of 100mm in the covered lane
  - For multi-lane section of state highway, channel flow may cover 1 lane (including priority lanes) at a maximum depth of 100mm
  - No surface runoff from unpaved areas is permitted to flow onto or across a traffic lane surface

Whilst the above design approach has been taken along the proposed alignment for Package 2, this does not mean that there will be no flooding in a 1% AEP. The capacity of the existing stormwater networks adjacent to the Package 2 alignment is unknown, and there may be stormwater which runs onto the Package 2 alignment from the existing roads or adjacent land which is not allowed for in the design. Also, as noted in Section 2.5, there are existing overland flow paths across the alignment.

4.4.1.2 Catchpits

The positioning of catchpits will be carried out to meet the requirements in Section 4.4.1.1 above.

Catchpits will be in accordance with the requirements set out in NZTA P46, i.e. minimum dimensions of 675mm x 450mm x 1200mm deep, with 450mm sump, and have a bypass (or back-entry). The following catchpit capacities will be used:

- 30 L/s for a single in-line catchpit
- 50 L/s for a single valley position catchpit
- 60 L/s for a double in-line catchpit
- 100 L/s for a double valley position catchpit

4.4.1.3 Manholes

All manholes will be designed in accordance with the requirements set out in NZTA P46.

4.4.1.4 Swales

Due to the high permeability gravel beneath the upper soil layer and low groundwater table, swales will generally be grassed as they are not required to tolerate long period of stormwater retention.

Swales do not form part of the formal treatment design but provide conveyance and some pre-treatment. The swales are designed to convey the peak flow 1% AEP event.

The design proposes that full treatment occurs in first flush basins, and therefore the water quality design criteria for swales will not apply. Where a swale is at a grade of less than 2%, an underdrainage system (subsoil drain) is usually required as per NZTA P46. Due to the relatively flat grades of the alignment and existing ground, subsoils would likely be required under all swales, however, due to the relatively high soakage rates observed within the project extents, it is proposed that subsoil drains will not be provided.

4.4.1.5 Pipework

All pipework within the state highway have been designed in accordance with the requirements set out in NZTA P46 and are designed to convey the 1% AEP critical duration event where there is no secondary flow path. Where localised networks service only SDC roads, the local standard (SDC ECOP) has been adopted as not to provide significantly larger network capacity than required.

On this basis, the minimum pipe size is 300 mm diameter for catchpit leads and reticulation not crossing lanes in the state highway and 225mm in SDC roads. The minimum pipe size for reticulation (other than catchpit leads) crossing lanes will be 375 mm diameter for the state highway and 225mm diameter for SDC roads.

Where possible, pipework has been designed to have a minimum velocity of 0.6 m/s at half the 50% AEP (2 year ARI) peak flow. However, as pipes will generally need to be graded with ground surface to prevent basins and soak pits becoming too deep, due to the relatively flat grade of the Canterbury plains it is not possible to meet the minimum velocity requirements for the majority of the network.

4.4.2 Treatment

Stormwater treatment will be provided using the industry standard method of treating the first flush (water quality) to significantly reduce contaminant loads. The first flush concept is based on the contaminant build-up / wash-off process, meaning that the smaller storms, or the first part of the larger storms, have the highest contaminant loads.

For Package 2, first flush basins will provide water quality volume treatment for the major catchments. Where space is restricted and treatment is required, flow-through proprietary devices will be used for first flush treatment (e.g. StormFilters). These will be sized based on the water quality flow rate.

4.4.2.1 First Flush Basins

In first flush basis, treatment will be provided by capturing the first flush volume (i.e. runoff volume from 25mm of rainfall) and treating this via infiltration through designed sand media, before discharging to the ground beneath the basins. Infiltration (biofiltration) treatment provides very good removal efficiencies of total suspended sediment (TSS), metals, and hydrocarbons, and good removal of nutrients.

The major catchment areas discharging to the first flush basins include the overpass, the majority of the south side of state highway 1 between Tennyson Street and the eastern extents of Package 2, and a section of the north side of state highway 1 where the existing network has been impacted by the Package 2 works.

The first flush basins will include a sediment forebay, sized for 15% of the first flush volume. The first flush basins, with exception of the forebays, will be lined with at least 400 mm thick constructed infiltration media. Rather than being designed to achieve a particular infiltration rate, the sand media will be designed to meet the CRC for Water Sensitive Cities, Adoption Guidelines for Stormwater Biofiltration Systems (2015), Appendix C Guidelines for filter media in stormwater biofiltration systems.

Although the initial infiltration rate for the infiltration media will be quite high (e.g. in the order of 300 mm/hr or higher) it will clog over time and the infiltration rate will reduce. The area and depth of the first flush infiltration basins will therefore be sized to allow the first flush volume to drain down at a minimum (long-term) infiltration rate of 20 mm/hr over a maximum of 48 hours, to maintain healthy grass cover.

Once the first flush basin is full, a weir immediately upstream of the first flush basin will divert inflows to the soakage basin.

4.4.2.2 Proprietary Devices

For minor catchments where space is constrained but treatment is required, treatment will be provided by proprietary treatment devices treating the water quality flow rate (i.e. runoff flow rate from 10mm/hr intensity rainfall), before discharging to the ground via a soak pit. Proprietary treatment devices provide good removal efficiencies of total suspended sediment (TSS), metals, hydrocarbons, and nutrients.

Once treated in the proprietary device, stormwater will discharge to ground via a soak pit. When the capacity of the proprietary device is exceeded, over and above the water quality flow, stormwater will bypass the treatment and discharge directly to the soak pit.

Selection of a suitable proprietary device will be based on existing regional approvals. SDC does not maintain an approved list of proprietary devices, but CCC has a draft list, which includes;

- Hynds Up-Flo Filter with CPZ Media
- Stormwater 360 Filterra/Bioscape
- Stormwater 360 Stormfilter with ZPG Media

4.4.3 Discharge to Ground

Infiltration, for the purpose of this report, is where stormwater from the first flush basins infiltrates through a design sand (biofiltration) media to provide treatment. This then discharges to ground beneath the infiltration media. A constructed soakage area may be required, subject to the quality of gravels at the subgrade level of the first flush basin.

Soakage, for the purpose of this report, is where stormwater that discharges to the soakage basins will flow towards an open manhole with a debris screen (scruffy dome), connecting to a network of below ground perforated pipes that discharges stormwater into a soakage area. A constructed soakage area may be required, subject to the quality of gravels at the subgrade level of the soakage basin.

For the Package 2 first flush basins and soakage basins, discharge to ground is expected to be achieved by discharging into the native gravels beneath the basins. Where constructed soak pits are required, this will consist of clean rock, with geotextile over top and sides (but not the base) to prevent migration of the surrounding soil into the soakage areas (but reduce the risk of the base clogging). The clean rock for constructed soakage areas will be selected based on its grading, voids ratio and transmissivity. The geotextile will be selected based on the grading of the surrounding soil.

Factored soakage rates that are used in the design have been determined based on the measured soakage rates observed in field soakage tests. These rates have been factored according to industry standard practice, as described in Section 3.4.6 Ground Soakage Rates of this report.

Discharge to ground will occur at a level sufficiently above predicted groundwater levels, with the unconfined water table observed on site as greater than 10m below ground level at the site.

The discharge of stormwater to ground has the potential to impact on groundwater mounding and wells depending on the volume and proximity of discharge to ground. This is described in more detail in Section 8.3 of this report.

4.4.4 Attenuation

Soakage basins discharging to ground have been sized to provide stormwater storage for buffering between the inflows from stormwater runoff and the discharge to ground. The outflows from the soakage areas have been calculated based on their base areas only. Although the inflows and volumes vary for different event durations, and the outflow varies according to the soakage area and rate, the calculations are relatively straightforward. Sizing of the soakage basins discharging to ground have therefore been carried out using spreadsheet calculations, with the soakage area and storage volume optimised, and a range of durations considered to establish the critical duration for each catchment/basin/soakage system combination. (The critical duration in this context being the duration which results in the largest storage volume.)

In both the overpass north and overpass south catchments, the stormwater network will discharge into a first flush basin, with a weir diversion structure immediately upstream of the first flush basin. The weir will be designed so that once the first flush basin is full, stormwater will begin discharging over the weir and into the soakage basin (which discharges via soakage to ground).

When inflows into the soakage basin exceed the soakage capacity of the ground, water will be stored in the basin and the basin will fill. When the inflow reduces and the soakage rate is greater than the inflow, the soakage basin will start to drain down, discharging to ground via the open manhole with scruffy dome debris screen and subsoil soakage network.

The soakage basin will be sized to provide sufficient storage to balance the inflows and outflows for all durations of events up to the 1% AEP event.

There are small sections of new impervious area at the extremities of the Package 2 footprint that are not able to be conveyed to the basins. This is because the proposed highway vertical alignment and the distance from the proposed basins means that stormwater from these areas is unable to drain to the basins. Stormwater from these areas will be managed in a way that matches the existing network in each catchment, generally with collection via catchpits and soakage to ground, with additional soakage allowance to cater for the increase in impervious areas where appropriate.

5 Stormwater Management by Catchment

5.1 Overview

The Package 2 stormwater catchments are shown in Figure 5-1.

The major catchment areas, listed in Table 5-1, are the catchments collected by the proposed new stormwater management network that discharge to the first flush and soakage basins. These catchments are described in more detail in Section 0.

Table 5-1: Major Catchments Areas

Catchment	Impervious Area (m^2)	Pervious Area (m^2)	Total
Overpass North Catchment	4,000	1,900	**5,900**
Overpass South Catchment	25,800	19,200	**45,000**
Total	**29,800**	**21,100**	**50,900**

The minor catchments are the catchments that are not able to be conveyed to the proposed new stormwater basins because of the geometric design. The additional impervious areas for these catchments are summarised in Table 5-2 and these catchments are described in more detail in Section 5.3.

Table 5-2: Minor Catchment Additional Impervious Areas

Catchment	Additional Impervious Area (m^2)
Western Catchment	580
Eastern Catchment	300
Rolleston Drive South Catchment	-
Kidman Street Catchment	-
Jones Road Catchment	450
Hoskyns Road Catchment	50
Total	**1,380**

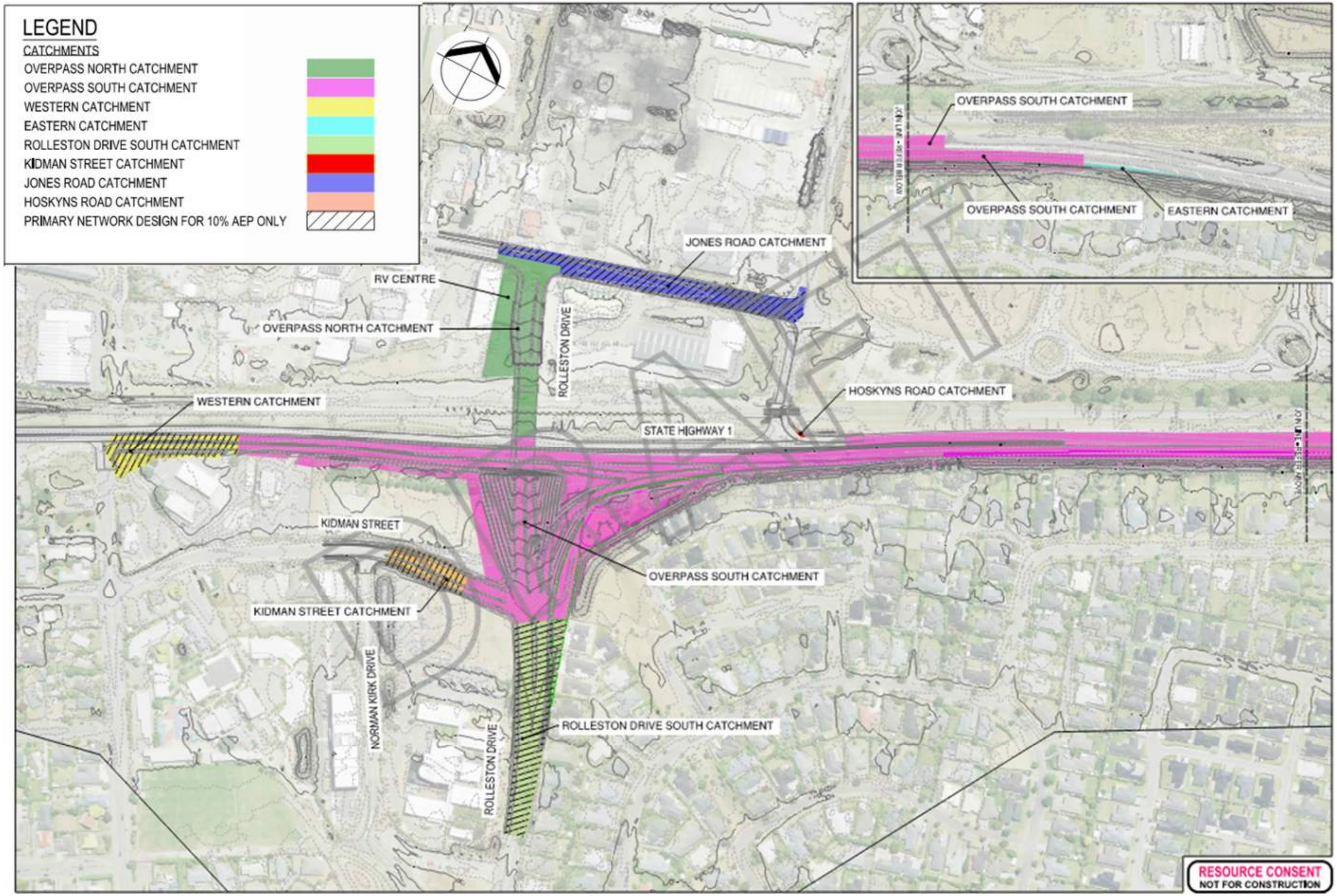

Figure 5-1: Proposed Catchment Plan for Package 2

5.2 Major Catchments

5.2.1 Overpass North Catchment

The Overpass North catchment is delineated by the high point in the new overpass and the junction with Jones Road to the north. A small section of Jones Road will form part of the overpass north catchment based on the geometric design levels.

The removal of the existing Christchurch RV Centre (currently a private commercial site) will provide space adjacent to the overpass abutment on the north-west side for the first flush and soakage basins for stormwater management (Overpass North basins). Stormwater from the overpass north catchment will have first flush treatment (via infiltration in a first flush basin) and runoff up to the 1% AEP event will be conveyed to the soakage basin, attenuated and discharged to ground.

Due to limited space, stormwater will be collected in catchpits and conveyed through a piped network to the first flush and soakage basins.

As noted in Section 2.3 the investigations to date show some existing contaminated land in the vicinity of the proposed basins. Further investigation is required, including proposed TP20 and TP21. The results of this additional testing will inform the management approach, which may include excavation of material and disposal to a facility licensed to accept the concentrations observed.

5.2.2 Overpass South Catchment

The Overpass South catchment is delineated by the high point in the new overpass and the tie into Rolleston Drive on the south side of the traffic light intersection. On Rolleston Drive, the extent of the road runoff catchment that can drain back to the proposed basins is limited due to the relative levels of the proposed basin and catchpit lid levels.

The catchment area includes the south side of SH1, from the BP service station to Beaumont Drive, and the north side of SH1 from east of Hoskyns Road to near the Iport Road roundabout. The extents of the catchment along SH1 have been determined by; the existing geometry of the road and the ability for the stormwater network to service the catchment due to existing levels; and the existing CSM2 catchpits on the both sides of SH1 connecting to the existing swale on the south side of SH1. The proposed Package 2 works remove this existing CSM2 swale on the south side of SH1, along with the current treatment, attenuation and discharge of stormwater for the catchments on both the north and south sides of SH1, and instead divert this catchment to the new Overpass South basins.

The alignment of the new off ramp from SH1 onto Rolleston Drive provides a space for the first flush and soakage basins required for stormwater management south-east of the off ramp, between the off ramp and the existing earth bund (Overpass South basins). Stormwater from the Overpass South catchment will have first flush treatment (via infiltration in a first flush basin) and runoff up to the 1% AEP event will be conveyed to the soakage basin, attenuated, and discharged to ground.

Stormwater will be captured by a mixture of swale and catchpits, and conveyed to the basins by a mixture of swales and piped networks. Swales have been utilised where space allows, with pipe networks where space constraints exist.

5.3 Minor Catchments

5.3.1 Jones Road Catchment

The proposed works on Jones Road include minor adjustment to existing kerb lines and widening of the existing footpath to facilitate a shared footpath / cycleway.

Jones Road contains a number of existing catchpits and soak pits. It is proposed that the existing catchpits are replaced along the new kerb alignment. Soak pits are also likely to be replaced as they are likely to be affected by the proposed works. Additional catchpits and soak pits are proposed to manage the additional runoff from the minor additional impervious area.

Due to the minor increase in impervious area being mainly from the shared footpath / cycleway (i.e. not trafficable areas), no water quality treatment is proposed for this catchment.

The existing catchpits and soak pits are assumed to have been designed to manage the 10% AEP rainfall event (i.e. SDC's primary system standard). On this basis, the replacement of existing catchpits and soak pits and any new network will be designed to manage the runoff from the contributing catchment and discharge it to ground in the 10% AEP event.

5.3.2 Hoskyns Road Catchment

The proposed works on Hoskyns Road include minor adjustment to the existing kerb line on the left turn onto SH1 and the removal of the left turn onto Hoskyns Road from SH1. The removal of the northbound lane on Hoskyns Road from SH1 to Jones Road is likely to result in the removal of impervious area and new pervious surface being created. However, at the time of writing this report this has not been confirmed and the area has been assumed as remaining impervious to cater for the worst-case scenario.

The small additional impervious area when turning left from Hoskyns Road onto SH1 is designed to drain via a kerb breakout, similar to that in the existing road, and into an existing swale that directs runoff to a soak pit. Therefore, the only stormwater works in this area will be the replacement kerb breakout to more effectively manage runoff from the road corridor.

Due to the minor increase in impervious area and the informal treatment provided by the existing swale, no additional water quality treatment is proposed for this catchment.

5.3.3 Western Catchment

The proposed works within the western catchment include widening of SH1 to facilitate the service lane and left turn into Tennyson Street. The extent of this catchment is delineated by the high point in the proposed geometrics alignment along the south side of SH1 and the existing catchpit on the corner of Tennyson Street.

There is also an existing catchpit and soak pit on the state highway which will need to be removed and replaced on the new kerb line.

The existing catchpits and soak pits are assumed to have been designed to manage the 10% AEP rainfall event. On this basis, the replacement of existing catchpits and soak pits and any new network will be designed to manage the runoff from the contributing catchment and discharge it to ground in the 10% AEP event

Due to the minor increase in impervious area and the significant improvements to treatment of road corridor runoff along the adjacent section SH1, no water quality treatment is proposed for this catchment.

5.3.4 Eastern Catchment

The proposed works within the eastern catchment includes minor widening of SH1 to facilitate the additional traffic lane. The extent of this catchment is delineated by the end of the overpass south catchment and the end of the proposed widening.

A short section of existing catchpit and pipe network will be replaced to tie in with the minor road realignment.

Due to the minor increase in impervious area, the treatment provided by the existing swale and the soakage to ground identified on the CSM2 drawings, no additional water quality treatment or attenuation is proposed for this catchment.

5.3.5 Kidman Street Catchment

The proposed works within the Kidman Street catchment include the realignment of Kidman Street to tie into the Rolleston Drive junction. The extent of this catchment is delineated by the existing network at the Norman Kirk Drive junction and the high point on Kidman Street prior to the tie-in at the junction.

There is an existing stormwater network that collects road corridor runoff and discharges to a soak pit within the landscaped area on the junction of Kidman Street and Norman Kirk Drive.

Due to the geometric design and proposed stormwater system for the overpass south catchment, the existing Kidman Street catchment will be notably reduced by the works. Presently, stormwater from the west side of Rolleston Drive, north of Kidman Street, flows south into the current network.

The existing catchpits and soak pits are assumed to have been designed based on a primary network to manage the 10% AEP rainfall event. On this basis, the replacement of existing catchpits and soak pits and any new network will be designed to manage the runoff from the contributing catchment and discharge it to ground in the 10% AEP event. Due to the notable reduction in proposed catchment area for the Kidman Street catchment compared to existing, no water quality treatment is proposed for this catchment.

5.3.6 Rolleston Drive Catchment

The proposed works within the Rolleston Drive catchment include the realignment of Rolleston Drive to tie into the junction. The extent of this catchment is delineated by the tie-in at the junction and the existing stormwater network on Rolleston Drive.

There are several existing catchpits and soak pits on Rolleston Drive that are proposed to be removed and replaced to align with new kerb lines and manage kerb and channel flow widths.

The existing catchpits and soak pits are assumed to have been designed based on a primary network to manage the 10% AEP rainfall event. On this basis, the replacement of existing catchpits, soak pits and any new network will be designed to manage the runoff from the contributing catchment and discharge it to ground in the 10% AEP event. Due to the additional traffic lanes and likely increase in contaminant loads due to vehicle movements including acceleration and braking, proprietary treatment devices are proposed to be incorporated into the design to provide first flush (water quality) treatment, prior to stormwater discharging to ground. The devices will be sized in accordance with the requirements set out in Section 4.4.2.2 of this report.

5.3.7 Existing Rolleston Drive South Catchment

Rolleston Drive (south), at the western extents of Package 2, is not shown on the catchment plan in Figure 4-2. The proposed works at the junction of Rolleston Drive and SH1 includes the removal of the right turn movement into and out of Rolleston Drive by the provision of a median island to prevent the traffic movements. There is no increase in impervious area or changes to the existing stormwater network.

In this location the design will need to manage the existing overland flow paths by maintaining the existing critical levels in the crown of the road to facilitate the overland flow.

5.4 Summary

Overall, there is an additional impervious area of 12,300m² of (refer Section 4.3.2.1) created by the Package 2 works.

The existing stormwater network on the south side of SH1 east of Hoskyns Road (constructed as part of CSM2) currently provides treatment, attenuation and discharge to ground for approximately 10,900m² of SH1. Due to the removal of this existing network to facilitate the Package 2 works, this existing impervious area will be conveyed to the proposed Overpass South basins for treatment, attenuation and discharge to ground, providing mitigation for the removal of existing network.

From Table 5-1 it can be seen that the proposed stormwater basins provide first flush treatment, attenuation and discharge to ground up to the 1% AEP event for 29,800m² of impervious area.

Additionally, the proprietary treatment devices within Rolleston Drive will provide water quality treatment for 2,000m² of road corridor runoff.

6 Construction Stormwater Management

6.1 General

During construction, the principal short-term potential effect of Package 2 will be on water quality, arising from runoff during earthworks.

Earthworks activities will be managed such that proposed erosion and sediment control (E&SC) measures will best practicably minimise erosion, sedimentation and dust generation. The fundamental principles of good E&SC practice for the Canterbury region are;

- Control run-on water entering the site;
- Separate ‘clean’ water from ‘dirty’ water;
- Protect the land surface from erosion;
- Minimise sediment leaving the site.

6.2 Site-Specific Erosion and Sediment Control

As mentioned in Section 2, Package 2 and its vicinity are mostly flat. Run-on to the site is a potential hazard due to existing overland flow paths noted in Section 2.5 and from the adjoining road corridor. The capacity of the current road corridor stormwater system is unknown and might result in run-on the site. There are existing soak pits and catchpits within Package 2 footprint, some of which will be removed and some which will remain.

Soils are generally free draining to the gravel layers below and the groundwater level is sufficiently deep so that natural drainage is likely to occur across most of the site. Dewatering may be required for the installation of the overpass foundations and this would be disposed of in a manner which manages erosion and sediment.

Construction stormwater runoff should be able to be effectively managed within the site. This could include adapting some of the long-term stormwater management system (e.g. swales) for erosion and sediment control purposes (e.g. establishing bunds along the swales), but not the basins. It is important that construction runoff and erosion and sediment devices are kept clear of the proposed first flush basins, soakage basins, and soak pit sites, to protect the underlying ground from clogging.

The DSI identified isolated areas of contaminated soils (see Section 2.3). Management of construction stormwater runoff in these areas should include clean water diversion and avoiding discharge onto or into the contaminated areas. Procedures for management of stormwater runoff in these areas will be included in an Erosion and Sediment Control Plan.

6.3 Erosion and Sediment Control Toolbox and Erosion and Sediment Control Plan

The effects of erosion and sediment will be mitigated in accordance with the Environment Canterbury Erosion and Sediment Control Toolbox.

The Contractor will develop a Package 2 specific Erosion and Sediment Control (E&SC) plan in accordance with the ECan toolbox, whilst suiting the construction programme. That E&SC plan will form part of the Construction Environmental Management Plan (CEMP), which is described in Section 6.6.

Some general principles of the E&SC framework will include:

- Earthworks and any vegetation clearance should, wherever practicable, be limited to the footprint of the works (including the contractor's yard and stockpile areas) to minimise site disturbance. This minimises the site erosion potential.
- Construction works should be staged in manageable sections where practicable, such that progressive reinstatement works can be carried out (i.e. logical work sections should be stabilised before moving on to the next section).
- Unless being actively worked, any exposed areas should be progressively stabilised, generally with topsoil, and grassing. Biodegradable geotextile, mulching, or hydro seeding may be required at some locations, such as protecting steeper slopes.
- Areas of contaminated material (see Section 2.3) should be noted in the plan for so that they can be avoided.
- Any excavated material should be removed from site (if not required for re-use) or stockpiled away from any swales, basins or overland flow paths. Appropriate E&SC measures should be applied to stockpiles, which could include silt fences, bunding, etc around it. Stockpiles should be located adjacent to vegetated areas, where practicable, that will act as vegetative buffers. Practical measures would be taken to divert clean runoff from working areas. Contaminated material should be stockpiled separately from other materials.
- Control of run-off and run-on through the work areas through the use of flow diversions, fluming, silt fences, temporary drains or bunding.
- Dust suppression during dry periods.
- All proposed erosion and sediment control measures are to be installed prior to carrying out physical works (as reasonably practicable) and maintained until satisfactory surface stabilisation is achieved.
- Enable the plan to evolve as the project progresses (e.g. weather, staging, altered drainage, etc. will require changes to the planned E&SC measures whilst achieving its purpose).
- Regularly inspect, monitor and maintain the erosion and sediment control measures. Any rips, tears, and the like to fabrics should be repaired immediately. Any sediment build-up should be cleared and appropriately disposed off site. Maintenance checks are to be carried out at regular intervals and after any significant rainfall event.

6.3.1 Specific Erosion, Sediment and Dust Control Measures

Specific erosion, sediment, and dust control measures which could be utilised, as appropriate as part of the E&SC Plan for the short-term management of the Package 2 works, include:

- Erosion Control Measures: runoff diversion channels and bunds, contour drains, check dams, level spreaders, surface roughening, stabilised construction entrances, and stabilisation techniques such as geosynthetic erosion control systems, and/or revegetation techniques (e.g. topsoiling, seeding, hydroseeding, mulching, and turfing);
- Sediment Control Measures: sediment retention ponds, grit traps, silt fences, super silt fences, inlet protection, decanting earth bunds, catchpit/sediment pits, wheel washdowns, etc;
- Dust Control Measures: watering of exposed areas, and/or stabilisation techniques such as geosynthetic stabilisation, revegetation, hydroseeding, mulching, or turf.

6.4 Monitoring and Maintenance

All temporary treatment devices are to be adequately monitored and maintained throughout the construction phase and the maintenance period until the areas are sufficiently stabilised to allow for the removal or decommissioning of E&SC devices. The Contractor will be required to prepare a maintenance programme together with a contingency plan for instances when the sediment control devices are not operating as required and for assessing sediment control devices after heavy rainfall.

6.5 Construction Environmental Management Plan

For Package 2 construction works, the contractor will be required to develop a Construction Environmental Management Plan (CEMP) prior to commencement of construction activities. The CEMP will include a detailed E&SC plan, with appropriate controls to manage the areas of contaminated soils identified in the Beca DSI. The CEMP will also include a Contaminated Soils Management Plan (CSMP) prepared by a suitably qualified and experienced practitioner (SQEP) for contaminated land, to outline procedures that shall be undertaken during the site development to control disturbance and movement of soils, including the identified contaminated soils within the proposed earthworks areas. The E&SC plan and CSMP will be considered “live” documents and be regularly updated.

The CEMP will also include mitigation measures for failure of the protection works for earthworks, access and haul roads, stockpile sites and work areas, etc, as well as a programme and details for the proposed monitoring and maintenance plan and operational procedures.

The CEMP will include locations and details of E&SC devices (incl expected efficiencies), where the E&SC devices discharge to, and the construction methodology and how this methodology will minimise the amount of sediment being controlled and released. The CEMP will also outline procedures to be undertaken should heavy rain be forecast, in terms of implementing erosion control measures as well as checking that devices are working adequately and monitoring these during and after the event.

7 Operation and Maintenance (O&M)

7.1 Stormwater O&M Manual

A Stormwater Operation and Maintenance Manual will be prepared as part of the detailed design. This will help NZTA, SDC and their contractors to understand the Package 2 stormwater system and its intended operation, and assist them in planning and undertaking effective inspection, monitoring and maintenance works. It will also help the stormwater management system operate as intended, assisting NZTA to fulfil its social and environmental responsibilities.

It is noted that NZTA already maintains significant lengths of state highway and the associated stormwater systems in the Canterbury region and around New Zealand, including SH1 immediately adjacent to the project. This project is a small addition to NZTA's existing operation and maintenance programme.

7.1.1 General Operational Objectives

The O&M Manual will provide a tool for use by the maintenance operators, SDC and NZTA, to monitor, plan and commission works associated with maintenance of the Package 2 stormwater management system.

The O&M Manual should provide guidance on the operational objectives and methods that relate to the monitoring and maintenance actions, including regular and reactive maintenance activities and procedures, including emergency spill incidents, and includes a list of important contacts. The monitoring and maintenance activities will manage the effects of the stormwater system on the local and downstream environments will be best practicably avoided, remedied or mitigated.

7.1.2 Stormwater Collection and Conveyance Systems

The principal operational objectives of the stormwater collection and conveyance system are:

- Conveyance to prevent surface flooding: System needs to be cleared of blockages and debris so stormwater from the road surface is safely conveyed away.
- Provide stability and safety of conveyance systems: Maintain grass cover/vegetation and inspect for erosion and repair as required. The hydraulic action of flowing water in the system can cause erosion or structural damage to infrastructure, in particular open channels, swales, outlets and culverts.
- Provide pre-treatment of stormwater: Maintain grass cover/planting and maintain any flow control devices (e.g. weirs) designed to direct the flows to provide suitable water quality treatment.

7.1.3 Stormwater Treatment Devices and Attenuation and Soakage Devices

The principal operational objectives of the stormwater treatment, attenuation and disposal devices are:

- First flush basins to provide water quality treatment via sediment and rubbish removal, filtration or settlement and biological process: Maintain grass cover/planting and maintain any inlet/outlet structures (e.g. headwalls) designed to direct the flows to provide suitable water quality treatment.
- First flush proprietary treatment devices to provide water quality treatment via efficient removal of sediments, oil and grease, metals, organics, and nutrients.
- Soakage basins to provide buffer storage of stormwater: Maintain grass cover/planting and maintain any inlet/outlet structures (e.g. headwalls) designed to direct the flows to provide suitable attenuation.
- Disposal devices to facilitate stormwater discharge to ground: Maintain any inlet/outlet structures (e.g. open manhole with scruffy dome) designed to direct stormwater discharge to ground.

All the above devices are designed and maintained to protect the receiving environment from erosion, flooding and contamination.

7.1.4 Landscape

The general objective of landscape maintenance as it relates to the stormwater management devices is to maintain appropriate planting and grass in a sound and healthy condition in a way that does not adversely impact upon flow conveyance, treatment characteristics or aesthetic values.

The overall landscape planting strategy aims to reinstate and enhance the underlying landscape elements, patterns, and processes using endemic native species.

The proposed stormwater management devices will be grassed, which will be mown, and planting will be used around the extremities of the devices which will naturalise the form of the basins and swales adding ecological and amenity value to the devices and adjacent spaces. By introducing areas of native planting to these devices ongoing mowing maintenance efforts will be reduced.

Throughout the establishment phase weed species will be managed as part of the landscape maintenance programme to increase levels of canopy closure in turn reducing the need for ongoing maintenance.

7.1.5 Implementation

The O&M Manual should include figures and drawings showing the system features, a description of how the system functions, and a schedule of recommended O&M activities. Appointed operators will need to be skilled and take responsibility for undertaking scheduled activities to an acceptable industry standard.

Maintenance works at specific stormwater management devices should be covered in checklists (i.e. swales, basins, etc.) in the O&M Manual. Such forms should be completed and submitted with the regular summary report of the maintenance activities.

7.2 Design Considerations for O&M

The stormwater management Preliminary Design includes consideration of convenient operational and maintenance features as follows:

- Swales: Ease of access for inspection, mowing (grass) or weeding (plants) and maintenance. Geometric design considerate of mowing and maintenance requirements (where appropriate).
- Basins: Treatment device features such as inlets, underdrainage and outlets (eg to soakage) will be readily accessible and incorporate visible features (e.g. observation wells) meaning that the site inspection and performance monitoring can be undertaken with ease.

Maintenance access to swales and basins has not been shown on the stormwater design drawings, however there is sufficient space to allow for access within the NOR footprint. The maintenance access design will need to be carried out at detailed design.

7.3 O&M Manual Review

A regular review of the O&M Manual should include:

- Updating the maintenance schedule where action and frequency refinements can be made based on the previous monitoring and maintenance report findings
- Review and update the regular and reactive monitoring and maintenance procedures for the stormwater collection and conveyance system and for the stormwater treatment devices based on the previous year’s report findings
- Review and update the regular and reactive monitoring and maintenance procedures for landscape features associated with the stormwater management system, including general elements, swales, basins, riparian margins, etc.
- Review and update the emergency spill procedures, based on any ‘incident’ experiences.
- Review and update the monitoring and maintenance contacts list
- Review and refinement of the budget estimates, based on the cost of monitoring and maintenance works incurred in the previous year.

8 Assessment of Effects

Sections 8.1 to 8.4 below address the operational effects, while Section 8.5 addresses the construction-related effects.

8.1 Effects on Water Quantity

8.1.1 Existing Overland Flow Paths and Effects on Flood Risk

8.1.1.1 Potential Effects

There are existing flow paths across State Highway 1 in the Package 2 footprint, as shown by SDC's flood modelling. The upgraded alignment crosses these flow paths. This has the potential to affect existing flow paths, increasing flood levels, or divert the flow paths, changing the flooded area.

8.1.1.2 Proposed Mitigation

The geometric design manages the improvements to the road corridor network by maintaining the existing critical levels within the Package 2 extents and retains the existing overland flow paths.

Due to the existing overland flow paths being managed as described above, no cross-drainage infrastructure is proposed and the flood risk effects for Package 2 will be less than minor.

8.1.2 Road Runoff and Effects on Flood Risk

8.1.2.1 Potential Effects

The additional impervious areas constructed as part of the Project will increase stormwater runoff. If not managed appropriately, this has the potential to:

- Increase peak discharges to surface water, potentially causing erosion and increasing flood levels.
- Increase the volume of water discharged to surface water during the critical duration event for the catchment, increasing flood levels.

8.1.2.2 Proposed Mitigation

During operation, effects on water quantity will be managed through attenuation and soakage to ground. The proposed Package 2 stormwater system includes soakage basins and soak pits with discharge to ground (via soakage), mitigating the water quantity effects. Stormwater runoff from at least an area equivalent to the additional impervious area within Package 2 will be discharged to ground, up to the 1% AEP event.

Where practicable, stormwater runoff will be collected and conveyed to stormwater basins or soak pits. Stormwater attenuation will be provided in the soakage basins (and in some locations, soak pits) which discharge to the ground below the basins (or soak pits). Attenuation storage in the basins (or soak pits) provides a buffer between the inflow of runoff into the basins (or soak pits) and the discharge to ground via soakage.

The basins and swales will be located and designed so that during events larger than the 1% AEP design event, stormwater will follow the existing overland flow paths.

There are small sections of new impervious area at the extremities of the Package 2 footprint that are not able to be conveyed to the basins. This is because the proposed highway vertical alignment and the distance from the proposed basins means that stormwater from these areas is unable to drain to the basins. Stormwater from these areas will be managed in a way that matches the existing network in each catchment, generally with collection via catchpits and soakage to ground, with additional soakage allowance to cater for the increase in impervious areas where appropriate.

Due to the proposed stormwater management system described above, the water quantity and flood risk effects for the Package 2 will be less than minor.

8.2 Effects on Water Quality

8.2.1.1 Potential Effects

The Package 2 improvements may result in additional contaminant loads from vehicular traffic, littering, pavement and landscape activities. The main contaminants will be gross pollutants (litter), suspended sediments, heavy metals (including zinc and copper from vehicle wear), hydrocarbons, and to a lesser extent nutrients. If not managed there is the potential for additional contaminant load to have a negative effect on the receiving environment (i.e. the local groundwater).

8.2.1.2 Proposed Mitigation

For the major catchments, during operation, effects on water quality will be mitigated through capture of first flush runoff and treatment via infiltration (also called biofiltration) through a designed sand media for the. The proposed Package 2 stormwater system includes first flush basins for infiltration treatment of first flush stormwater, before soakage to ground, to mitigate these effects.

As noted above, there are small sections of new impervious area at the extremities of the Package 2 footprint that are not able to be conveyed to the basins. This is because the proposed highway vertical alignment and the distance from the proposed basins means that stormwater from these areas is unable to drain to the basins. Stormwater from these areas will be managed in a way that matches the existing network in each catchment, generally with collection via catchpits and soakage to ground, with additional soakage allowance to cater for the increase in impervious areas where appropriate. Generally, no treatment will be provided for these catchments, with the exception of the Rolleston Drive catchment, where effects on water quality will be mitigated through treatment of the first flush flow rate via a proprietary treatment device.

Swales have been provided where space allows. While the swales have been designed for conveyance purposes, and are not part of the formal treatment design, they will provide some pre-treatment before the first flush basin (i.e. treatment train).

The proposed stormwater design for the Package 2 will include first flush treatment of a larger area than the increase in impervious area across the Package 2 footprint. As such, the water quality effects for the Package 2 will be less than minor.

8.3 Effects on Groundwater Levels

8.3.1.1 Potential Effects

As the stormwater system for Package 2 discharges stormwater to ground at basins and soak pits, this has the potential to locally increase groundwater levels and affect nearby groundwater users.

8.3.1.2 Proposed Mitigation

As part of the site investigations, the local water table has been identified at depths greater than 10m below ground level at the Project (Package 2) site. Soakage from basins is therefore expected to be subject to at least 7m vertical unsaturated flow through the vadose zone from the soakage basins (assuming basin depth of less than 3m from existing ground level).

The discharge of stormwater to ground via soakage will locally raise the water table, however the mounding at the water table is likely to be delayed, and of lesser magnitude than the peak flooding event at the basin, due to the soakage through the unsaturated zone.

Currently the local groundwater is recharged by soakage from rainfall events. The proposed basins will not increase the total volume of water infiltrated to the water table but will redirect it to infiltrate over a smaller area (i.e. at the basins and soak pits). This will locally raise the water table directly under the basins and soak pits, and then as pressures equalise, result in a groundwater level change comparable to current groundwater level response to rainfall events, i.e. within the seasonal range.

As such, effects on groundwater levels due to the proposed soakage basins are expected to be localised, and short term, with no long-term effects on groundwater levels in the area. No effects on water availability are anticipated at any nearby bores due to the net neutral change in groundwater recharge due to the stormwater being discharged to ground.

8.4 Effects on Groundwater Quality

8.4.1.1 Potential Effects

As the stormwater system for Package 2 discharges stormwater to ground at basins and soak pits, this has the potential to impact groundwater quality and affect nearby groundwater users.

The soakage basins in the overpass north catchment are also located within an area where some elevated levels of contaminants have been detected in the shallow fill (see Section 2.3), and there is potential for increased mobilisation of these contaminants to groundwater due to the proposed discharge.

8.4.1.2 Proposed Mitigation

The proposed first flush and soakage basins for Package 2 do not fall within any existing Community Drinking Water Protection Zones (CDWPZ), as currently indicated on CanterburyMaps. There is a recently installed SDC public supply bore BX23/0918 with screen greater than 180m depth located nearby which does not yet have a water take consent and therefore does not have a CDWPZ. A future CDWPZ for this bore, likely to be a 100m radius circle as per Schedule 1 of the LWRP, would include an area of the basins servicing the overpass south catchment (see Section 2.4). However, SDC has advised that it is not going to use this bore (email received 25 October 2024).

There are four domestic supply bores listed in the ECan wells database located downgradient of the eastern extent of the package 2 site, near Haymakers Crescent (BX23/1598, BX23/1129, BX23/1130 & BX23/1047, see Figure 2-3-4). The actual status and current use of these bores is not confirmed. The proposed stormwater management upgradient of these bores is described in Section 5.3.4. There is no proposed increase in soakage, or changes to the existing treatment measures in this area, and therefore a change in groundwater quality due to this project are not expected at these bores. Additionally, there is an 8m vadose zone, and greater than 35m of vertical transport through the aquifer between the project area and the bore intakes, which will provide treatment and attenuation of any highway related contaminants.

These domestic bores are located 1.5km across gradient from the proposed stormwater treatment and soakage basins at the overpass, and are not expected to be adversely affected by the works.

Where swales or basins are located in areas with elevated levels of contaminants in the soil, the stormwater basins will be constructed with mitigation measures to prevent the potential mobilisation of contaminants due to the additional soakage. Mitigation measures may include removal of the contaminated material and/or lining the sidewalls of the infiltration devices with, for example, an appropriate depth clean cover or an impermeable liner to mitigate the risk of contaminant mobilisation from adjacent soils.

Implementation of these mitigation measures would prevent potential mobilisation of contaminants in soil and migration into groundwater within the swales or basins. As such, the groundwater quality effects for the Package 2 will be less than minor.

8.5 Construction Effects

8.5.1.1 Potential Effects

During construction, there is the potential for adverse effects to arise as a result in discharges of sediment from earthworks during road construction, and from construction of pipes, culverts, swales and stormwater basins.

Earthworks activities will be managed in accordance with the E&SC plan and CSMP plan. The CSMP will identify areas of contaminated land and how to manage earthworks and temporary stormwater management in the vicinity of the contaminated land. The proposed erosion and sediment control (E&SC) measures will best practicably minimise erosion, sedimentation and dust generation. The fundamental principles of good E&SC practice for the Canterbury region are:

- Control run on water
- Separate 'clean' water from 'dirty' water
- Protect the land surface from erosion
- Minimise sediment leaving the site

The Project footprint and surrounding area is relatively flat and low lying. Run-on to site is unlikely to be an issue during lower intensity rainfall events due to existing stormwater infrastructure. However, during more intense periods of rainfall there is the potential for the existing stormwater infrastructure to become exacerbated and for cross-catchment flows to discharge onto State Highway 1, resulting in some areas of the Project footprint becoming inundated.

Soils are generally free draining to the gravel layers below and the groundwater level is sufficiently deep so that natural drainage is likely to occur across most of the site. Dewatering is not expected to be required for the Project, however, if required, this would be disposed of in a manner which manages erosion and sediment.

Construction runoff will be able to be effectively managed within the Project footprint, avoiding areas of contaminated land. This could include adapting a long-term stormwater management system (e.g. swales) for erosion and sediment control purposes (e.g. establishing bunds along the swales), but not the basins. It is important that construction runoff and erosion and sediment devices are kept clear of the proposed first flush basins, soakage basins, and soak pit sites, to protect the underlying ground from clogging.

8.5.1.2 Proposed Mitigation

During construction, movement of soils and the potential discharge of sediment during construction will be mitigated via the preparation and implementation of a CSMP and Erosion and Sediment Control Plan, including maintenance of the erosion and sediment control (E&SC) measures such as:

- Construction staging to limit stripped/open areas and stabilising surfaces as soon as possible
- Diversion of run-on water
- Silt fences
- Decanting earth bunds
- Sediment ponds
- Construction soakage basins – not in locations where long-term soakage is proposed.

9 Conclusions

The proposed State Highway 1 Rolleston Access Improvements, Package 2 stormwater management, mitigates the operational stormwater effects of the proposed works by providing stormwater treatment, attenuation and discharge to ground. Where practicable, stormwater will be conveyed to basins which provide first flush treatment, attenuation and discharge to ground up to the 1% AEP event.

Small areas at the extremities of Package 2 unable to drain to new basins due to the geometry of the road corridor and will drain to new or existing catchpits and soak pits, designed to manage the road corridor runoff in those catchments. Runoff from these areas will generally be managed in the same way as the existing runoff, with no treatment being provided and discharge to ground. The exception to this is Rolleston Drive, where proprietary treatment devices are proposed to mitigate the additional contaminant load expected due to the new traffic light junction and vehicle movements.

The existing catchpits and soak pits are assumed to have been designed to manage the 10% AEP rainfall event (i.e. SDC’s primary system standard). On this basis, the replacement of existing catchpits and soak pits and any new network for these minor catchments will be designed to manage the runoff from the contributing catchment and discharge it to ground in the 10% AEP event.

Whilst some minor catchments will be managed as existing, these areas are relatively small, and overall, the stormwater system will manage, treat and discharge to ground runoff from a larger impervious area than the additional impervious area created by the Package 2 works.

The geometric design will generally maintain the existing levels within Package 2. Where proposed raised centre islands would block existing overland flow paths, then options to mitigate changes to overland flow paths will be considered and confirmed during detailed design (e.g. cut-downs to allow flow through the island, or changing the extents of the islands). Therefore no cross-drainage infrastructure is proposed and the flood risk effects for Package 2 will be less than minor.

During construction, potential erosion and discharge of sediment will be mitigated by the preparation and implementation of an Erosion and Sediment Control Plan.

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A

Appendix A – RM Approval Drawings

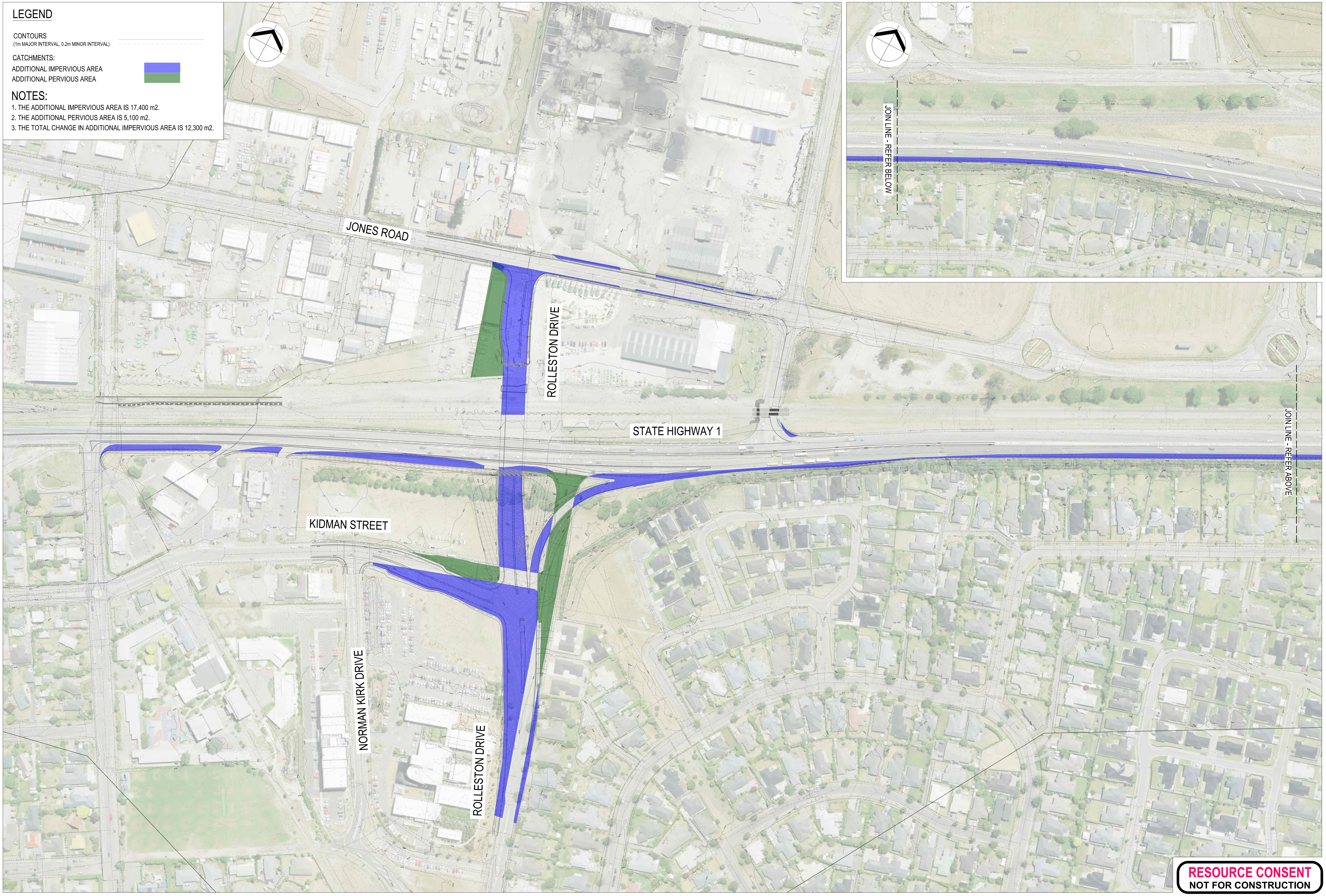
LEGEND

CONTOURS
(1m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

CATCHMENTS:
ADDITIONAL IMPERVIOUS AREA
ADDITIONAL PERVIOUS AREA

NOTES:

1. THE ADDITIONAL IMPERVIOUS AREA IS 17,400 m2.
2. THE ADDITIONAL PERVIOUS AREA IS 5,100 m2.
3. THE TOTAL CHANGE IN ADDITIONAL IMPERVIOUS AREA IS 12,300 m2.



RESOURCE CONSENT
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SH	DA	12.11.24

Original Scale (A1) 1:1500	Design	G.HAYDEN	19.06.24	Approved For Construction*
	Drawn	G.HAYDEN	19.06.24	
Reduced Scale (A3) 1:3000	Dsg Verifier	K.PURTON	12.11.24	
	Drg Check	S.HARVEY	12.11.24	Date
	* Refer to Revision 1 for Original Signature			

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER ADDITIONAL IMPERVIOUS AND PERVIOUS AREAS PLAN

Discipline: CIVIL - DRAINAGE

Drawing No.: 3338703-20-CD-2010

Rev.: A

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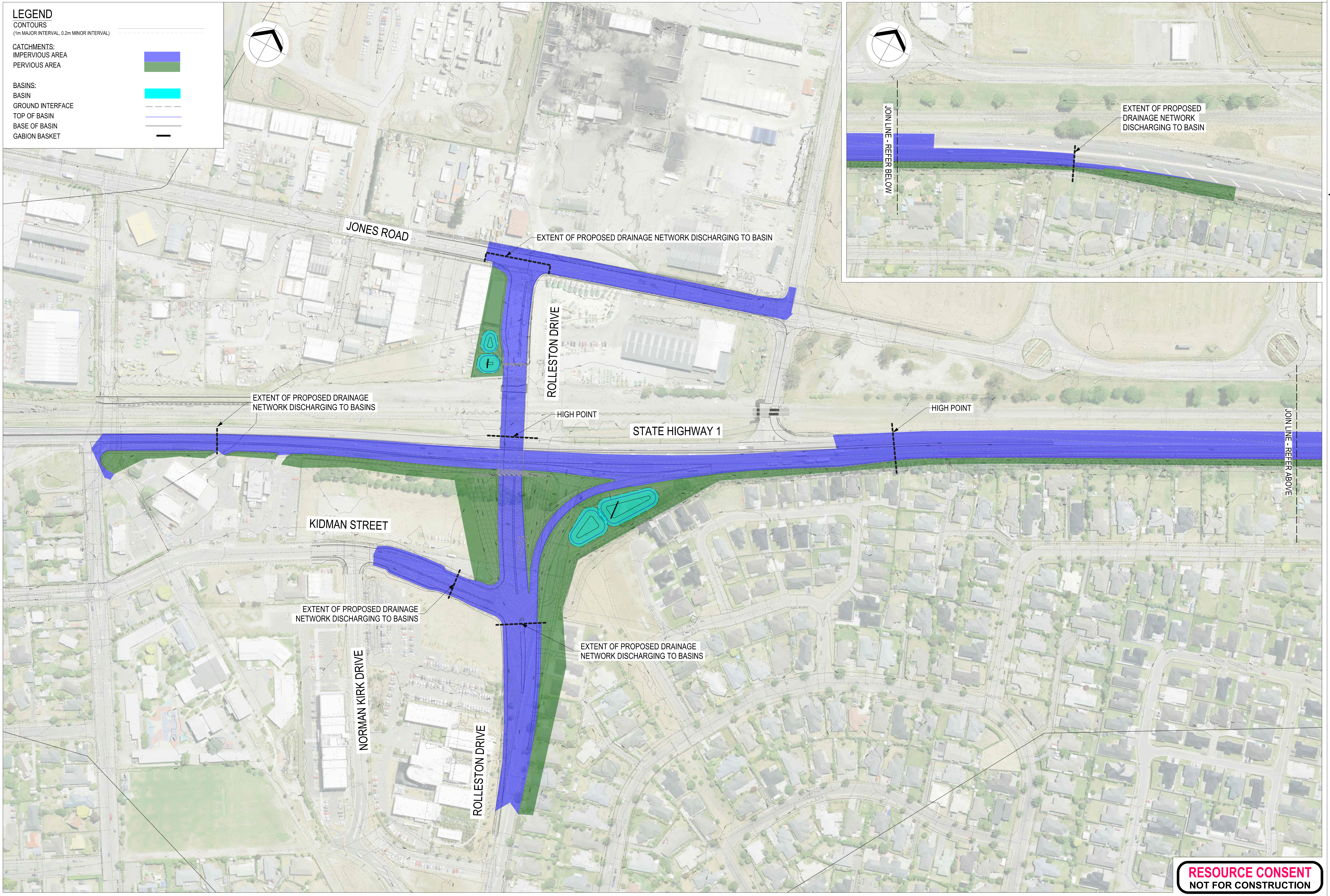
Document No. ...\Plans\3338703-20-CD-2010

Drawing Plotted: 12/11/2024 2:04:40 pm



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No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SH	DA	12.11.24

Original Scale (A1) 1:1000	Design	G.HAYDEN	19.06.24	Approved For Construction*
Reduced Scale (A3) 1:2000	Drawn	G.HAYDEN	19.06.24	
	Dsg Verifier	K.PURTON	12.11.24	
	Drg Check	S.HARVEY	12.11.24	Date

* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER IMPERVIOUS AND PERVIOUS AREAS PLAN

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2012 Rev. A

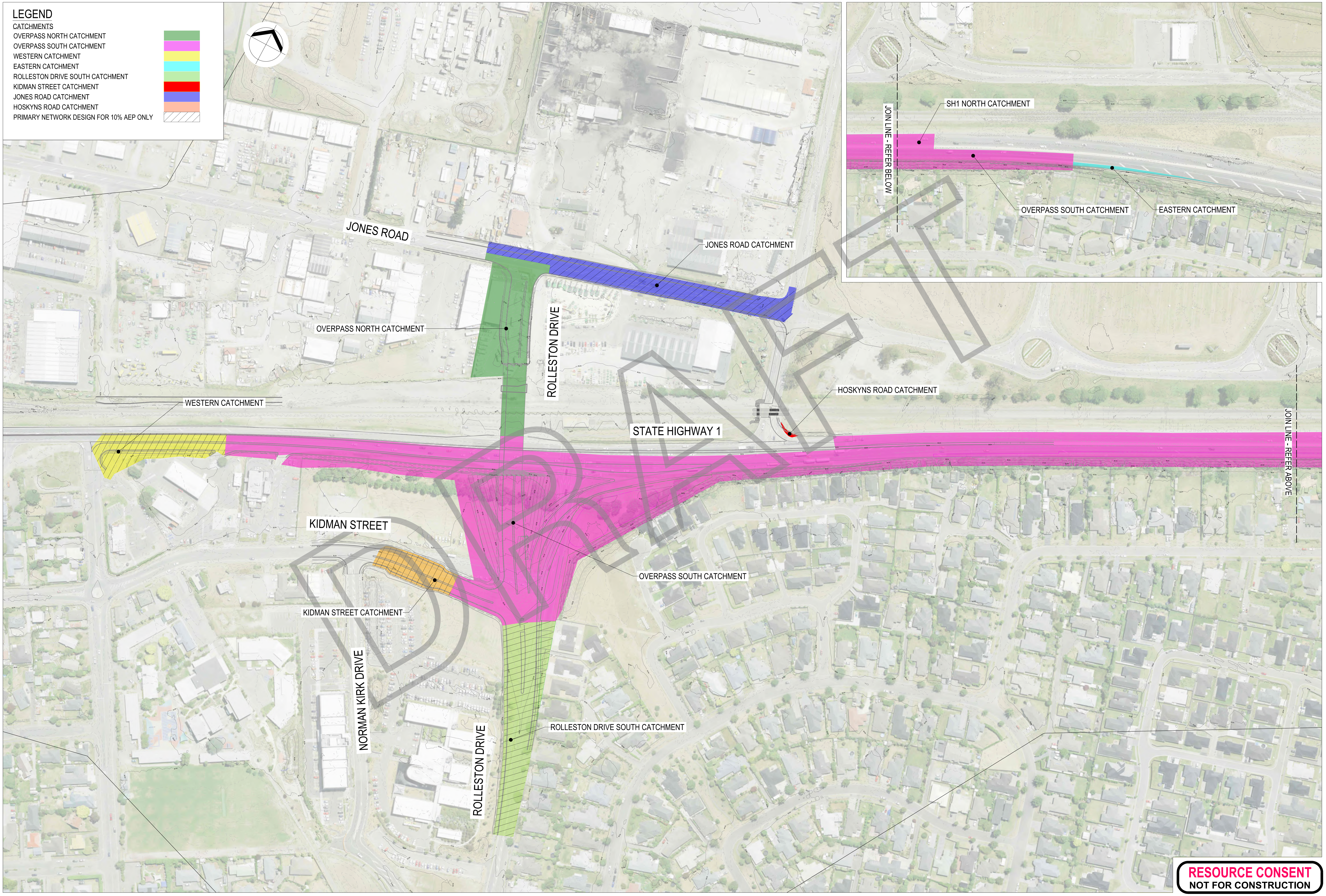
Document No. ...\Plans\3338703-20-CD-2012

DO NOT SCALE FOR SET OUT DIMENSIONS

DO NOT SCALE

IF IN DOUBT ASK.

Drawing Ploted: 11/11/2024 5:07:13 pm



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RESOURCE CONSENT
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	JB	SH	DA	12.11.24

Original Scale (A1) 1:1500
Reduced Scale (A3) 1:3000

Design	G.HAYDEN	19.06.24
Drawn	J.BRYAN	18.09.24
Dsg Verifier	K.PURTON	12.11.24
Drg Check	S.HARVEY	12.11.24

Approved For Construction*
Date
* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER CATCHMENT PLAN

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2014

Rev. A

Document No. ...\Plans\3338703-20-CD-2014

DO NOT SCALE FOR SET OUT DIMENSIONS

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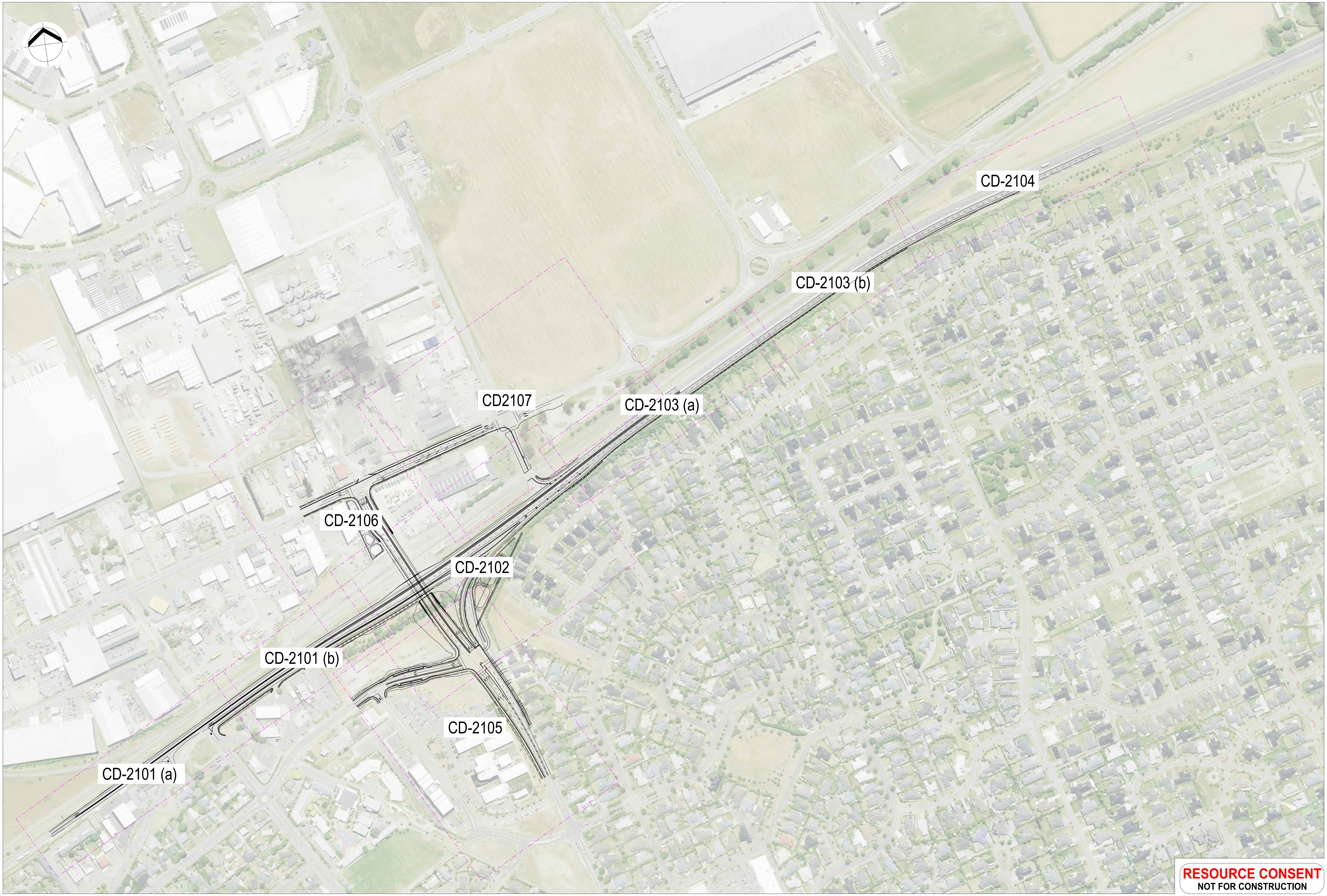
Drawing Ploted: 13/11/2024 4:39:21 pm



RESOURCE CONSENT
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No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENTS	JAB	SPH	DGA	12.11.24

Original Scale (A1) 1:2000	Design	G.HAYDEN	15.10.24	Approved For Construction*
	Drawn	J.BRYAN	16.10.24	
Reduced Scale (A3) 1:4000	Dsg Verifier	K.PURTON	12.11.24	
	Drg Check	S.HARVEY	12.11.24	Date

* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: SHEET LAYOUT

Discipline: CIVIL ENGINEERING

Drawing No. 3338703-20-CD-2100

Rev. A

Document No. ...\Plans\3338703-20-CD-2100

DO NOT SCALE FOR SET OUT DIMENSIONS

DO NOT SCALE

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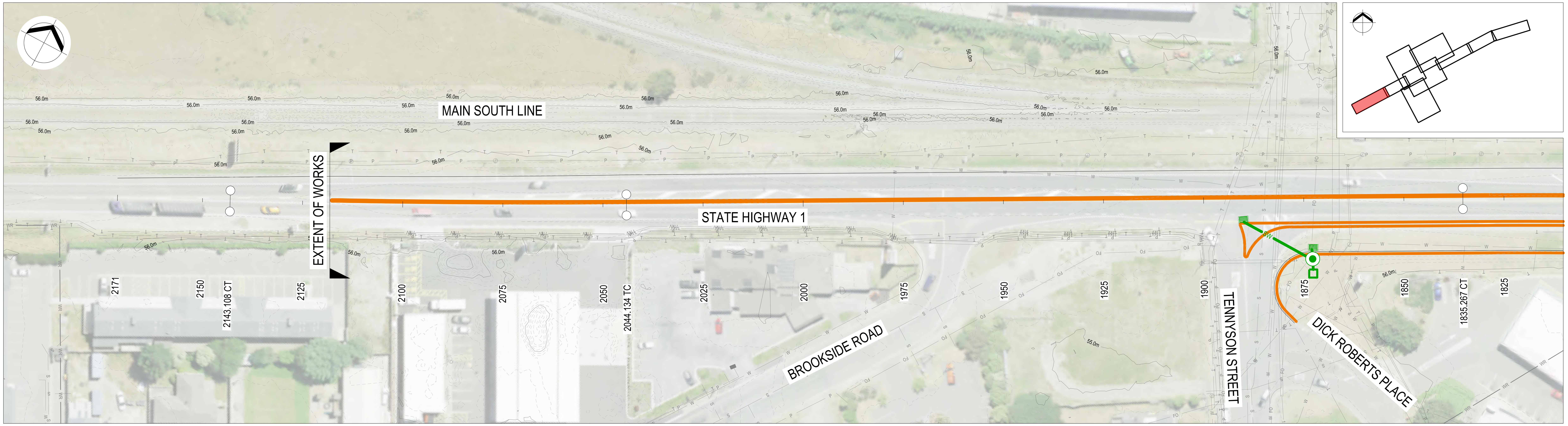
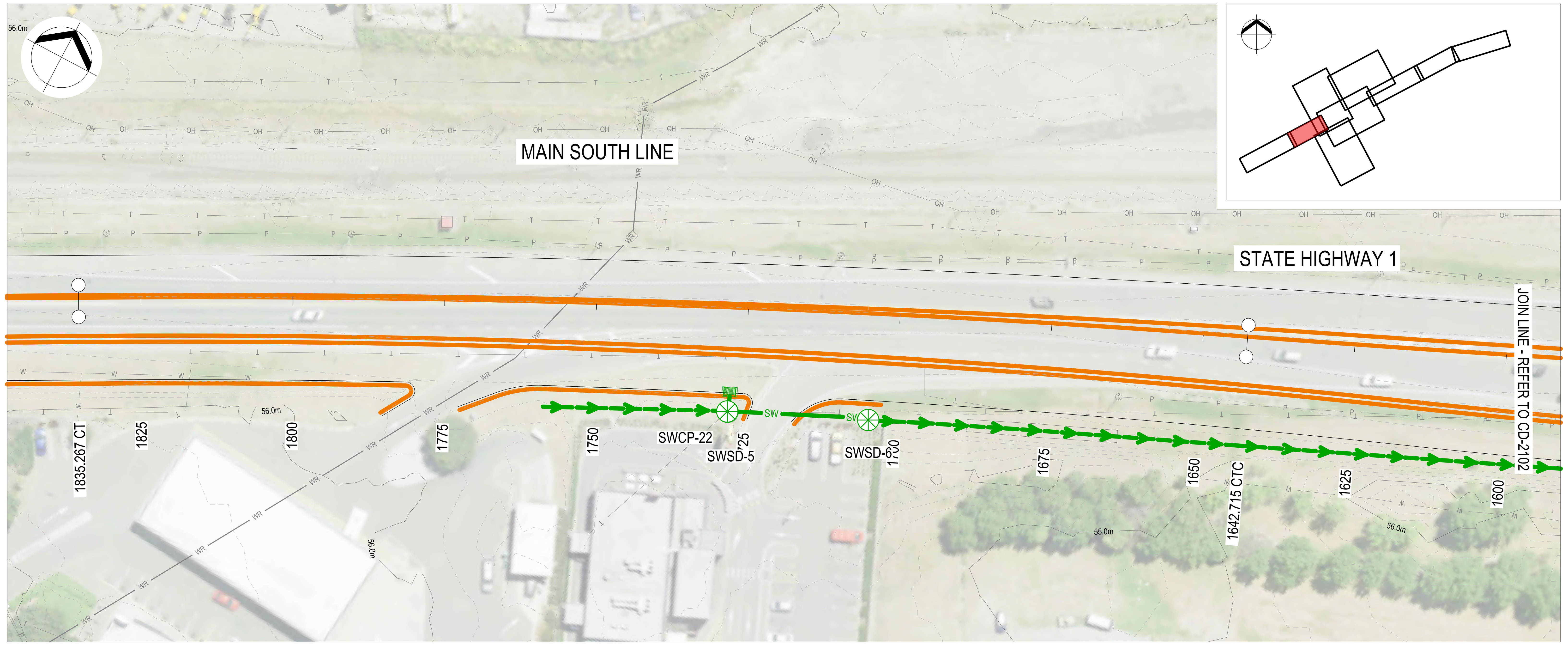
Drawing Ploted: 12/11/2024 5:15:54 pm





LEGEND

CONTOURS
(1.0m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

EXISTING SERVICES
- POWER (11kV)
- POWER (OVERHEAD)
- FIBRE OPTIC
- CHORUS
- WATER MAIN
- STORMWATER
- SEWER
- STOCK WATER RACE
- STORMWATER MANHOLE
- SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT

PROPOSED DESIGN
- STORMWATER MANHOLE
- STORMWATER SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT
- HEADWALL
- KERB BREAKOUT
- ACO DRAIN
- STORMWATER PIPE
- SWALE
- SWALE AND PIPE
- KERB
- EDGE OF SEAL
- GROUND INTERFACE OF BASIN
- TOP OF BASIN
- BASE OF BASIN
- SOAKAGE EXTENT
- SUBSOIL

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No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1) 1:500	Design	G.HAYDEN	19.06.24
	Drawn	G.HAYDEN	19.06.24
Reduced Scale (A3) 1:1000	Dsg Verifier	K.PURTON	12.11.24
	Drg Check	S.HARVEY	12.11.24

Approved For Construction* Date
* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER GENERAL ARRANGEMENT SHEET 1 OF 7

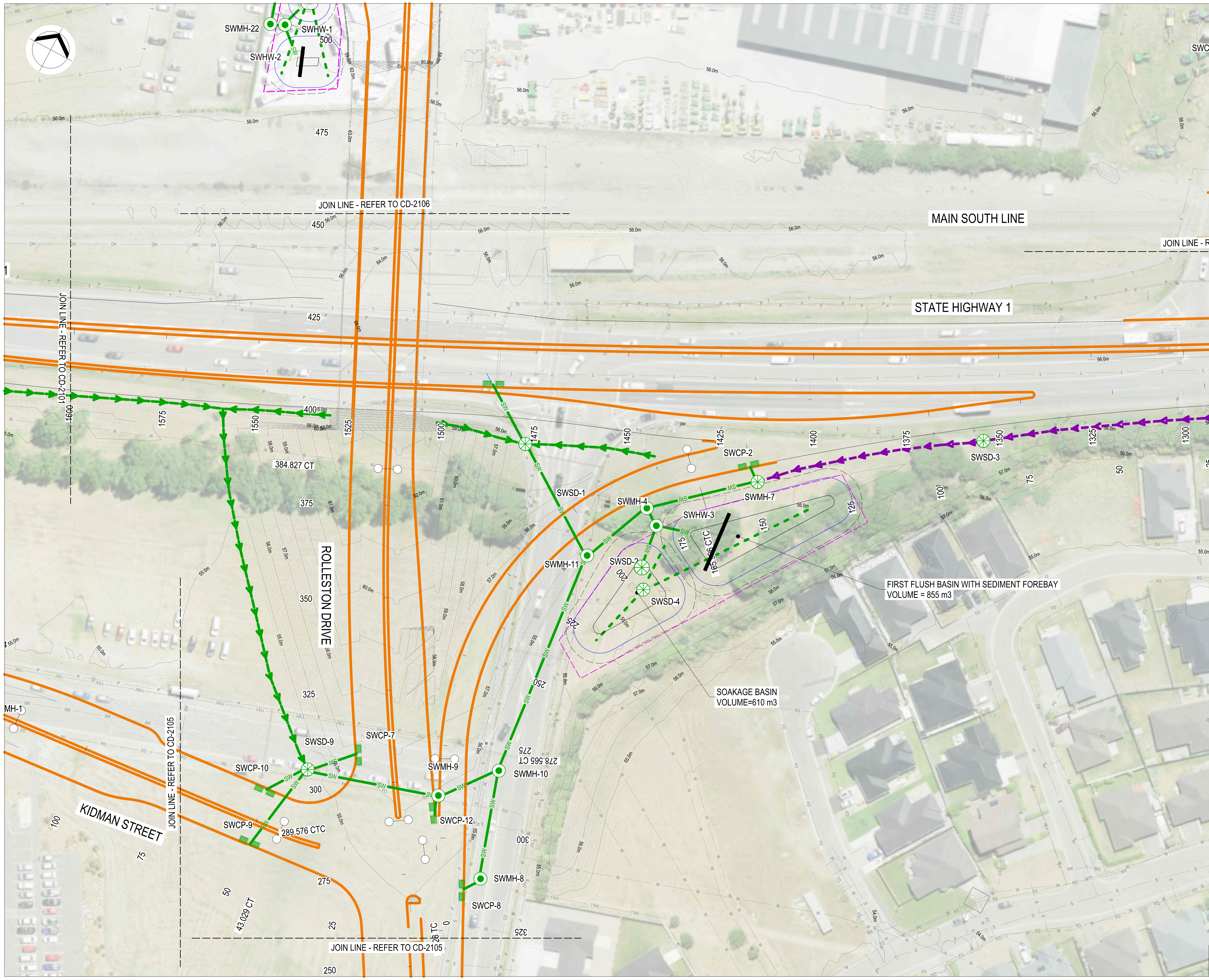
Discipline: CIVIL - DRAINAGE

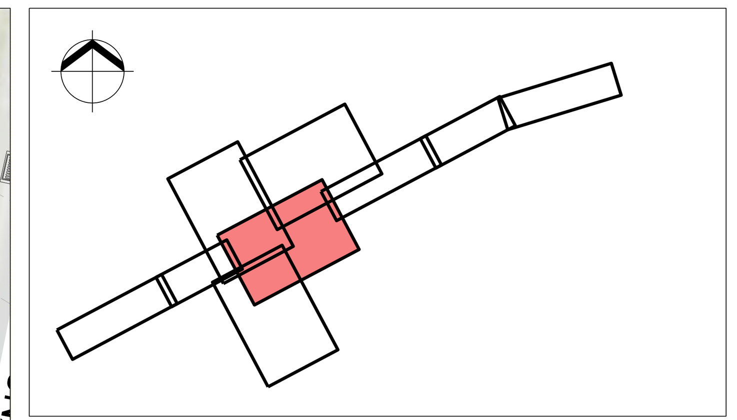
Drawing No. 3338703-20-CD-2101 Rev. A

Document No. ...\Plans\3338703-20-CD-2101

DO NOT SCALE FOR SET OUT DIMENSIONS DO NOT SCALE IF IN DOUBT ASK.

Drawing Plotted: 12/11/2024 5:14:04 pm





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LEGEND

CONTOURS
(1.0m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

EXISTING SERVICES
- POWER (11kV)
- POWER (OVERHEAD)
- FIBRE OPTIC
- CHORUS
- WATER MAIN
- STORMWATER
- SEWER
- STOCK WATER RACE
- STORMWATER MANHOLE
- SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT

PROPOSED DESIGN
- STORMWATER MANHOLE
- STORMWATER SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT
- HEADWALL
- KERB BREAKOUT
- ACO DRAIN
- STORMWATER PIPE
- SWALE
- SWALE AND PIPE
- KERB
- EDGE OF SEAL
- GROUND INTERFACE
- TOP OF BASIN
- BASE OF BASIN
- SOAKAGE EXTENT
- SUBSOIL

RESOURCE CONSENT
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1) 1:500	Design	G.HAYDEN	19.06.24
	Drawn	G.HAYDEN	19.06.24
Reduced Scale (A3) 1:1000	Dsg Verifier	K.PURTON	12.11.24
	Drg Check	S.HARVEY	12.11.24

* Refer to Revision 1 for Original Signature

Approved For Construction*

Date

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER GENERAL ARRANGEMENT SHEET 2 OF 7

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2102

Rev. A

Document No. ...\Plans\3338703-20-CD-2102

DO NOT SCALE FOR SET OUT DIMENSIONS

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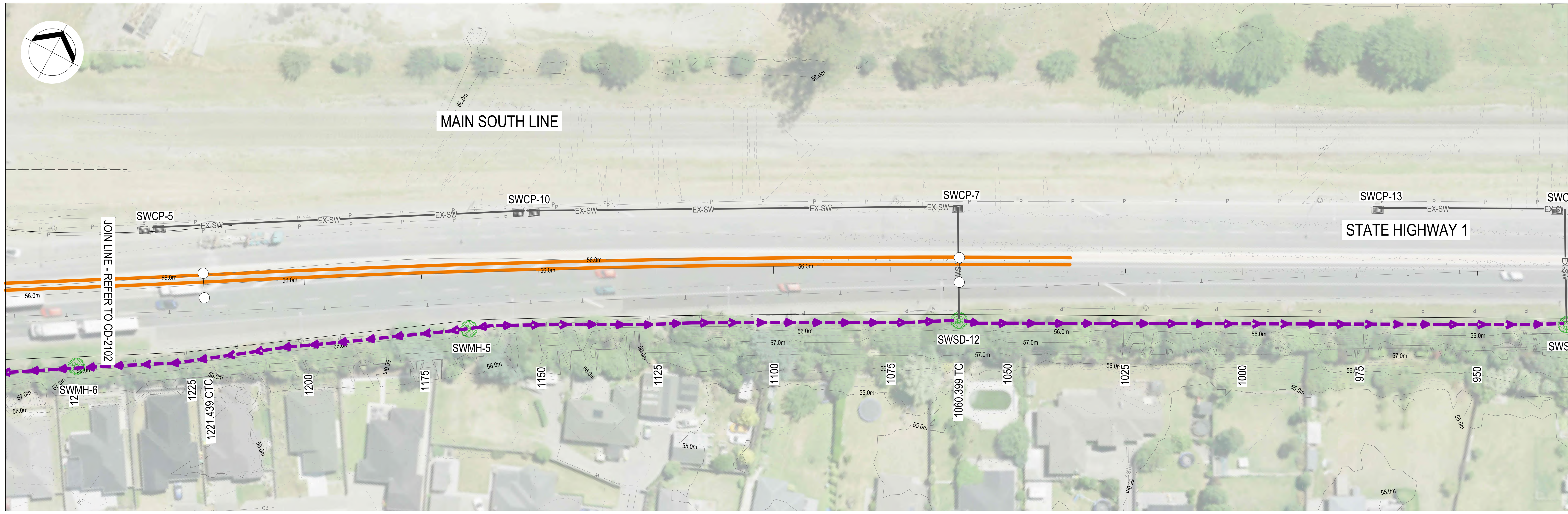
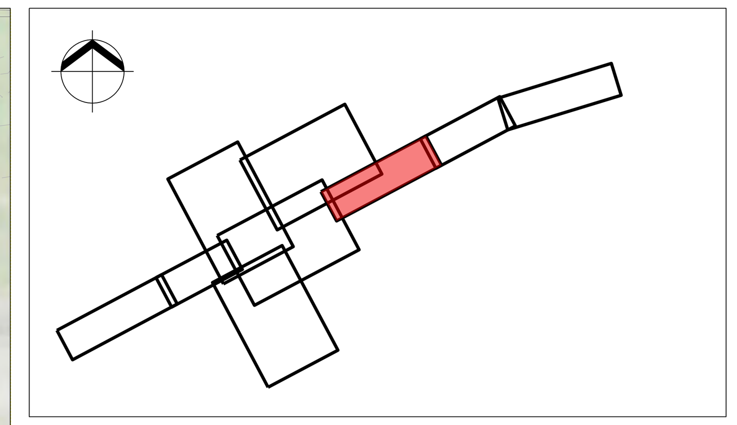
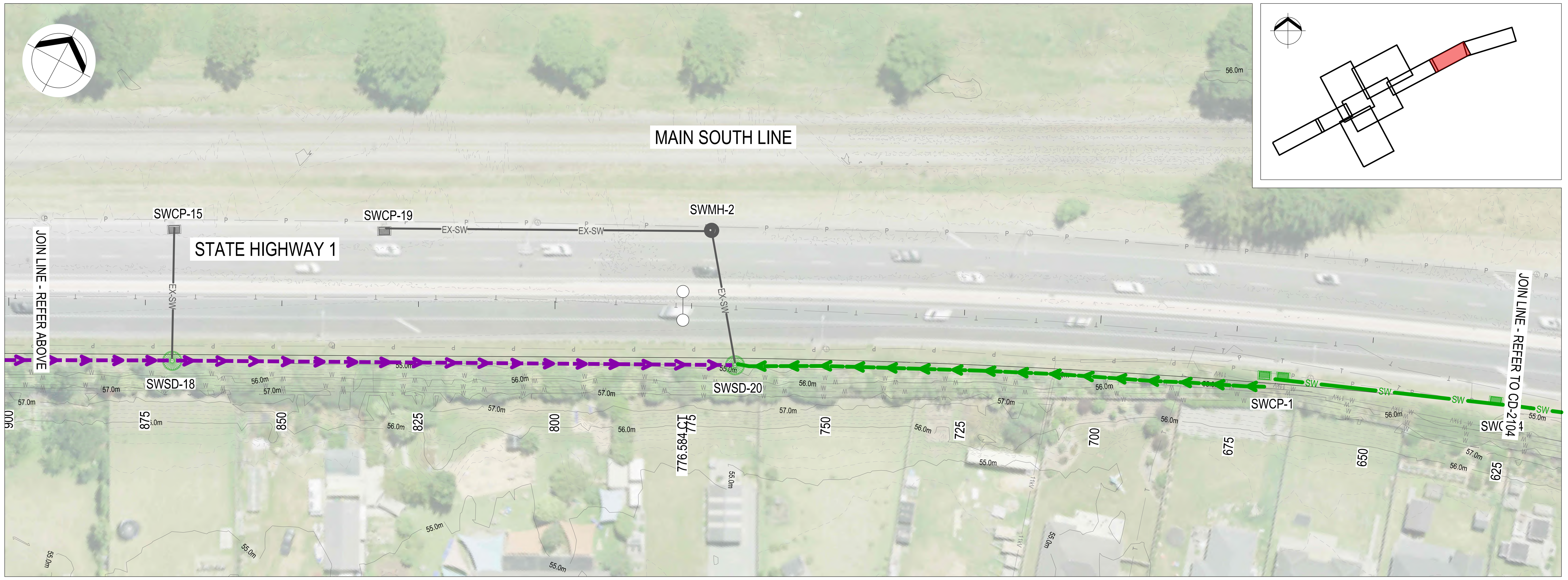
Drawing Ploted: 12/11/2024 5:18:14 pm







LEGEND

CONTOURS
(1.0m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

EXISTING SERVICES
- POWER (11kV)
- POWER (OVERHEAD)
- FIBRE OPTIC
- CHORUS
- WATER MAIN
- STORMWATER
- SEWER
- STOCK WATER RACE
- STORMWATER MANHOLE
- SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT

PROPOSED DESIGN
- STORMWATER MANHOLE
- STORMWATER SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT
- HEADWALL
- KERB BREAKOUT
- ACO DRAIN
- STORMWATER PIPE
- SWALE
- SWALE AND PIPE
- KERB
- EDGE OF SEAL
- GROUND INTERFACE OF BASIN
- TOP OF BASIN
- BASE OF BASIN
- SOAKAGE EXTENT
- SUBSOIL

RESOURCE CONSENT
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No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1) 1:500	Design	G.HAYDEN	19.06.24
	Drawn	G.HAYDEN	19.06.24
Reduced Scale (A3) 1:1000	Dsg Verifier	K.PURTON	12.11.24
	Drg Check	S.HARVEY	12.11.24

Approved For Construction* Date

* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER GENERAL ARRANGEMENT SHEET 3 OF 7

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2103 Rev. A

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Document No. ...\Plans\3338703-20-CD-2103

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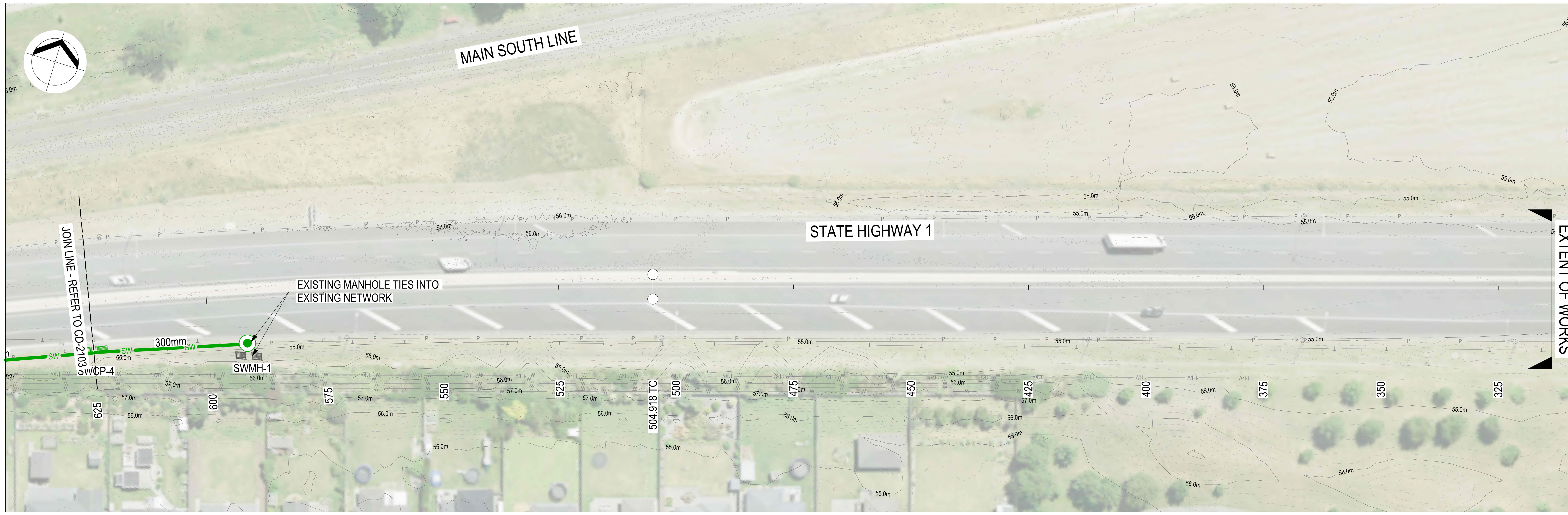
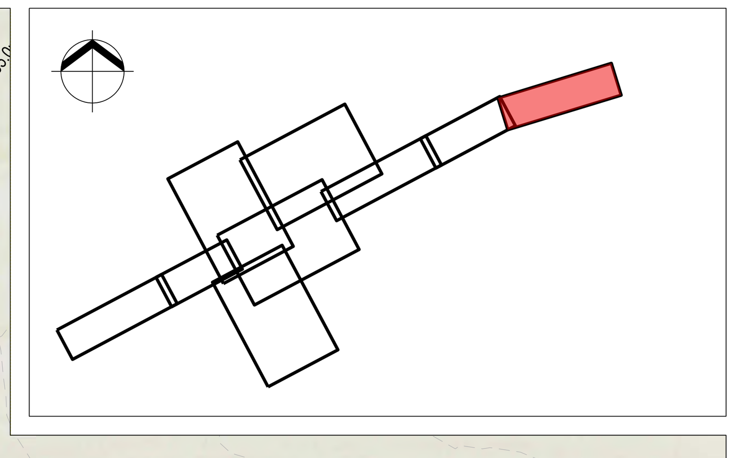
Drawing Plotted: 12/11/2024 5:28:02 pm





LEGEND

CONTOURS
(1.0m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

EXISTING SERVICES
- POWER (11kV)
- POWER (OVERHEAD)
- FIBRE OPTIC
- CHORUS
- WATER MAIN
- STORMWATER
- SEWER
- STOCK WATER RACE
- STORMWATER MANHOLE
- SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT

PROPOSED DESIGN
- STORMWATER MANHOLE
- STORMWATER SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT
- HEADWALL
- KERB BREAKOUT
- ACO DRAIN
- STORMWATER PIPE
- SWALE
- SWALE AND PIPE
- KERB
- EDGE OF SEAL
- GROUND INTERFACE OF BASIN
- TOP OF BASIN
- BASE OF BASIN
- SOAKAGE EXTENT
- SUBSOIL

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No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1) 1:500	Design	G.HAYDEN	04.07.24
	Drawn	M.ALLEN	11.11.24
Reduced Scale (A3) 1:1000	Dsg Verifier	K.PURTON	12.11.24
	Drg Check	S.HARVEY	12.11.24

Approved For Construction*
Date
* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

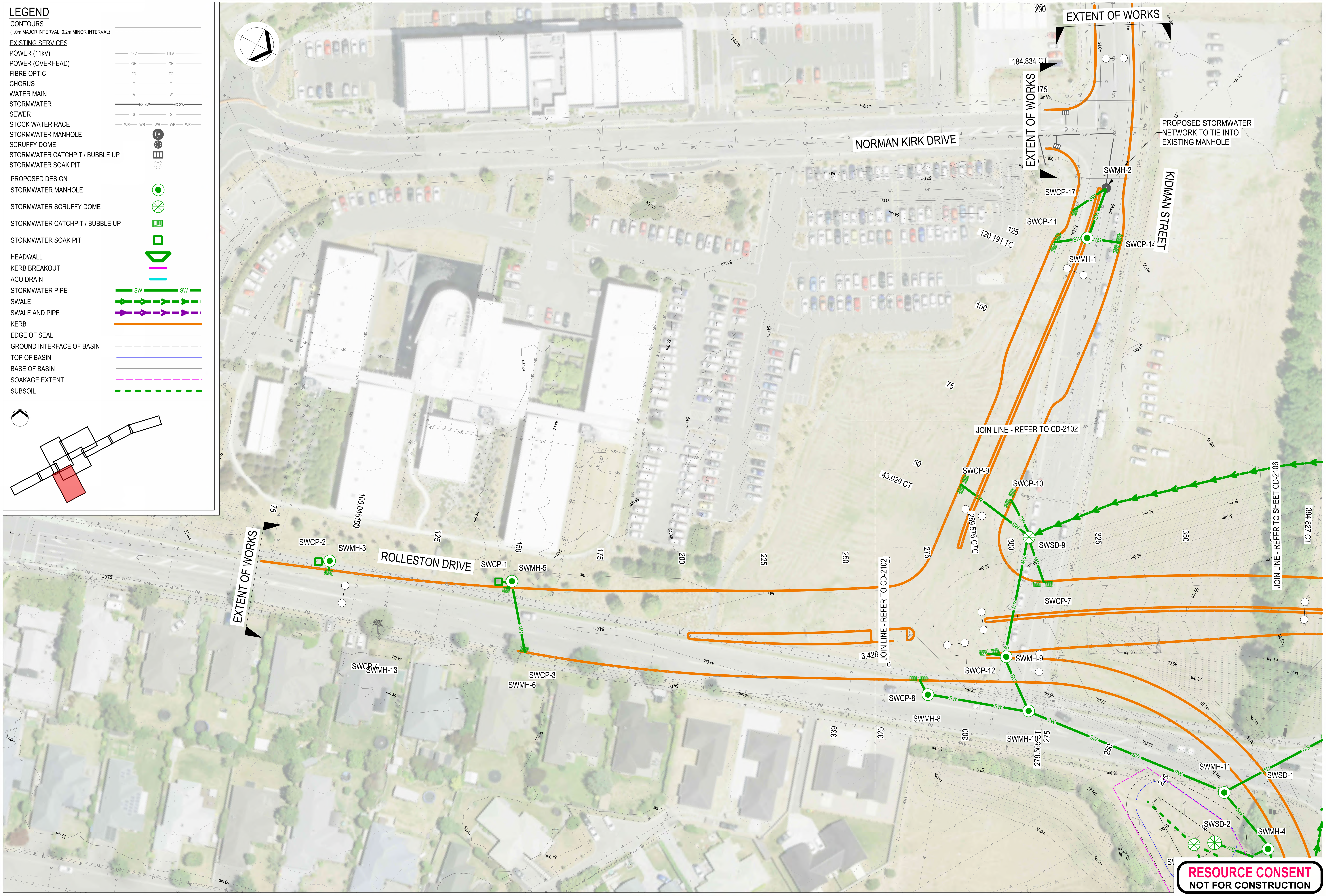
Title: STORMWATER GENERAL ARRANGEMENT SHEET 4 OF 7

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2104 Rev. A

Document No. ...\Plans\3338703-20-CD-2104

Drawing Plotted: 13/11/2024 8:34:50 am



No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1) 1:500
Reduced Scale (A3) 1:1000

Design	G.HAYDEN	05.06.24
Drawn	G.HAYDEN	05.06.24
Dsg Verifier	K.PURTON	12.11.24
Drg Check	S.HARVEY	12.11.24

Approved For Construction*
Date
* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER GENERAL ARRANGEMENT SHEET 5 OF 7

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2105 Rev. A

Document No. ...\Plans\3338703-20-CD-2105

DO NOT SCALE FOR SET OUT DIMENSIONS

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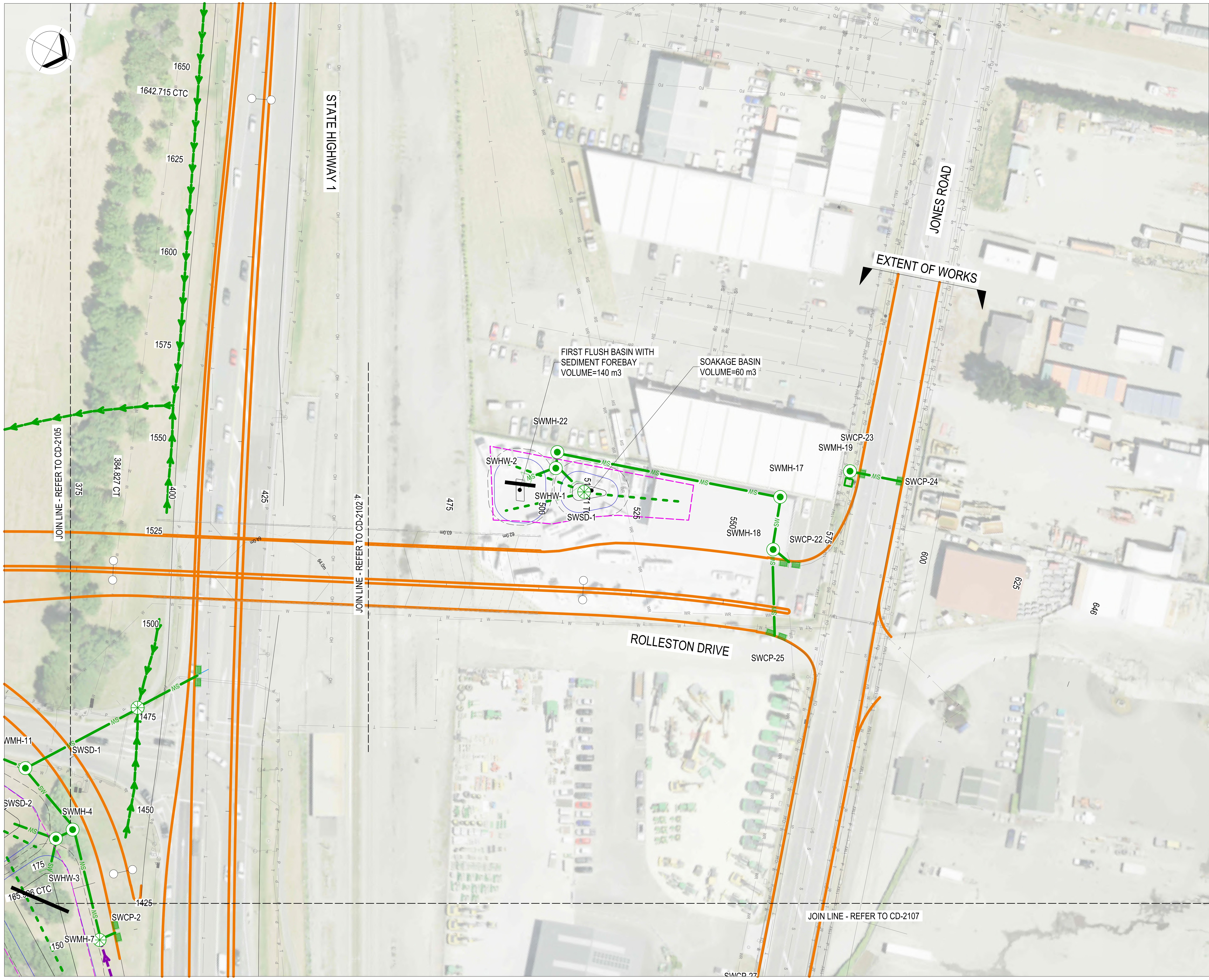
Drawing Ploted: 13/11/2024 8:40:15 am

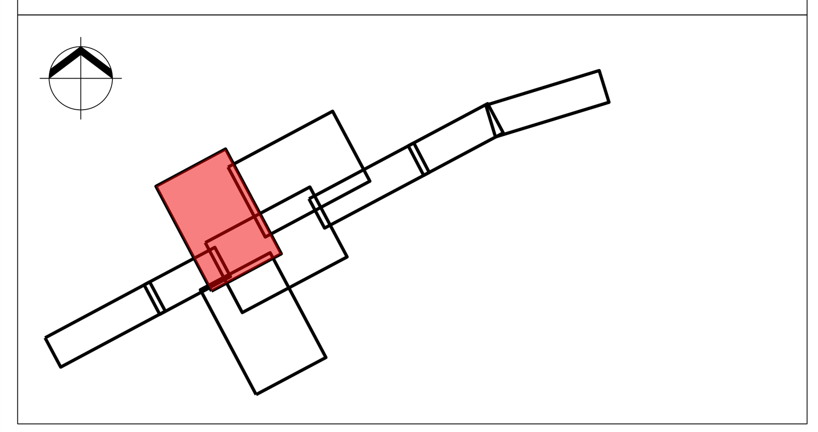


LEGEND

CONTOURS (1.0m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

EXISTING SERVICES
- POWER (11kV)
- POWER (OVERHEAD)
- FIBRE OPTIC
- CHORUS
- WATER MAIN
- STORMWATER
- SEWER
- STOCK WATER RACE
- STORMWATER MANHOLE
- SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT

PROPOSED DESIGN
- STORMWATER MANHOLE
- STORMWATER SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT
- HEADWALL
- KERB BREAKOUT
- ACO DRAIN
- STORMWATER PIPE
- SWALE
- SWALE AND PIPE
- KERB
- EDGE OF SEAL
- GROUND INTERFACE OF BASIN
- TOP OF BASIN
- BASE OF BASIN
- SOAKAGE EXTENT
- SUBSOIL



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No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1)	1:500
Reduced Scale (A3)	1:1000

Design	G.HAYDEN	19.06.24	Approved For Construction*
Drawn	G.HAYDEN	19.06.24	
Dsg Verifier	K.PURTON	12.11.24	
Drg Check	S.HARVEY	12.11.24	Date

* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER GENERAL ARRANGEMENT SHEET 6 OF 7

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2106 Rev. A

Document No. ...\Plans\3338703-20-CD-2106

DO NOT SCALE FOR SET OUT DIMENSIONS

DO NOT SCALE

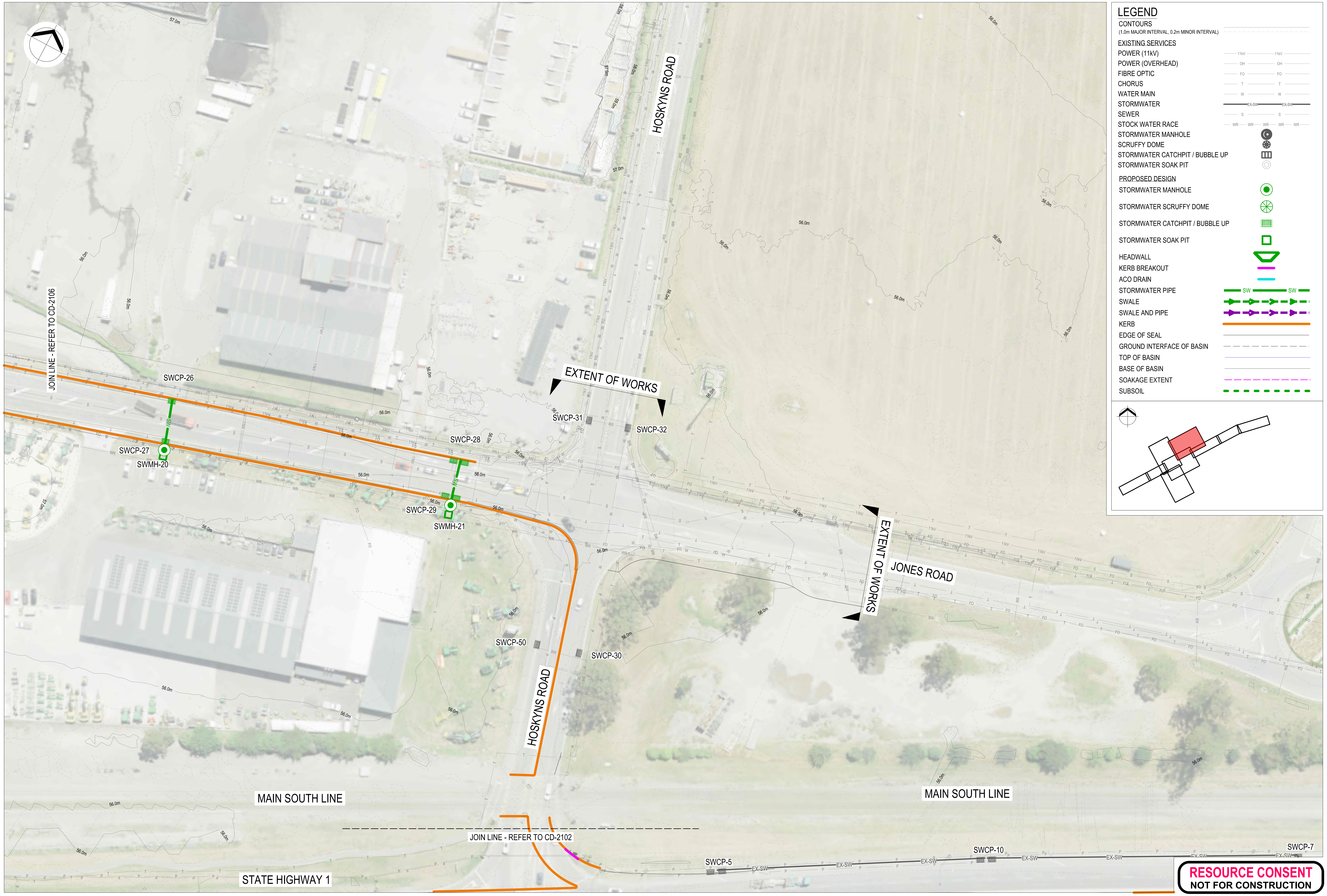
IF IN DOUBT ASK.

LEGEND

CONTOURS
(1.0m MAJOR INTERVAL, 0.2m MINOR INTERVAL)

EXISTING SERVICES
- POWER (11kV)
- POWER (OVERHEAD)
- FIBRE OPTIC
- CHORUS
- WATER MAIN
- STORMWATER
- SEWER
- STOCK WATER RACE
- STORMWATER MANHOLE
- SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT

PROPOSED DESIGN
- STORMWATER MANHOLE
- STORMWATER SCRUFFY DOME
- STORMWATER CATCHPIT / BUBBLE UP
- STORMWATER SOAK PIT
- HEADWALL
- KERB BREAKOUT
- ACO DRAIN
- STORMWATER PIPE
- SWALE
- SWALE AND PIPE
- KERB
- EDGE OF SEAL
- GROUND INTERFACE OF BASIN
- TOP OF BASIN
- BASE OF BASIN
- SOAKAGE EXTENT
- SUBSOIL



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RESOURCE CONSENT
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	GRH	SPH	DGA	12.11.24

Original Scale (A1) 1:500	Design	G.HAYDEN	19.06.24
	Drawn	G.HAYDEN	19.06.24
Reduced Scale (A3) 1:1000	Dsg Verifier	K.PURTON	12.11.24
	Drg Check	S.HARVEY	12.11.24

Approved For Construction* Date
* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCESS IMPROVEMENTS PACKAGE 2

Title: STORMWATER GENERAL ARRANGEMENT SHEET 7 OF 7

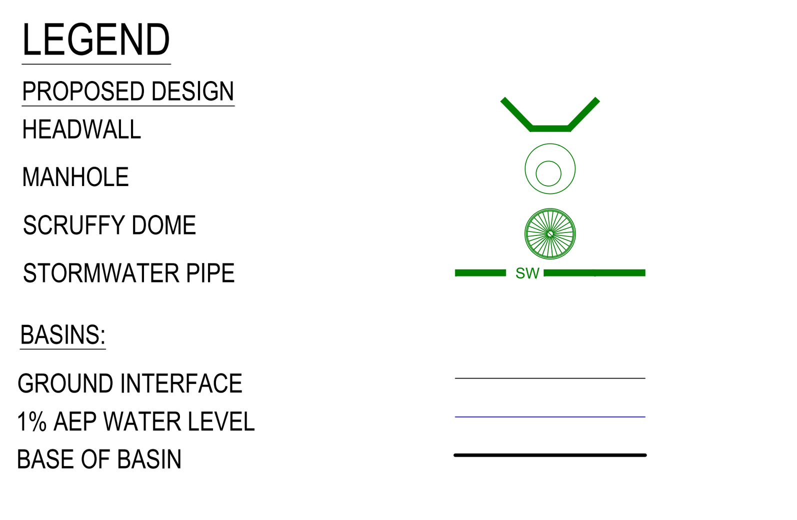
Discipline: CIVIL - DRAINAGE

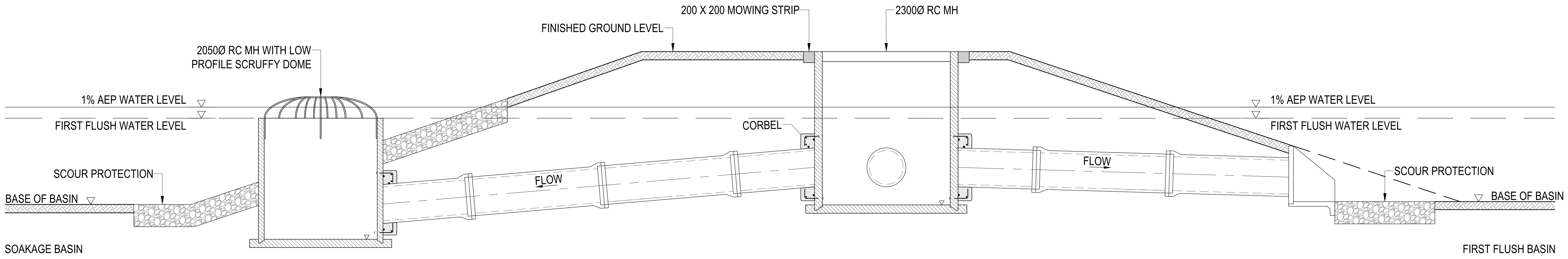
Drawing No. 3338703-20-CD-2107 Rev. A

Document No. ...\Plans\3338703-20-CD-2107

Drawing Plotted: 5:03 pm

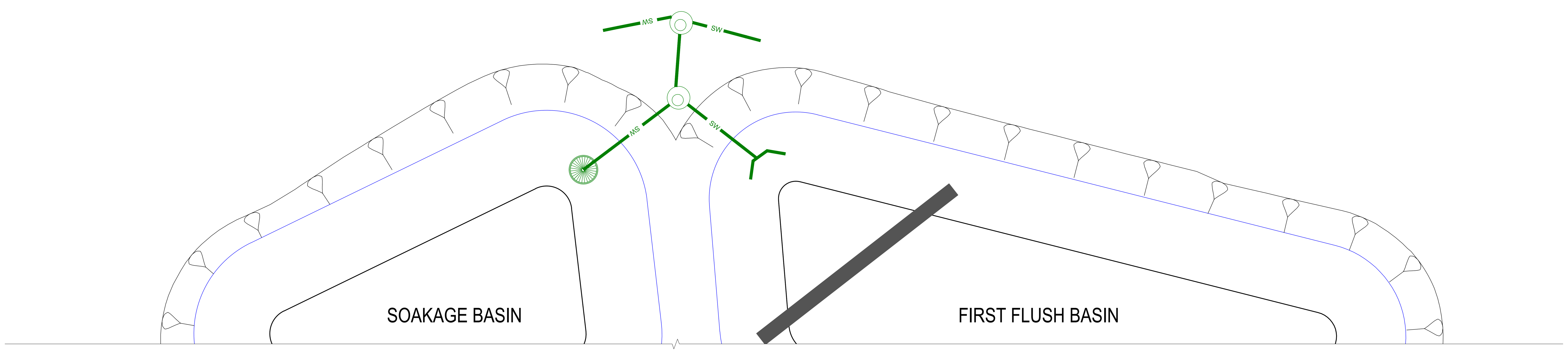
LEGEND





SCRUFFY DOME WEIR OUTLET TO SOAKAGE BASIN TYPICAL SECTION

SCALE 1:50



FLOW SPLITTER BUND TYPICAL DETAIL

SCALE N.T.S

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RESOURCE CONSENT
NOT FOR CONSTRUCTION

A	ISSUED FOR CONSENT	MA	SH	DA	12/11/24
No.	Revision	By	Chk	Appd	Date

Original Scale (A1) AS SHOWN	Design	M.ALLEN	24/10/24	Approved For Construction*
	Drawn	M.ALLEN	24/10/24	
Reduced Scale (A3) 1/2 SHOWN	Dsg Verifier	K.PURTON	12/11/24	
	Drg Check	S.HARVEY	12/11/24	Date
	* Refer to Revision 1 for Original Signature			

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCES IMPROVEMENTS PACKAGE 2

Title: STORMWATER TYPICAL DETAILS SHEET 1

Discipline: CIVIL - DRAINAGE

Drawing No. 3338703-20-CD-2701

Rev. A

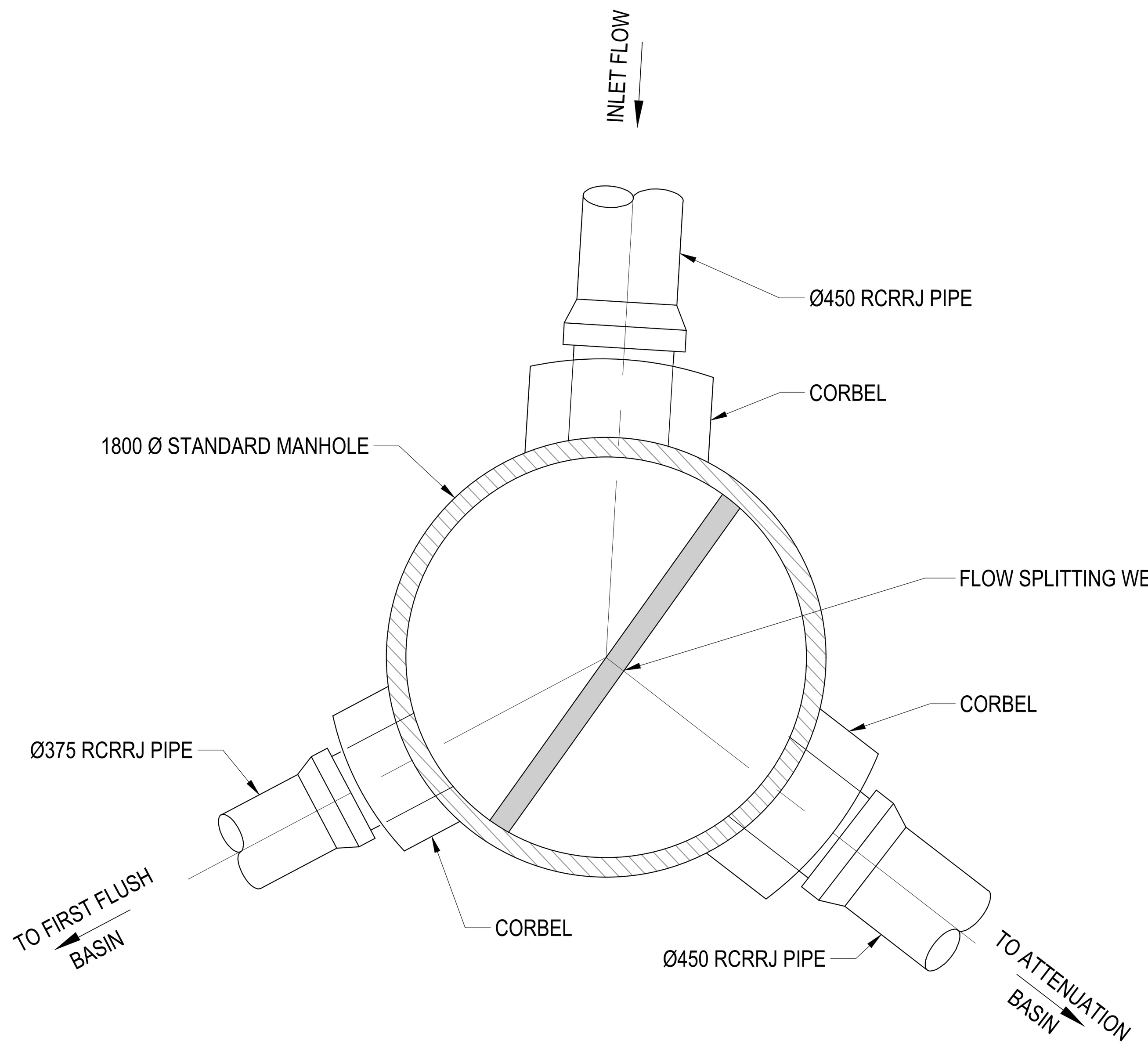
Document No. 3338703-20-CD-2701.DWG

DO NOT SCALE FOR SET OUT DIMENSIONS

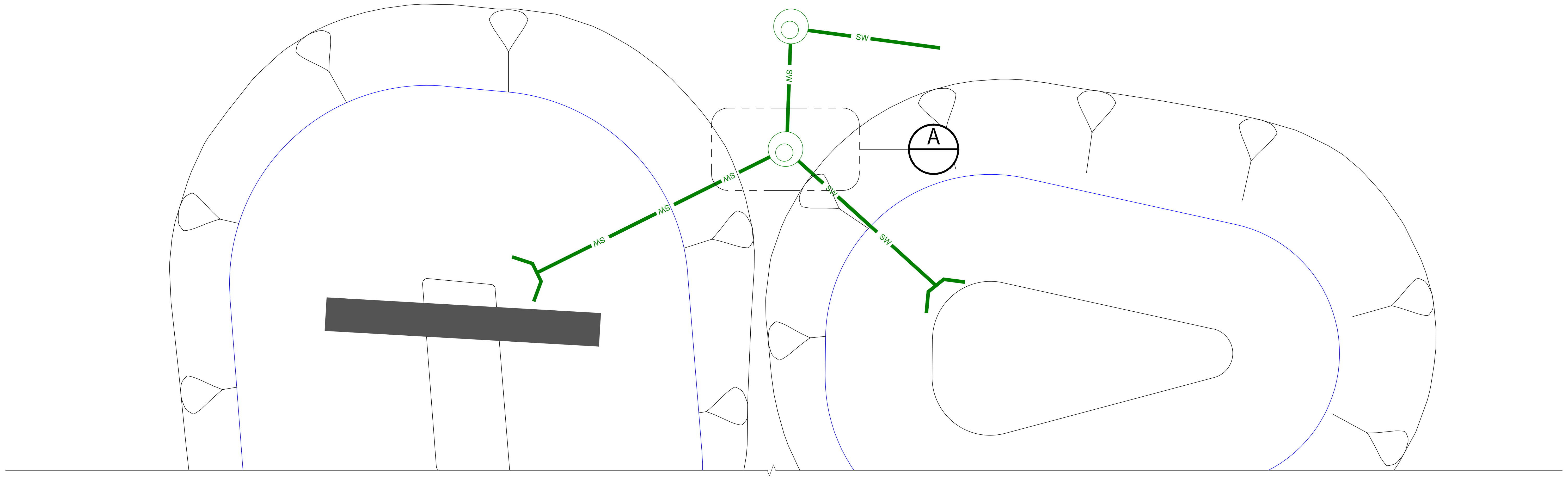
LEGEND

PROPOSED DESIGN
- HEADWALL
- MANHOLE
- SCRUFFY DOME
- STORMWATER PIPE

BASINS:
- GROUND INTERFACE
- 1% AEP WATER LEVEL
- BASE OF BASIN



A A01 FLOW SPLITTER MANHOLE TYPICAL DETAIL
SCALE 1:25



FLOW SPLITTER MANHOLE TYPICAL DETAIL
SCALE N.T.S

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RESOURCE CONSENT
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	MA	SH	DA	12/11/24

Original Scale (A1) AS SHOWN	Design	M.ALLEN	24/10/24	Approved For Construction*
Reduced Scale (A3) 1/2 SHOWN	Drawn	M.ALLEN	24/10/24	
	Dsg Verifier	K.PURTON	12/11/24	
	Drg Check	S.HARVEY	12/11/24	Date

* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCES IMPROVEMENTS PACKAGE 2

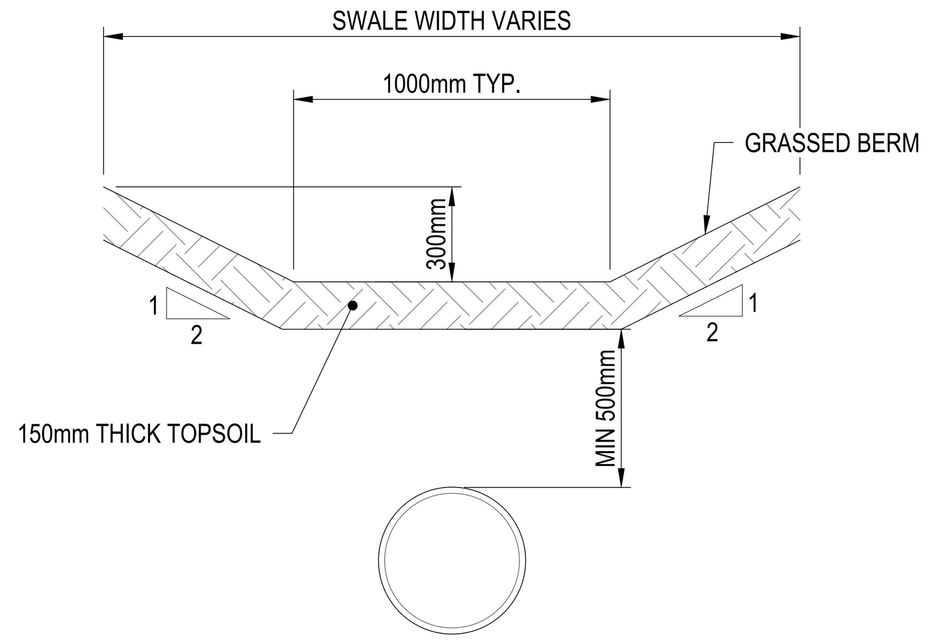
Title: STORMWATER TYPICAL DETAILS SHEET 2

Discipline: CIVIL - DRAINAGE

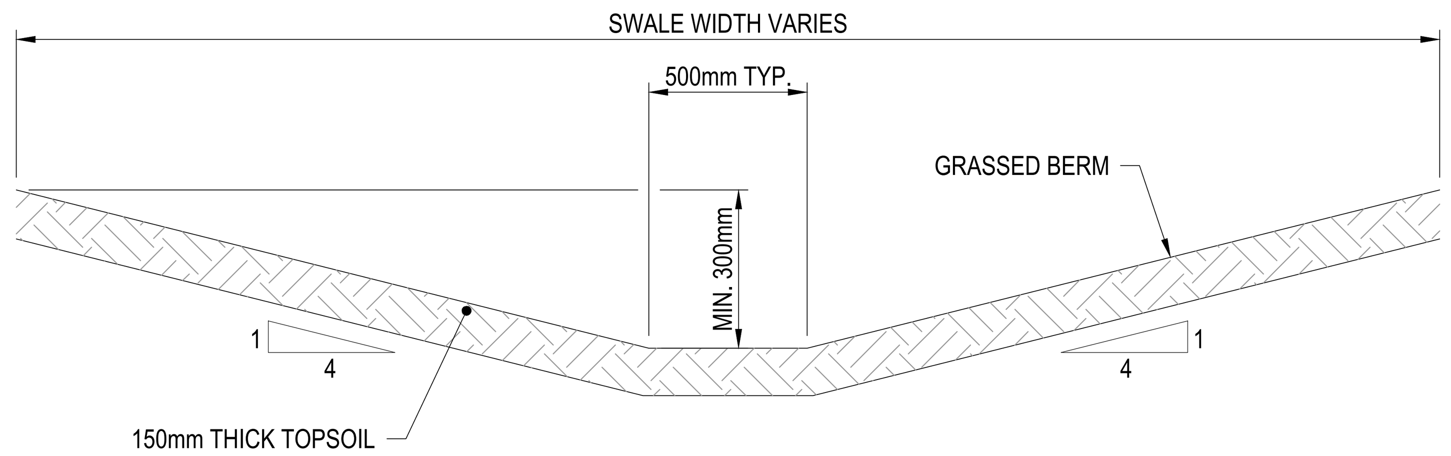
Drawing No. 3338703-20-CD-2702 | Rev. A

Document No. 3338703-20-CD-2702.DWG

DO NOT SCALE FOR SET OUT DIMENSIONS



SWALE AND PIPE TYPICAL SECTION
SCALE 1:20



SWALE TYPICAL SECTION
SCALE 1:20

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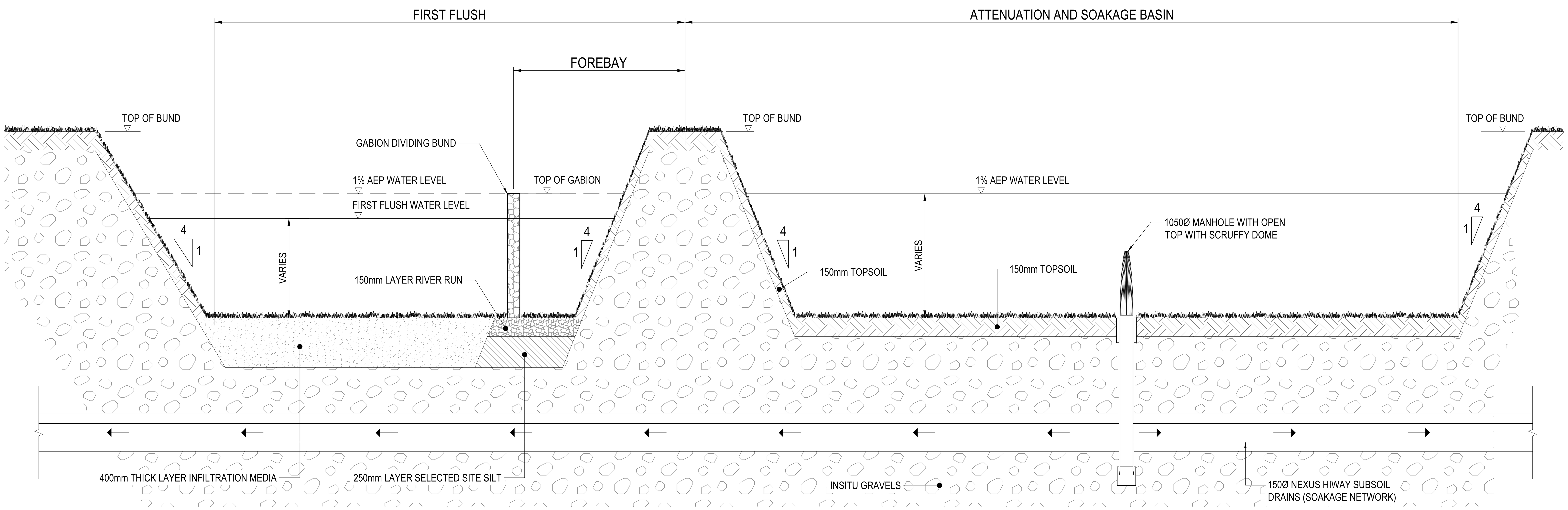

FLOW SPLITTER MANHOLE TYPICAL DETAIL
SCALE N.T.S

RESOURCE CONSENT
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	ISSUED FOR CONSENT	MA	SH	DA	12/11/24

Original Scale (A1) AS SHOWN	Design	M.ALLEN	24/10/24	Approved For Construction*
	Drawn	M.ALLEN	24/10/24	
Reduced Scale (A3)	Dsg Verifier	K.PURTON	12/11/24	
1/2 SHOWN	Drg Check	K.PURTON	12/11/24	Date

* Refer to Revision 1 for Original Signature

Beca

Client: NZ TRANSPORT AGENCY WAKA KOTAHI

Project: SH1 ROLLESTON ACCES IMPROVEMENTS PACKAGE 2

Title: STORMWATER TYPICAL DETAILS SHEET 3

Discipline: CIVIL - DRAINAGE

Drawing No.: 3338703-20-CD-2703

Rev.: A

DO NOT SCALE FOR SET OUT DIMENSIONS

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B

Appendix B – Geotechnical Interpretive Report

Sensitivity: General

SH1 Rolleston Access Improvement

Geotechnical Interpretive Report

Prepared for New Zealand Transport Agency Waka Kotahi (NZTA)
Prepared by Beca Limited

7 October 2024

Contents

Appendices

Revision History

Revision Nº	Prepared By	Description	Date
1	Nellie Yung Ryder O'Neill	For Issue	07/10/2024

Document Acceptance

Action	Name	Signed	Date
Prepared by	Nellie Yung Ryder O'Neill		07/10/2024
Reviewed by	Richard Young		07/10/2024
Approved by	David Aldridge		07/10/2024
on behalf of	Beca Limited		

1 Introduction

New Zealand Transport Agency Waka Kotahi (NZTA) is proposing to improve the transport safety in and around the growing population of Rolleston. Beca Limited (Beca) has been commissioned by NZTA to undertake the transport safety improvements for the Rolleston Access Improvement project. The scope of the work undertaken by Beca is outlined in 8832 SH1 Rolleston Access Improvements Pre-Implementation and Implementation Standard Form Agreement for Professional Services dated September 2023.

The purpose of this report is to summarise the geotechnical assessment undertaken at the preliminary design stage for the development of the Rolleston transport safety improvements. This geotechnical interpretive report presents the following:

- Interpretation and assessment of results, written appraisal of ground and water conditions, analyses, presentation of design parameters and earthworks and structure foundation recommendations;
- Preliminary design details to establish cut and fill slopes, foundation treatments, construction staging and progress constraints plus slip remedial works;
- The location and extent of any additional investigations/testing required to complete the final design and implementation of the recommended option;
- Recommendations on geotechnical parameters to be used for the design and construction of the project.

This report should be read in conjunction with Rolleston Access Improvement – Geotechnical Factual Report (Beca, 2024a) and Rolleston Access Improvement – Preliminary Geotechnical Appraisal Report (Beca, 2024b).

2 Proposed Development

The overall Rolleston Access Improvement Project is a NZTA led project that will improve the transport safety in and around the growing population of Rolleston, the Project comprises two packages:

- Package 1 - SH1 / Dunns Crossing Road Roundabout and associated works.
- Package 2 - Overpass and balance of the works.

Package 1 involves the construction of a two-lane roundabout and associated works to support the safe transport movement along SH1, Dunns Crossing and Walkers Roads. The associated works includes the closure of Dunns Crossing Road to SH1 and cycle subway. The cycle subway will provide for a safe crossing of the State Highway at the Walkers Road / Dunns Crossing Road roundabout. The subway connects the proposed Burnham Cycleway (along Runners Road) with the Rolleston residential area and a walking and cycling connection to the expanding industrial area and shared use paths along Walkers Road and Two Chain Road.

Package 2 involves the construction of an overpass and a balance of works to support the safe transport movement along SH1, Rolleston Drive North and Jones Road. The balance works includes the closure of Rolleston Drive North and SH1 intersection, closure of Hoskyns Road and SH1 intersection and service and access lanes to the overpass.

3 Site Location and Description

The site is located along SH1 and the adjoining roads in Rolleston, which is approximately 20 km southwest of Christchurch CBD. The site is relatively flat with an elevation of approximately 57 m RL (Lyttleton Vertical Datum, LVD 1937). Apart from shallow irrigation channels, the closest water feature is Baileys Creek which flows in a southeast direction and is located approximately 6 km south of site at its closest point.

Figure 1 shows the extent of the proposed improvements (highlighted blue) in relation to the wider Canterbury area. Preliminary drawings are included in Appendix A showing the works along SH1 between the proposed roundabout to the west of Rolleston and overpass to the north, as well as work on the adjoining streets; Rolleston Drive, Jones Road and Hoskyns Road.

Figure 1: Site Location (Image sourced from Canterbury Maps, 2024)

4 Site Geology

The published 1:250,000 geological map for the Christchurch area (Forsyth, P. J; Barrell, D. J. A; Jongens, R., 2008) shows the site to be underlain by two geological units. The proposed new roundabout to the west of the site is underlain by Quaternary aged, brownish-grey river alluvial sands and gravel deposit of late Pleistocene age (Q2a). The proposed overpass is underlain by Holocene river deposits comprising unweathered gravel, sand, silt, and clay (Q1a). Figure 2 shows the geology underlying the site.

The New Zealand Active Faults database (GNS Science, 2024) indicates the nearest mapped active fault is the Greendale Fault, oriented west to east, approximately 2.5 km north of the site. The recurrence interval for the Greendale Fault is stated to be between 10,000 to 20,000 years and is indicated to have a low slip rate. The Greendale Fault was the source of the Mw 7.1 Darfield earthquake that occurred on 4 September 2010 and was not previously recorded in the fault source models for the New Zealand National Seismic Hazard Model (NSHM). There is a possibility that unidentified faults with low slip rates exist in the area but are obscured beneath the thick sediments of the Canterbury Plains (Stirling, et al., 2012).

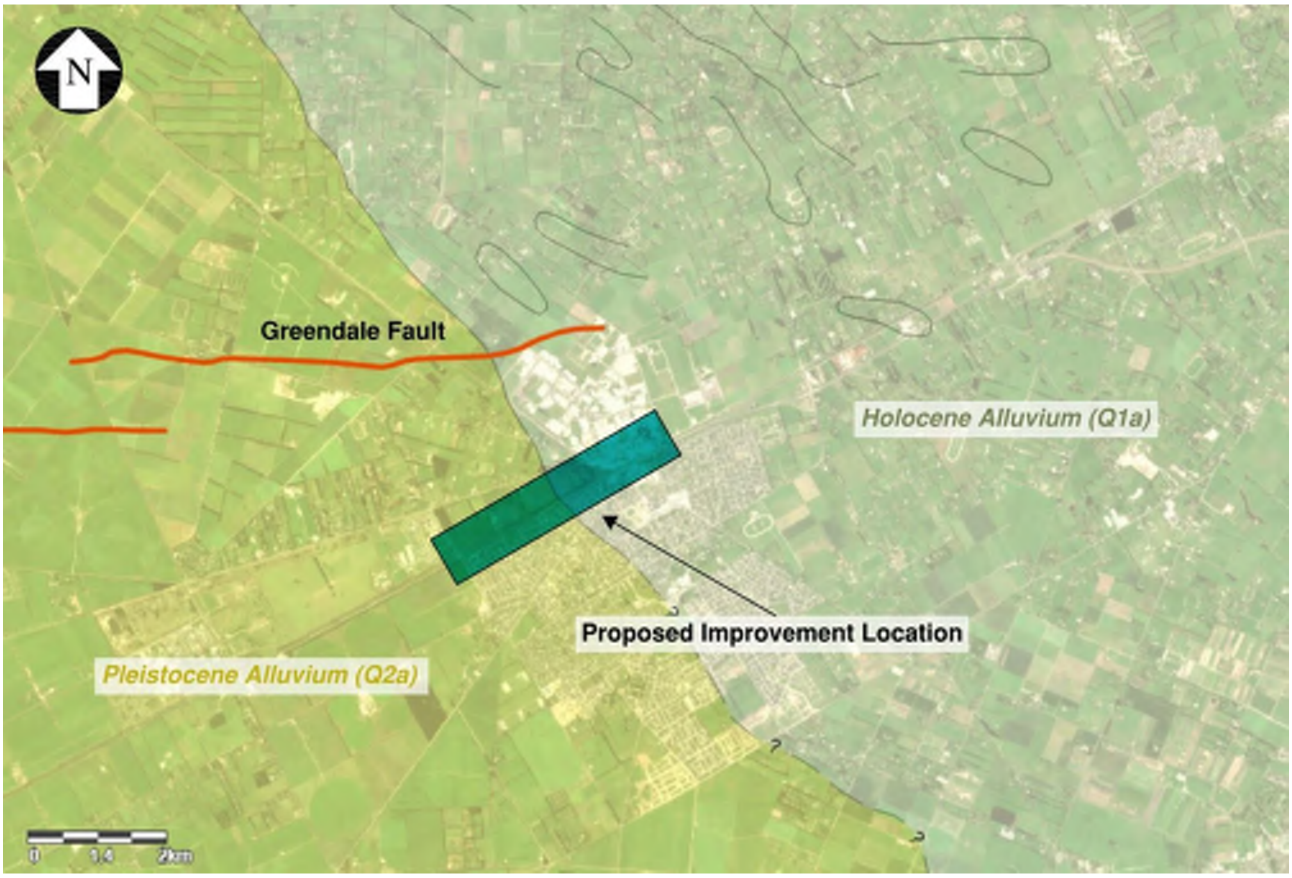

Figure 2: Mapped Geology of the Site (GNS Science, 2024)

5 Geotechnical Investigation

The geotechnical investigation commenced on 17th June 2024 and was completed by 9th August 2024. The investigation locations are shown in Appendix A. The results of this investigation are presented in Rolleston Access Improvement – Geotechnical Factual Report (Beca, 2024a).

The geotechnical investigation comprised:

- 5 machine boreholes to depths between 15.0 to 30.0 m below ground level (m bgl) with Standard Penetration Tests (SPTs) carried out at nominal 1.5 m intervals
  - 3 of the machine boreholes were installed with a piezometer to monitor groundwater levels
  - 2 of the machine boreholes were installed with a PVC casing grouted in place to allow for downhole shear wave testing to be subsequently undertaken
- 2 hand augers to depths to 0.5 m bgl
- 33 machine excavated test pits to depths between 0.6 to 4.0 m bgl
- 23 machine excavated pavement pits to depths between 0.5 to 1.0 m bgl
- Scala penetrometer testing was undertaken at all hand auger, machine excavated test pit and pavement pit locations
- Collection of bulk samples and associated laboratory testing

Based on the quantity and spread of exploratory holes in the geotechnical investigations and uniformity of test results, no further geotechnical investigations are considered necessary for detailed design.

6 Ground Model

6.1 Ground Conditions

The ground model has been assessed based on the geotechnical site investigation data. The ground conditions encountered during the geotechnical site investigation were consistent with the mapped geology, generally comprising of alluvial silts and sands overlying sandy gravel at depth. The ground conditions are relatively consistent across the site with the depth to sandy gravel varying slightly across the site. The generalised ground profile for the site is presented in Table 1.

Embankment fill (Unit 0) was only present within the noise bund situated above ground level and running parallel to the SH, and was only encountered in HA01 and HA02. Topsoil (Unit 1a) was encountered at the ground surface in areas where no prior development had occurred. Fill material (Unit 1b) was found at the ground surface consisting of sandy gravel and was predominantly encountered at the northern end of the proposed overpass between SH1 and Jones Road. The in situ soils comprise a thin layer of loose to medium dense silty gravel or silty sand with minor amounts of organics and trace cobbles (Unit 2), which overlay an unproven thickness of dense to very dense sandy gravel to cobbly sandy gravel with some silt (Unit 3).

Table 1: Generalised Ground Profile

Unit No.	Description	Depth to Top of Layer (m)	Thickness (m)	Scala Penetrometer (blows/50mm)	SPT N (blows/ 300mm)
0	Loose sandy SILT/silty SAND [Embankment FILL]	-	-	1 – 2	-
1a	Loose sandy SILT/silty fine SAND, minor gravel, trace organics [TOPSOIL]	0	0.05 – 0.45 [0.2]	1 – 3	-
1b	Loose sandy GRAVEL/GRAVEL, trace organics, trace silt [FILL]	0 – 3 [0]	0.2 – 0.8 [0.5]	2	-
2	Loose to medium dense silty SAND/silty GRAVEL, minor organics, trace cobbles	0.05 – 0.7 [0.3]	0.15 – 0.8 [0.3]	1 – 2	-
3	Dense to very dense SAND, GRAVEL and COBBLE, silty to some silt	0.1 – 1.5 [0.5]	Unproven	1 – 10+	29 – 50+ [50+]

[ ] indicates typical value adopted for design.

6.2 Groundwater Conditions

The depth to groundwater within the piezometers installed in the boreholes was measured between two to eight weeks following their installation and development. Where piezometers were not installed, the depth to groundwater in two of the boreholes (BH02 and BH04) was measured following completion of drilling. At the time of the measurements the boreholes were cased over their full depths and the boreholes had not been developed to remove drilling muds or other fluids added during the drilling process, hence the water level is indicative only and does not allow for the interpretation of water levels or vertical gradients between individual units. Table 2 summarises the highest groundwater level recorded in each of the boreholes.

Table 2: Groundwater Measurements

Borehole ID	Date of Measurement	Depth to Groundwater (m bgl)	Groundwater Level (m RL)[1]	Type of Measurement (Borehole or Piezometer)	Screened Depth (m bgl)
BH01	21/08/2024	9.77	48.83	Piezometer	10 to 15
BH02	04/07/2024	13.75	41.65	Borehole	-
BH03	21/08/2024	11.98	43.32	Piezometer	10 to 15
BH04	09/08/2024	14.60	41.90	Borehole	-
BH05	21/08/2024	11.91	44.09	Piezometer	10 to 15

[1] RL in terms of NZVD2016

7 Geotechnical Design Parameters

7.1 Soil Parameters

The soil parameters for the in-situ materials have been derived based on correlations to in-situ tests, along with past design experience with these soils at other sites in Rolleston. The parameters adopted for geotechnical analyses are summarised in Table 3.

The ground conditions encountered are predominantly coarse grained (cohesionless), therefore undrained shear strength values have not been assigned for short term load cases. No soil parameters have been assigned for the topsoil as it is assumed it will be stripped during construction.

Table 3: Soil Parameters

Unit No.	Description	Unit Weight, γ (kNm^{-3})	Friction Angle, Φ' (°)	Cohesion, c' (kPa)	Youngs Modulus (MPa)
A	Imported Engineered Fill [AP65]	21	35	-	80
0	Loose sandy SILT/silty SAND [Embankment FILL][1]	17	26	0	-
1a	Loose sandy SILT/silty fine SAND, minor gravel, trace organics [TOPSOIL]	-	-	-	-
1b	Loose sandy GRAVEL/GRAVEL, trace organics, trace silt [FILL]	17	30	0	10
2	Loose to medium dense SILT, SAND and GRAVEL, minor organics, trace cobbles	19	35	0	20
3	Dense to very dense SAND, GRAVEL and COBBLE, silty to some silt	21	39	0	120

7.2 Groundwater Levels

The subsurface conditions of the site can be described as undifferentiated gravel with no confining layers or artesian aquifers, and the hydraulic gradient slopes downward. The groundwater elevation contours for the site, sourced from Canterbury Maps (2024), are shown in Figure 3.

Figure 3: Groundwater Piezometric Contours (RL in terms of metres above sea level)

A review of the groundwater measurements at the site was completed in the Rolleston Access Improvement – Preliminary Geotechnical Appraisal Report (Beca, 2024b). No groundwater data was available for the proposed roundabout within 350 km. The previous investigations near the proposed overpass indicated the groundwater levels ranged from 4.8 to 17.8 m bgl (49.9 to 35.1 m RL, LVD2016). It was inferred that the shallow groundwater readings were likely drilling induced in the boreholes and unlikely to reflect the actual groundwater depth as published maps suggest the depth to groundwater is greater than 6 m (ECan, 2017).

The data collected during the 2024 geotechnical investigation indicates the groundwater is generally 9.7 m to 14.6 m bgl which equates to 48.8 to 41.9 m RL (NZVD2016). The highest levels were recorded shortly after a rainfall event during winter. The groundwater level may rise to higher levels than those recorded following high intensity rainfall events or wet periods, conversely the groundwater levels may be lower than those recorded in drier periods. The depth to groundwater varies between the roundabout and overpass with the shallowest groundwater measurement of 9.7 m bgl (48.8 m RL, NZVD2016) at the roundabout site and deepest groundwater measurement of 14.6 m bgl (41.9 m RL, NZVD2016) at the overpass site.

A design groundwater level of 9.5 m depth was adopted for geotechnical analyses.

7.3 Infiltration

An assessment of the ground infiltration characteristics of the site has been made to inform the development of flood and stormwater design. The assessment showed the ground infiltration characteristics are variable with location and depth. The assessment, including recommended design infiltration rates and factors of safety, is detailed in the Rolleston Access Improvements Infiltration Testing Memorandum attached in Appendix B.

8 Design Basis

8.1 Design Life and Importance Level

A design life of 100 years and an Importance Level (IL) of IL 3 have been assessed for the proposed overpass and subway structures that cross the SH. For the retaining walls that support the noise bund adjacent to the SH, a design life of 100 years and IL2 has been adopted based on the Bridge Manual (NZTA, 2022).

8.2 Seismic Design Criteria

8.2.1 Site Subsoil Class

The site subsoil class was assessed for the site in accordance with NZS 1170.5:2004 and is dependent on the depth of soils or rock at the site.

The geological map for the Christchurch area (Forsyth, P; Barrell, D; Jongens, R., 2008) indicates that dense alluvial materials are likely to continue beyond 100 m depth. This aligns with extensive geophysical investigations that have been undertaken throughout the Canterbury Region to aid in a wider regional site period characterisation. Wotherspoon et. al. (2016) found that site periods away from the Canterbury foothills should be classified as Site Class D in accordance with NZS 1170.5 (Standards New Zealand, 2004).

This is corroborated by the natural period derived from the site specific shear wave velocity measurements in the boreholes for the overpass, which is interpreted to exceed 0.6 seconds.

Based on the geotechnical site investigation encountering dense to very dense sandy gravel at depth and due to the expected thickness of alluvial deposits overlying the basement rock, Site Class D – Deep Soil has been adopted for design.

8.2.2 Seismic Loading

The seismic accelerations for the design of the proposed structures have been determined in accordance with the Bridge Manual (NZTA, 2022). Table 4 presents the seismic loading inputs for design.

Table 4: Geotechnical Seismic Loading Inputs

Parameter	Input
Method	Bridge Manual, Third edition, Amendment 4, Section 6.2.3
Site Subsoil Class	D
Site Location	Rolleston
Structural Importance Level	2 or 3
Structural Design Life	100 Years
Structural Type	Retaining Walls and Bridge
Spectral Shape Factor	1.12
Hazard Factor	0.3

Limited guidance is available for deriving Peak Ground Acceleration (PGA) for Importance Level (IL) 3 structures in Christchurch with both the Bridge Manual (NZTA, 2022) and NZGS Module 1 (2021) not providing information for 1/1000-year annual probabilities of exceedance. Therefore, the PGA has been derived in accordance with Section 6.2.3 of the Bridge Manual and NZS 1170.5. Corresponding displacements calculated for soil structures will be conservative due to PGA being weighted to a magnitude 7.5 earthquake.

In line with Section 5.1.2 of the Bridge Manual (NZTA, 2022) the Collapse Avoidance Limit State (CALS) has been derived by scaling the Damage Control Limit State (DCLS) return period factor (Ru) by 1.5. Similarly, the Serviceability Limit State (SLS) has been derived by scaling the DCLS return period factor (Ru) by 0.25.

Table 5 presents the seismic loadings for the design of the IL2 retaining walls adjacent to SH1.

Table 5: Seismic Design Loadings for Earth Retaining Structure

Design Event	Annual Probability of Exceedance	Return Period Factor (R_u)	Weighted Peak Ground Acceleration (g)	Effective Magnitude (M_w)
CALS	1/1500	1.5	0.50	7.5
DCLS	1/500	1.0	0.34	
SLS	1/25	0.25	0.08	

Table 6 presents the seismic loadings for the design of the IL3 subway and overpass structures.

Table 6: Seismic Design Loadings for Overpass Structure

Design Event	Annual Probability of Exceedance	Return Period Factor (R_u)	Weighted Peak Ground Acceleration (g)	Effective Magnitude (M_w)
CALS	-	1.95	0.66	7.5
DCLS	1/1000	1.3	0.44	
SLS	1/50	0.33	0.11	

9 Liquefaction and Cyclic Softening

9.1 Overview

Liquefaction describes the short-term loss of strength of a loosely packed coarse grained (cohesionless) soil below the water table during an earthquake or other dynamic loading. Liquefaction occurs when the soil particles are sheared and try to contract during dynamic loading, temporarily raising pore water pressures and reducing the effective stress between particles to near zero. This causes the affected soil to behave essentially like a liquid until the excess pore pressures are dissipated.

Liquefaction can have several significant effects where it occurs, including large lateral displacements (lateral spreading), post liquefaction settlements (due to the densification and loss of material to the surface) and potentially large and uneven settlement of shallow founded structures.

9.2 Liquefaction Risk

The geological map for the Christchurch area (Forsyth, P. J; Barrell, D. J. A; Jongens, R., 2008) indicates that dense alluvial materials are likely to continue beyond 100 m depth. The observed seismic performance during the Canterbury Earthquake Sequence (CES) has been described in the Rolleston Access Improvement – Preliminary Geotechnical Appraisal Report (Beca, 2024b) and found that there was no observed liquefaction or lateral spread in the site area.

Recent geotechnical investigations encountered dense to very dense alluvial soils from below the groundwater surface to depths of 30 m bgl, which predominantly consisted of gravel and cobbles with varying proportions of sands and silt. Investigation results within the boreholes consistently recorded SPT N blow counts of 50+. The downhole shear wave velocity (Vs) results typically indicated Vs of no less than 300 m/s, with average values greater than 500 m/s. Based on the depth to groundwater, and the density of the site soils, both SPT N and Vs based analyses indicate that the site has a negligible risk of liquefaction. Hence the site soils are not considered susceptible to lateral spread or liquefaction induced settlement.

10 Subway

10.1 Introduction

The subway is an approximate 28 m long structure that passes beneath SH1 Main South Road providing pedestrian access between Dunns Crossing Road and Walkers Road. During concept design, three different structural forms were considered for the proposed subway including a box culvert, a trapezoidal culvert and a bridge under the carriageway. Further information regarding the different options is discussed in detail within the Structures Options Report - Dunns Crossing Road Subway and Minor Structures (Beca, 2024c).

10.2 Proposed Structure

The chosen structure is a box culvert comprising precast concrete sections that can be segmentally constructed. Around the structure and adjacent to the approach ramps, cut slopes have adopted a batter angle of 1V:3H to accommodate planting / vegetation. If required slopes could be steepened but would need to be assessed during detailed design.

A concept sketch of the proposed subway is shown in Figure 4.

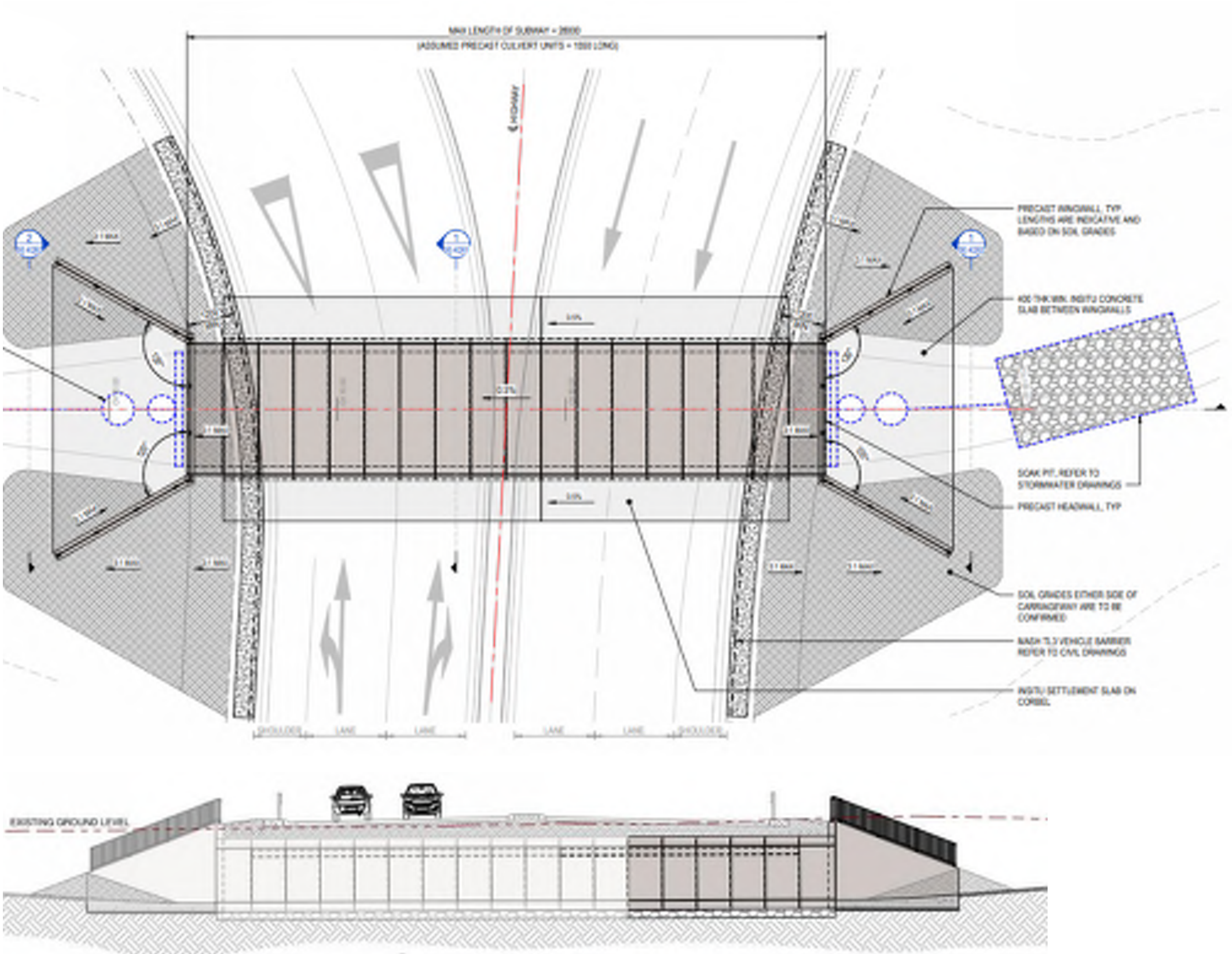

Figure 4: Subway Plan and Long Section

10.3 Foundation Design

10.3.1 Earth Pressure

The static earth pressures acting on the wingwalls was estimated based on the earth pressure coefficients by Eurocode7 NAVFAC (2004). The height of the wingwall considered was 3.7 m at the highest point with no sloping backfill behind the wingwall and Imported Engineered Fill (Unit A) soil parameters were used to derive the earth pressure coefficients. An interface friction coefficient of 0.67 was assumed for the concrete interface.

- K_a = 0.23
- K_o = 0.43

The seismic earth pressures for the wingwalls were calculated based on a stiff wall (Wood & Elms, 1990).

- DCLS = 95 kN/m
- CALS = 140 kN/m

An assessment for the maximum compaction pressures acting behind the culvert and wingwalls was undertaken following CIRIA C516 (2000). Assumptions for the loadings included:

- 1,500 kg vibratory roller
- Roller width of 1 m
- Imported Engineered Fill (Unit A) soil parameters
- Wall height of 3.7 m

The additional pressures induced by compaction loading behind the wingwall, in addition to the at-rest earth pressure, is estimated to be 24 kPa to a depth of 2.7 m. Below 2.7 m, only the at-rest earth pressure is acting.

10.3.2 Bearing Capacity

The box culvert will effectively replace the existing soil and is fully encased by the surrounding soils and proposed to be founded on 300 mm well compacted GAP65 structural fill overlying the dense to very dense gravels. As such, bearing capacity shear failure of the soils beneath the box culvert is not expected. A preliminary assessment of settlement has been carried out using the RocScience software Settle3 (version 5.023) to estimate the total settlement for static loading conditions and as a basis for estimating the modulus of subgrade reaction (soil springs).

10.3.3 Settlement

The box culvert dimensions of 28 m x 5 m x 4 m (length, width, depth) have been modelled in Settle3 software assuming a Boussinesq stress distribution. An indicative dead load of 21 kPa with live loading of 52 kPa was applied to the soil due to the box culvert loading which, based on the initial stiffness of the soil, resulted in less than 5 mm of settlement. However, as there is a net unloading for the box culvert (i.e. the soil weight removed during construction is more than the box culvert loading) the settlement due to culvert induced loading will be a function of the 'rebound' stiffness of the soil, which is expected to be small, less than the calculated 5mm.

10.3.4 Modulus of Subgrade Reaction

The modulus of subgrade reaction, also referred to as 'soil springs', is used by structural designers to model the load to deformation behaviour of soils. It is computed by comparing the ratio of imposed pressure on the soil, to the deflection of the soil. Based on the estimated settlement of 5 mm and indicative loading of 73 kPa, the modulus of subgrade reaction is estimated to be 14,600 kN/m^3. It is recommended that for design a value of 15,000 kN/m^3 is adopted and a sensitivity check on the structure considering 50% and 200% of this value is undertaken.

10.4 Subway Construction Considerations

Site won material could be reused as engineered fill around the box culvert and behind the wingwalls provided that it is appropriately selected and placed. Material larger than 65 mm should be screened and removed from fill that is to be placed below the box culvert and apron slab foundations. The native sandy gravels contain cobbles that may influence the layer thickness and plant needed to compact them and removal of these larger materials may be required.

We recommend that the fill intended to support structures be placed in horizontal lifts, not exceeding 200 mm in loose thickness, and be compacted to at least 95% of the maximum dry density, as determined by the heavy compaction test (NZS 4402:1986 Test 4.1.2). Structural fill (imported or site won) should be clean, well graded granular AP65 material and free of organics and debris. The Geotechnical Engineer should approve the fill material prior to placement.

An allowance for compaction and construction plant induced lateral loads will be included in the design of retaining structures. Location and situation-specific loading conditions estimated to be higher than the above, such as heavy crane loads near temporary cuts, will need to be considered individually by the Contractor, as appropriate.

All construction plant, and all other vehicles having a mass exceeding 1,500kg shall be kept at least 2 m away from the back of the culvert and wingwall facing. Within 2 m of the culvert and wingwall facing, the plant used for compacting the fill material shall be restricted to:

- Vibrating rollers having a mass per metre width of roll not exceeding 1,300kg with total mass not exceeding 1,500 kg
- Vibrating plate compactors having a mass not exceeding 300 kg
- Vibro tampers having a mass not exceeding 75 kg

11 Overpass

11.1 Introduction

The overpass is an approximate 100 m long three-span bridge crossing over SH1 between Rolleston Drive and Jones Road. During concept design, the foundation options considered for the proposed overpass included bored reinforced concrete piles, driven steel piles and shallow raft foundations. Further information regarding the different options is discussed in detail within the Structures Options Report - Overpass Rolleston Access Improvements (Beca, 2024d). A Mechanically Stabilised Earth (MSE) wall is proposed at both ends of the bridge to minimise the area needed to form the approach embankments. The MSE wall is independent of the bridge abutments, which are supported on piles.

11.2 Proposed Foundation

The chosen foundation solution comprises 1500 mm diameter bored reinforced concrete piles at the piers and southern abutment, and a 1200 mm diameter bored reinforced concrete pile at the northern abutment to meet the structural demands. A concept sketch of the proposed long section for the site is shown in Figure 5.

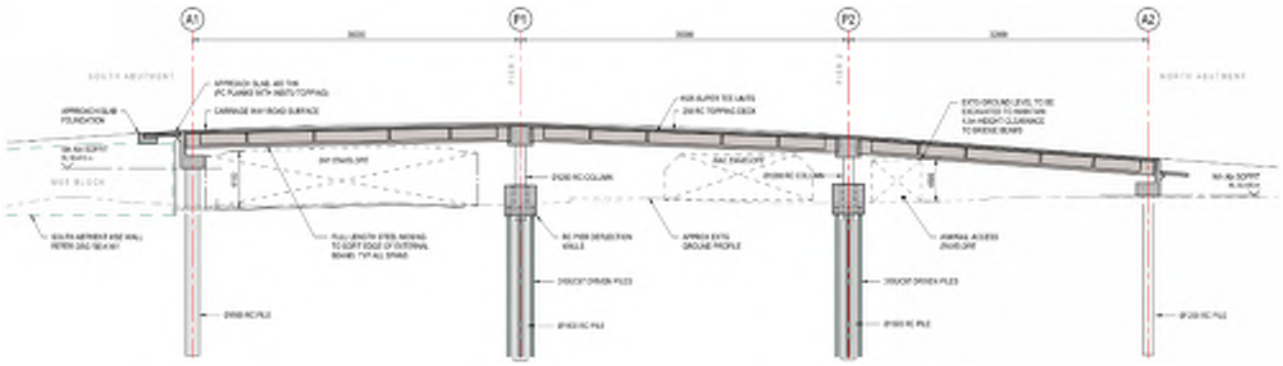

Figure 5: Overpass Long Section

The preliminary structural axial compressive demands are presented in Table 7. The axial compressive demands are taken at the top of the pile (close to ground level) for both the pier and abutment piles and do not account for pile self-weight.

Table 7: Preliminary Structural Axial Loads

	South Abutment	South Pier	North Pier	North Abutment
Pile diameter	1,500 mm	1,500 mm	1,500 mm	1,200 mm
SLS axial load	4,520 kN	8,090 kN	7,510 kN	4,200 kN
ULS axial load	6,910 kN	11,360 kN	10,430 kN	5,820 kN

11.3 Foundation Design

11.3.1 Pile Axial Compressive Capacity

An assessment of the axial compressive capacity has been undertaken using the CivilTech software AllPile (version 7.23a) for the proposed bored pile sizes (1200 mm and 1500 mm). The following assumptions were adopted for the assessment:

- Permanent steel casing was modelled (δ=20º).
- A single pile was modelled and group effects have not been considered.
- The top 2 m of skin friction was ignored to account for soil disturbance during construction.
- Maximum unit end bearing in dense gravels was limited to 12 MPa (API, 2002).

No liquefaction-induced settlement or settlement from the consolidation of the soils beneath the approach embankments / MSE walls is expected based on an assumed construction sequence (which will be confirmed during detailed design). Hence over the design life of the structure, down drag loads have not been considered in the current design of the piles.

The axial compressive capacity considers both skin friction and end bearing for both static and seismic cases and is dependent on careful construction of the piles. The design case at SLS is governed by the tolerable (SLS) settlement being limited to 25mm and at this limited displacement the mobilised axial capacity is less than the ultimate. No strength reduction factor has been applied to the SLS capacity as the method followed a displacement-based design approach. It is recommended that the sensitivity of the structural design be checked against a range of SLS capacities based on the range of soil stiffnesses given below. A geotechnical strength reduction factor of 0.56 in accordance with AS2159 (2009) is to be applied to the ULS capacities for comparison with Load and Resistance Factor Design (LRFD) based demands.

For the SLS cases, it is recommended that vertical p – delta curves (springs) be modelled beneath each pile. The spring stiffness can be derived by dividing the capacity by the estimated settlement including a sensitivity check considering a range of 50% to 200% of the settlement. It is recommended that the difference in spring stiffness be considered between spans, and transversely on a single pile across the row of piles at a pier or abutment to thoroughly assess the potential effect of differential settlement.

The assessed geotechnical axial compressive capacity compared against the pile demands calculated for preliminary design are shown in Table 8.

Table 8: Preliminary Geotechnical Pile Axial Compressive Capacity Summary

Detail	Pile Location		
	North Abutment	South Pier	North Pier and South Abutment
Minimum Pile Length (m)	15	18.5	15
Pile Diameter (mm)	1,200	1,500	1,500
Critical Factored Structural Axial Demand[1] (kN)	**4,200 (SLS)** **5,820 (ULS)**	**8,090 (SLS)** **11,360 (ULS)**	**7,510 (SLS)** **10,430 (ULS)[5]**
Unfactored Pile Weight (kN)	360	660	565
Factored Pile Weight[2] (kN)	360 (SLS) 490 (ULS)	660 (SLS) 890 (ULS)	565 (SLS) 763 (ULS)
Unfactored Skin Friction (kN)	1,100	2,000	1,370
Unfactored Tip Resistance (kN)	13,570	21,200	21,200
SLS capacity (kN) - limiting displacement to 25mm	6,420	8,780	8,475
Factored ULS Capacity[3] (kN)	8,210	12,995	12,640
Dependable Capacity[4] (kN)	**6,060 (SLS)** **7,730 (ULS)**	**8,120 (SLS)** **12,100 (ULS)**	**7,910 (SLS)** **11,880 (ULS)**

Notes:
[1] Structural demand provided by the Structural Engineer excludes pile self-weight.
[2] Pile weight load factor of 1.0 for SLS and 1.35 for ULS.
[3] Strength reduction factor of 0.56 for the ULS case has been applied.
[4] Dependable capacity determined by the difference in capacity and factored pile self-weight for SLS and ULS cases.
[5] Critical factored structural axial demand for North Pier.

11.3.2 Lateral Pile/Bridge Design

Preliminary pile lateral capacity has been assessed using non-linear springs derived by P-Y (horizontal force/displacement) curves generated within the Ensoft LPile 2022 software package to represent both elastic stiffness and lateral capacity. These springs will be used by the Structural Engineers to model pile response in structural analysis software.

The soil profile was modelled using the Sand (Reese, et al., 1974) with nonlinear springs derived for a 1200 mm diameter and a 1500 mm diameter pile at vertical spacing of 1 m assuming no group effects. The proposed P-Y curves have been supplied to the Structural Engineer and a sensitivity check on stiffness considering 50 to 200% was recommended. The springs provided were for preliminary design use and further development of these springs may be required during detailed design as structural design develops.

11.4 MSE and Approach Embankments

11.4.1 Introduction

For the purposes of the current preliminary phase of design simplifying assumptions have been made regarding the configuration and characteristics of the approach embankments and MSE walls and slopes. The intent is to allow the design to progress with due regard for geotechnical considerations.

As the design develops it will be necessary to review these assumptions and confirm the final details. For example, assumptions have been made regarding the MSE walls, however these may be a design-build element that would need to be designed by the supplier.

11.4.2 Settlement

A preliminary settlement analysis for the approach embankments was undertaken using the RocScience software Settle3 (Version 5.023) to estimate their total settlement. The estimated total settlement is less than 25 mm, with immediate settlement typically taken to be 90% of the total in coarse grained soils. A nominal amount of consolidation settlement is anticipated for the site. Whilst the estimated total settlement is relatively low, there is potential for cyclic loading, such as from vehicles, vibration and/or thermal effects. Cyclic loading can cause additional settlement up to 1.5 times the total settlement (Burland & Burbidge, 1985).

11.4.3 Global Slope Stability

11.4.3.1 General Approach

The global slope stability assessment was conducted in accordance with the Bridge Manual with design loads of 12kPa for normal static case and 24kPa for overload static case. Limit equilibrium slope stability analyses were performed using GeoStudio Slope/W (2024.2.0) software package and the Morgenstern-Price method. A pseudo static approach was adopted for modelling seismic loading.

Seismic induced slope displacements are assessed by comparing the critical (yield) PGA to the design PGA. The 50th percentile displacement is assessed under DCLS and CALS loading in accordance with the Bridge Manual. The displacements have been assessed by taking the upper bound of the following methods by:

- Jibson (2007)
- Ambraseys & Menu (1988)
- Ambraseys & Srbulov (1995)

The focal depth and source distance for the Ambraseys & Srbulov (1995) method was assumed to be 6 km and 24 km for the Canterbury Earthquake Region, respectively.

11.4.3.2 Design Cases and Design Loads

Table 9 provides a summary of the design cases and loads assessed as part of the limit equilibrium analysis.

Table 9: Slope Stability Design Cases

Design Case	Target Factor of Safety	Description
Static – Long Term	1.5	Long-term operational conditions encountered by the MSE structure, approach embankments and reinforced slopes. These include normal traffic loads (12 kPa) and long-term water levels (9.5 m bgl). Effective stress parameters are used for all materials.
Static – Short Term	1.2	Short-term analysis of conditions encountered on an irregular basis. These include oversized traffic loads (24 kPa), or potential construction works as well as extreme hydrological conditions associated with flood events (assumed groundwater rises to 0.5 m bgl). Effective stress parameters are used for all materials.
Seismic	1.0 or displacement-based approach if < 1.0	A pseudo-static seismic analysis undertaken with the design peak horizontal ground acceleration, as described in Section 8.

11.4.3.3 Geogrid Reinforcing

Geosynthetic reinforcing is proposed within the embankments and behind the abutment to improve stability where there is insufficient space to construct an unreinforced slope. For preliminary design and modelling purposes Tensar RE580 has been adopted as the geosynthetic reinforcing and has been modelled in Slope/W considering the inbuilt reinforcing parameters. The choice of reinforcing will need to be reviewed and confirmed during detailed design. Slope modelling inputs for the reinforcing are:

- Ultimate tensile capacity = 137 kN/m
- Geogrid reinforcing installed within imported engineered fill
- Geogrid layers spaced at 0.5 m vertical centres
- Geogrid length of 8 m

11.4.3.4 Slip Surface Locations

Slope stability analyses have assessed both longitudinal and transverse sections at the southern abutment where the MSE wall is proposed. Circular slip surfaces have been used and the analyses have considered local failures in front of the abutment, as well as larger global failures. The transverse section considered a stormwater basin located 2 m away from the MSE wall, featuring a berm with a batter angle of 1V:3H.

11.4.3.5 Slope Stability Assessment Results

Table 10 summarises the results of the global slope stability assessment in the longitudinal and transverse sections, and the local slope stability for the longitudinal section. Output from critical slope stability slip circles are presented in Appendix C.

Table 10: Slope Stability Analysis Results

Load Case	Min Factor of Safety (FoS)	Seismic Induced Displacement Limit	Slope/W FoS
Global Longitudinal – 8m geogrid length			
Static – HN	1.5	-	2.1
Static – HO	1.2	-	2.0
Seismic – SLS 0.11g	1.0	-	1.7
Seismic – DCLS 0.44g	1.0	-	1.0
Seismic CALS – 0.66g	-	50 mm	0.7 [<10 mm]
Seismic – Yield PGA	-	-	0.44g
Global Transverse			
Static – HN	1.5	-	2.1
Static – HO	1.2	-	2.0
Static - Flood	1.2	-	1.8
Seismic – SLS 0.11g	1.0	-	1.7
Seismic – DCLS 0.44g	1.0	-	1.0
CALS – 0.66g	-	50 mm	0.7 [<10 mm]
Seismic – Yield PGA	-	-	0.44g
Local Longitudinal – 8m geogrid length			
Static – HN	1.5	-	2.6
Static – HO	1.2	-	2.8
Seismic – SLS 0.11g	1.0	-	2.3
Seismic – DCLS 0.44g	1.0	-	1.3
Seismic CALS – 0.66g	1.0	-	1.0
Seismic – Yield PGA	-	-	-

[ ] values in parentheses are the seismic induced displacements.

11.4.4 Internal Stability

A preliminary assessment for the internal stability of the MSE wall was undertaken in Slope/W including 'local' slip surfaces and the results are captured in Table 10. Based on the geogrid vertical spacing of 0.5 m and the specified ultimate tensile strength of the geogrid (137 kN/m), the factors of safety indicate adequate internal stability of the MSE wall. However, this will need to be reviewed and confirmed during detailed design as the internal stability depends on the facing unit for the MSE wall and geogrid adopted for construction.

11.5 Overpass Construction Considerations

- Bored cast-in-situ piles are typically constructed by drilling a hole supported by either temporary casing or a support fluid (bentonite or similar), installing reinforcement and then placing concrete through a tremie into the base of the hole, displacing the drilling fluid in the process. Experience from previous construction of such piles in New Zealand has shown that the risk of hole instability through alluvial soils (especially below the groundwater table) is significant when adopting a temporary casing or support fluid approach, and permanent casings provide a more reliable outcome, although the steel / soil interface has a corresponding reduction in shaft friction.
- The contractor's methodology will need to consider, as necessary, the advancement of the casing through dense gravels. The effects from noise and vibration during construction will need to be addressed and adverse effects on any adjacent works or properties managed. This includes the installation and withdrawal of any proposed temporary works or staging that the contractor would install to facilitate construction of the bridge.
- During casing installation, cobbles, boulders or other obstructions could be encountered that hinder advancement of the casing.
- A precast driven plug at the pile toe during driving of the steel casing (if adopted) with associated Pile Driver Analysis (PDA) would provide a reliable means of proving end bearing.
- It is recommended that excavated soils within the bored pile be logged by a suitably qualified and experienced individual to confirm an appropriate pile termination depth.
- To achieve the design outcomes it is necessary that the base and steel casing of bored piles be cleaned of loose and otherwise unsuitable material and that this be confirmed prior to placement of concrete.
- Undercutting of unsuitable surficial material may be required prior to embankment construction.
- Two samples were taken from the boreholes; one above the groundwater at depth 4.7 m bgl and one below the groundwater at depth 15.1 m bgl. Samples were tested for soil pH, chloride content and electrical conductivity. The results are summarised in the Rolleston Access Improvement – Geotechnical Factual Report (Beca, 2024a). The results indicate the soil condition is non-corrosive in accordance with SNZ TS 3404:2018.

12 Noise Bund Retaining Wall

12.1 Background

Retaining walls are proposed into an existing noise bund parallel to the southbound lane of SH1 to facilitate widening of the road. Timber post and panel retaining walls are proposed at the required locations. Timber walls were selected based on a high level, qualitative assessment of value-for-money; improved sustainability compared to concrete or steel options; and to facilitate a wall form that can be constructed in a spatially constrained area between SH1 and adjacent property boundaries.

A formal departure will be required from NZTA to accept the use of timber retaining walls for the project, as the design life of timber is assessed to be less than the 100-year design life required by the Bridge Manual (NZTA, 2022). For this project, a reduction to a 50-year design working life is proposed to enable the use of H5 treated timber pole and rail retaining walls. The use of timber pole retaining walls aligns with the intent and examples presented within the ‘NZTA Standardised Design solutions for use on State Highway Roads of National Significance’ (NZTA, 2024).

The currently proposed timber pole walls are considered IL2 structures with retained heights no greater than 1.5 m. Some lengths of the walls will be over 1 m in height and hence, to meet typical design requirements, would require fall protection. However, where the walls are less than 1m high fall protection would not be required.

The area above the walls is relatively inaccessible, being constrained between SH1 and a fence abutting private properties. It is currently densely vegetated and the intention is to re-establish vegetation above the retaining walls along the noise bund, which would require minimal maintenance.

At present it is proposed that consideration be given to omitting fall protection along the top of the walls and this will be discussed with NZTA Waka Kotahi. To mitigate the risk of unplanned visits in these areas, end fences could be constructed. Where temporary planned work is required, specific job safety measures could be adopted to mitigate the fall from height risk.

12.2 Cantilever Wall Design

The locations and typical dimensions of the timber pole retaining walls are summarised in Table 11. Dimensions are based on the preliminary design and will need to be reviewed and confirmed in the next design stage.

Table 11: Retaining Wall Summary

Work Package	SH Route Position (RP) RP-01S-0365	Length (m)	Typical Retaining Wall Height (m)
Package 1	5.148 to 5.565	17	1.5
Package 2	2.005 to 2.065	60	1.0
Package 2	1.460 to 1.565	105	1.5

The cantilever retaining wall analysis was carried out using Geosolve software WALLAP. Design of the timber pole and lagging was undertaken in accordance with NZS AS 1720.1:2022 and load factors from the Bridge Manual (NZTA, 2022). For preliminary design a critical cross section was assessed that considered:

- A retained height of 1.5 m
- The weight from a further 0.5 m of fill above the wall to reflect the increased height of the noise bund
- No live loading above the top of the wall

- A factor of safety on the embedment depth of the pole > 1.5, and deflection at the top of the wall < 20 mm for permanent static load cases
- A factor of safety on the embedment depth of the pole > 1.0, and deflection at the top of the wall < 150 mm for temporary seismic load cases

Based on the assessment of the critical cross section, the design requires 200 mm small end diameter (SED) high density timber poles installed at 1.2 m centre-to-centre spacing. An overall minimum pole length of 3.6 m was assessed with an embedment depth of 2.1 m bgl, founded within a 400 mm diameter bored hole and concreted in place.

For the design features given above, the typical retained height has been considered over the full length of the wall, however, wall heights can be refined during detailed design, with embedment depths reduced where the retained height decreases.

A temporary cut will need to be formed behind the proposed retaining walls to provide sufficient space for construction. The Contractor will need to determine a safe batter angle as part of their temporary works but this will be restricted from crossing the adjacent private property boundary or affecting the existing fencing along the crest of the noise bund. Temporary supports may be required by the Contractor if batter angles are expected to cross the boundaries of private property. Material removed during the temporary cuts could be retained and reused as site won backfill behind the retaining walls; unsuitable and excess material can be cut-to-waste.

13 Pavement Design

Scala Penetrometer / Dynamic Cone Penetrometer blow counts across the site range from 2 to 30 blows per 100 mm. The subgrade can be split into two typical zones, greenfield or virgin ground and pavement pit subgrade. Under the existing pavement is a horizon of lower subbase dense fill underlain by the natural gravel subgrade. The natural gravel subgrade was typically encountered between 0.5 to 0.7 m bgl which recorded typical Scala blow counts of between 3 to 8 per 100 mm.

The greenfield locations have typically lower Scala blow counts due to the presence of silty sandy topsoil. Below the top 200 to 300 mm of topsoil typical Scala blow counts of 3 to 8 per 100 mm were recorded.

Based on the Scala blow counts for the gravel, below any surficial soils, a CBR of 8% can be adopted for pavement design.

14 Applicability

This report has been prepared by Beca on the specific instructions of our Client. It is solely for our Client's use for the purpose for which it is intended in accordance with the agreed scope of work. Any use or reliance by any person contrary to the above, to which Beca has not given its prior written consent, is at that person's own risk.

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

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