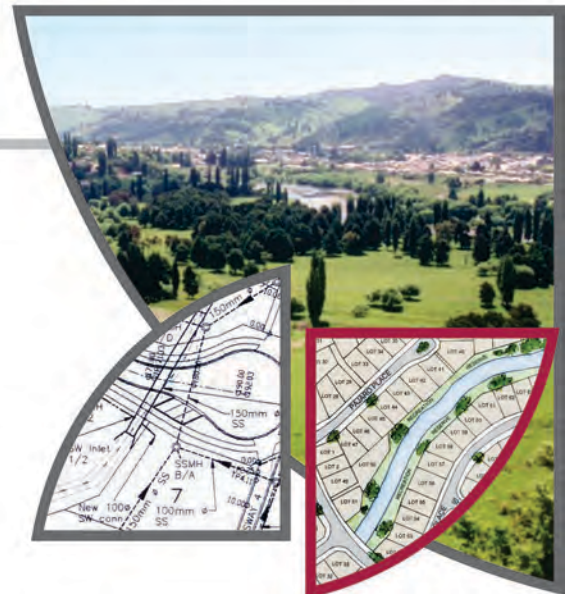


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ENGINEERS • RESOURCE MANAGERS • SURVEYORS



OCEAN RIDGE, KAIKOURA
PROPOSED PLAN CHANGE

STORMWATER MODELLING

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KAIKOURA
PROPOSED PLAN
CHANGE

STORMWATER MODELLING

Project No.	HB00187	Approved for Issue	
Version No.	2	Name	Greg Maddren
Status	Final	Signature	
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STORMWATER MODELLING STUDY

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- B – INFILTRATION TEST
- C – MODELLING MAPS
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OCEAN RIDGE, KAIKOURA - PROPOSED PLAN CHANGE

STORMWATER MODELLING STUDY

1.0 INTRODUCTION

We understand that Cargill Station Ltd. intends to develop the northern and western parts of the site into residential areas and to add a commercial area at the southern end, as illustrated in the proposed Master Plan in Figure 1 below, referred to as the 'post-development condition'.

This report assesses the changes to the 10 and 100-year stormwater conditions in relation to the current land use in 2025, referred to as the 'pre-development condition'. It includes the proposed earthworks and the overall stormwater mitigation, but excludes the local stormwater network within the development areas.

This report should be read in conjunction with the attached Addendum 1, which assesses the dry stormwater basin options in Kekenno Park associated with the future reserve exchange.

This study used the TUFLOW Version 2023-03-AE 2D Direct Rainfall approach for hydrology and hydraulic modelling.



Figure 1: Proposed Master Plan

2.0 DESIGN MODELLING

Table 1: Key Modelling Information

Standards	Water New Zealand National Stormwater Modelling Guide 2024 Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia), 2019.
Terrain data	Canterbury-kaikoura-and-waimakariri-lidar-1m-dem-2022 Vertical Accuracy Specification is +/- 0.2m (95%) Horizontal Accuracy Specification is +/- 1.0m (95%) Fraser Thomas proposed ground level, April 2025
Pipe Network	Council GIS data and as-built information supplemented with the August 2024 Stormwater Pipe Network Survey by Fraser Thomas Ltd Pipes and culverts smaller than 300mm in diameter were not modelled Fraser Thomas proposed embankments and major culverts, May 2025 (the local stormwater network within the development areas was excluded)
Datum	Vertical datum is NZVD2016 Coordinates are in terms of NZGD2000 / Marlborough 2000
Model type	2D Direct Rainfall
Software	TUFLOW Version 2023-03-AE
Rainfall	HIRDS V4 Rainfall Depths Historical Data + 2.1 degree increase to 2090 Synthetic Temporal Design Storm
Losses	Manning 'n' surface roughness Green-Ampt infiltration model based on infiltration test results
Boundaries	Upstream – n/a Downstream stage-discharge condition was generated automatically by TUFLOW based on the estimated water surface slope.
Model grid size	5m with subgrid sampling
Computation interval	Variable time step
Event	10-year and 100-year
Scenarios	2025 Land Use, 'Pre-development' Proposed Plan Change with stormwater mitigation, 'Post-development'

2.1 SOFTWARE

This study used the TUFLOW Version 2023-03-AE 2D Direct Rainfall approach for hydrology and hydraulic modelling.

2.2 SITE INVESTIGATION

A site investigation was undertaken in May 2024 by Fraser Thomas to confirm the existing stormwater management system, including catchments, flow routes, channels and basin infrastructure. An additional survey was undertaken in August 2024 to supplement the Council GIS and as-built information used in the TUFLOW model. A copy of the site investigation report is included in the appendices.

An additional survey was undertaken in April 2025 to locate the culverts under the railway to the north of the site.

2.3 RAINFALL

The Resource Management Act Amendment Act (March 2004) requires councils to have particular regard for the effects of climate change. Incorporating climate change predictions into stormwater design is important if infrastructure is to maintain the same level of service throughout its lifetime.

The climate change allowance was included in the model based on the Ministry of Environment, climate change effects and impacts assessment, A Guidance Manual for Local Government in New Zealand – 2nd Edition, May 2008.

In Kaikoura, the increase in annual mean temperature up to 2090 is expected to be 2.1 degrees. While the average yearly rainfall is expected to decrease slightly, the intensity of storms is expected to increase. The values in MOE Table 5.2 are multiplied by 2.1 to provide an expectation of rainfall for a given storm. For example, for a 10-year 1-hour rainfall, the rainfall taken from HIRDS should be increased by 15.5% (7.4×2.1) to account for global warming.

The historical rainfall depths obtained from HIRDS V4 were adjusted with the climate change factors for design.

ARI (years) → Duration ↓	2	5	10	20	30	50	100
< 10 minutes	8.0	8.0	8.0	8.0	8.0	8.0	8.0
10 minutes	8.0	8.0	8.0	8.0	8.0	8.0	8.0
30 minutes	7.2	7.4	7.6	7.8	8.0	8.0	8.0
1 hour	6.7	7.1	7.4	7.7	8.0	8.0	8.0
2 hours	6.2	6.7	7.2	7.6	8.0	8.0	8.0
3 hours	5.9	6.5	7.0	7.5	8.0	8.0	8.0
6 hours	5.3	6.1	6.8	7.4	8.0	8.0	8.0
12 hours	4.8	5.8	6.5	7.3	8.0	8.0	8.0
24 hours	4.3	5.4	6.3	7.2	8.0	8.0	8.0
48 hours	3.8	5.0	6.1	7.1	7.8	8.0	8.0
72 hours	3.5	4.8	5.9	7.0	7.7	8.0	8.0

Note: This table recommends *percentage* adjustments to apply to extreme rainfall per 1°C of warming, for a range of average recurrence intervals (ARIs.). The percentage changes are mid-range estimates per 1°C and should be used only in a screening assessment. The entries in this table for a duration of 24 hours are based on results from a regional climate model driven for the A2 SRES emissions scenario. The entries for 10-minute duration are based on the theoretical increase in the amount of water held in the atmosphere for a 1°C increase in temperature (8%). Entries for other durations are based on logarithmic (in time) interpolation between the 10-minute and 24-hour rates. Refer to the discussion in section 2.2.4.

Figure 2: MOE Climate Change Effects and Impact Assessment 2008 – Table 5.2

Table 2: HIRDS V4 Rainfall depths (mm): Historical Data

ARI	10m	20m	30m	1h	2h	6h	12h	24h
2	5.17	7.49	9.43	14.1	21.3	40.1	57.7	79.7
5	7.28	10.5	13.2	19.8	29.8	55.6	79.9	110
10	8.98	12.9	16.3	24.3	36.5	68	97.4	134
50	13.6	19.5	24.4	36.3	54.3	101	144	197
100	15.8	22.7	28.4	42.2	63	116	166	227

Table 3: HIRDS V4 Rainfall depths (mm): Historical Data + 2.1 degree increase to 2090

ARI	10m	20m	30m	1h	2h	6h	12h	24h
2	6.04	8.67	10.86	16.08	24.07	44.56	63.52	86.90
5	8.50	12.19	15.25	22.75	33.99	62.72	89.63	122.47
10	10.49	15.00	18.90	28.08	42.02	77.71	110.70	151.73
50	15.88	22.78	28.50	42.40	63.42	117.97	168.19	230.10
100	18.45	26.51	33.17	49.29	73.58	135.49	193.89	265.14

A synthetic temporal rainfall distribution was adopted for the site. The 100-year cumulative and incremental rainfall patterns are shown in Figure 3. Other rainfall events follow a similar pattern.

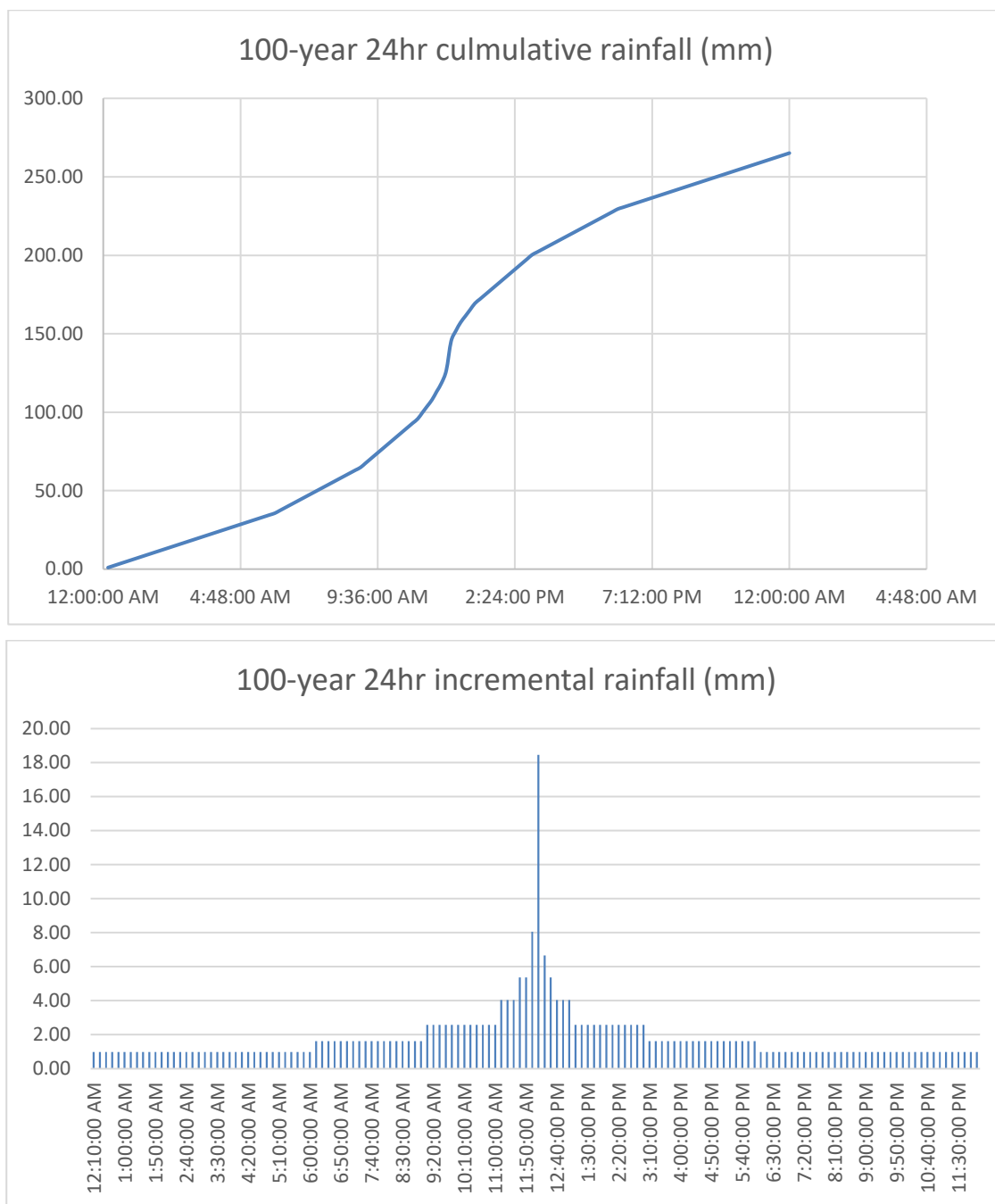


Figure 3: Synthetic temporal rainfall pattern

2.4 ROUGHNESS

The following Manning's roughness coefficients were used in the model. Depth-varying roughness was excluded as it is insignificant in large events.

Table 4: Manning's Roughness Coefficients

ID	Minimum	Average	Maximum	Impervious Fraction	Description
1	0.03	0.04	0.05	0	Open area, grass
2	0.05	0.06	0.07	0	Open area, scrubs
3	0.07	0.10	0.12	0	Open area, trees
4	0.02	0.025	0.03	1	Road and building
5	0.05	0.07	0.08	1	Open water
6	0.03	0.04	0.05	0.2	Rural lifestyle less than 20% impervious
7	0.06	0.09	0.12	0.35	Urban Area - low density, less than 35% impervious
8	0.08	0.12	0.15	0.5	Urban Area - medium density, 35-50% impervious
9	0.1	0.15	0.2	0.7	Urban Area - high density, 50-70% impervious
10	0.1	0.15	0.2	0.7	Commercial/Industrial, 50-70% impervious
11	0.02	0.03	0.04	0	Natural channel, regular section and minimal vegetation
12	0.04	0.05	0.06	0	Natural channel, irregular section and/or heavy vegetation
13	0.06	0.11	0.15	0	Natural channel, very rocky and/or large obstacles
14	0.015	0.018	0.02	1	Concrete channel

2.5 LOSSES

We assume that the initial interception and evaporation of vegetation are negligible. Depression storage is accounted for in the 2D rain-on-grid model. Infiltration is represented by the Green-Ampt model, which varies the infiltration rate over time based on the soil's hydraulic conductivity, suction, porosity, and initial moisture content.

Fraser Thomas conducted ground infiltration testing on 22 July 2024 to ascertain the soil's hydraulic conductivity in the lower section of the site. The infiltration test results range from 64 to 371 mm/hr in this area. The test results are attached.

The higher infiltration rates were disregarded, resulting in a more conservative result. The following soil parameters were used in the Green-Ampt model.

Table 5: Soil Parameters for the Green-Ampt Model

ID	Description	K (mm/hr)	Suction (mm)	Porosity	Initial Moisture	Max ponding (m)
1	Well-drained (sand/gravel)	150	100	0.437	0.02	0
2	Moderately well-drained (loamy sand)	60	120	0.437	0.035	0
3	Poorly/imperfectly drained (clay)	0.5	600	0.475	0.09	0

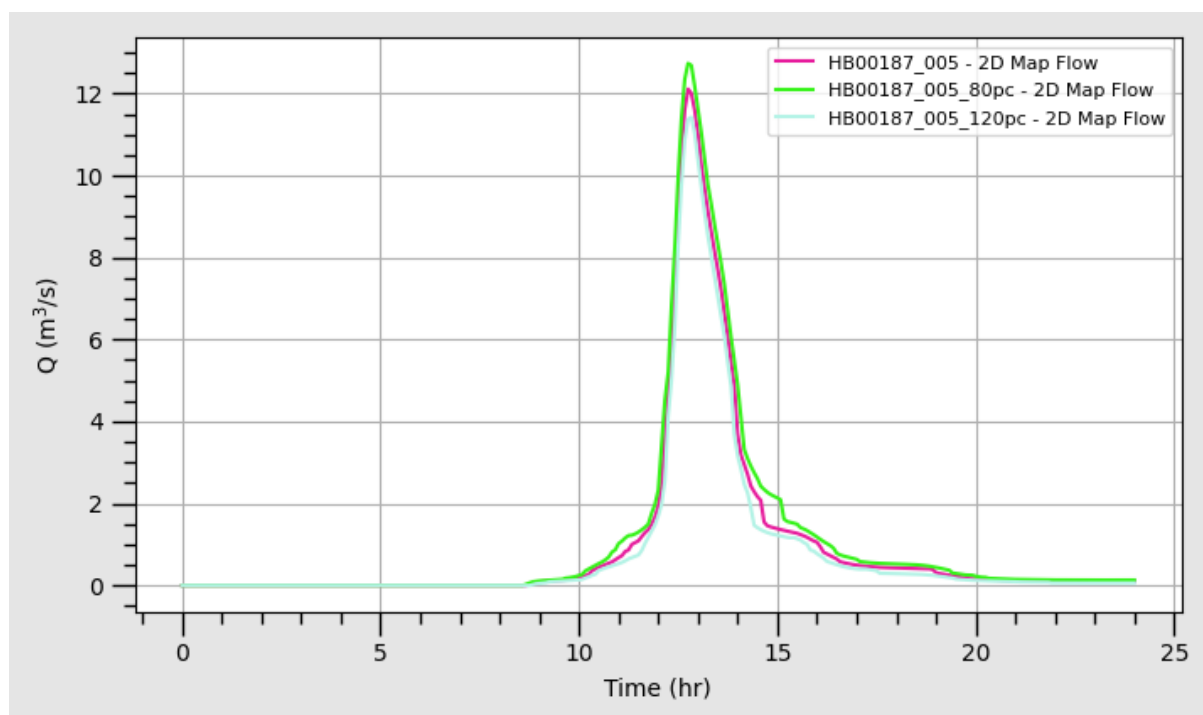
2.6 BOUNDARIES

No upstream inflow is applied in this study area. The downstream stage-discharge condition was generated automatically by TUFLOW, based on the estimated water surface slope.

2.7 SENSITIVITY TEST

Manning's roughness coefficients fall within the recommended values of the industry, which are deemed suitable for this study.

The infiltration test results vary significantly across the site. A sensitivity test was conducted by altering the infiltration rates by +/- 20%, resulting in only a +/- 5% change in the flow rate. Therefore, the infiltration rates adopted for the model are deemed appropriate.

**Figure 4: Sensitivity Test with +/- 20% Infiltration Rates**

2.8 CALIBRATION

The model was not calibrated due to the limited scope of the project.

3.0 EXISTING STORMWATER CONDITION

The current stormwater system within the site performs satisfactorily during 10-year and 100-year events. Most of the flooding occurs in the existing gullies and stormwater detention areas, with minor flooding on the roads. However, due to downstream constraints, water backs up in the system and floods the area immediately north of the state highway, as shown in Figures 5 and 6.

During a 100-year event, it is predicted that flooding will occur on the state highway. Any future intensification should be managed to avoid worsening the current flooding condition.

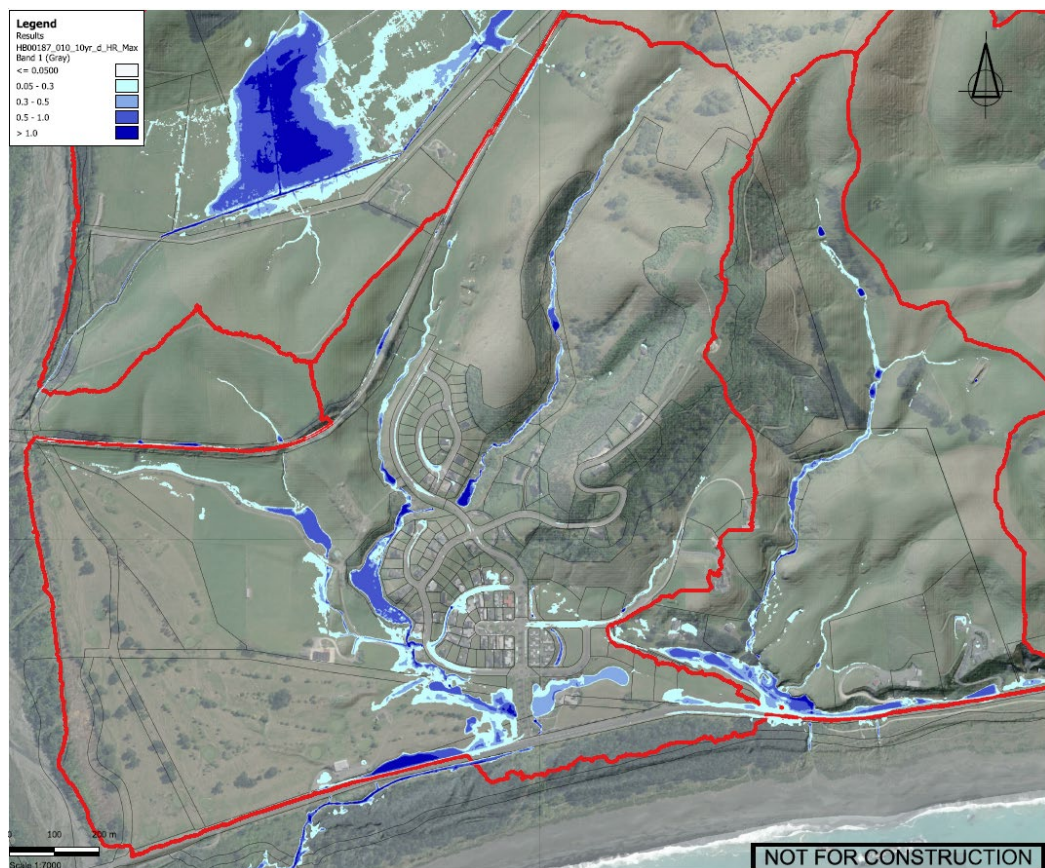


Figure 5: 10-year Pre-development Flood Extent

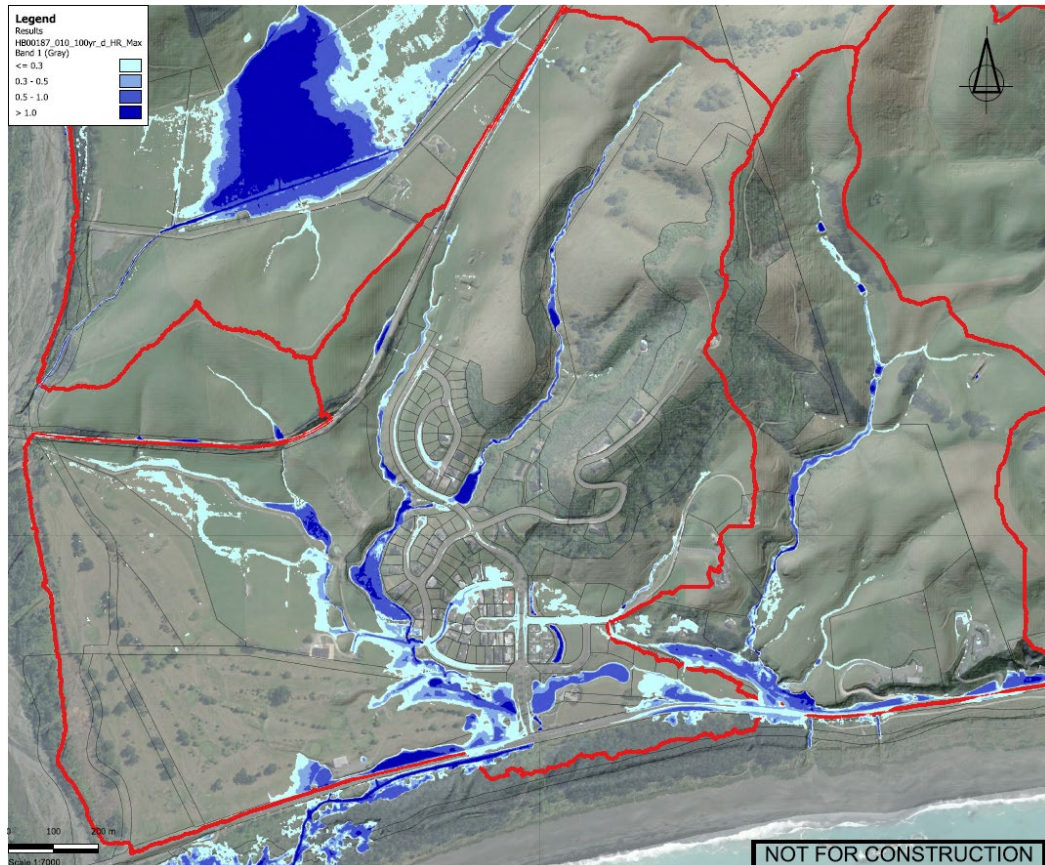


Figure 6: 100-year Pre-development Flood Extent

4.0 PROPOSED DEVELOPMENT AND MITIGATION

The plan change is seeking to alter the anticipated land use activities of various areas within the Ocean Ridge Outline Development Plan, namely:

1. Areas C and G will be retained for residential purposes, but with an overall increase in residential density
2. Portions of Area F identified on the Outline Development Plan as 'Open Space / Grazing' will be repurposed for residential and community uses
3. Portions of Area H and Kekenno Park will be repurposed from their respective low-density residential and utility uses to enable an overall increase in the land identified on the Outline Development Plan as 'Mixed Use Area'.

Existing areas within the proposed Ocean Ridge plan change development, identified as Areas A, B, D(i) and E, have been previously subject to development.

The proposed plan change will increase the impervious area within the site, resulting in greater stormwater runoff and potential impacts on existing stormwater infrastructure. To assess these effects and demonstrate a possible approach for stormwater mitigation, stormwater modelling work has been undertaken. Further details and concept drawings have been included in the appendices.

The concept mitigation considered the following key elements:

- Residential development in the northern and western portions of the site.
- A commercial area located at the southern boundary adjacent to the state highway.
- Relocation of the dry basin closer to the state highway.¹
- Raising the future residential land north of the golf course above the river breakout flood level.
- Abandonment of the culvert under the railway and construction of a new swale to divert runoff from the northern catchment toward the existing southern gullies.
- New embankments and culverts within the gullies to provide additional detention volume.
- Upgrading and extending the existing culvert beneath Ludley Drive to connect with the proposed dry basin.
- A new swale north of the commercial area to convey runoff to the existing open drain.

It is important to note that this stormwater mitigation strategy is presented as a concept only, to illustrate that feasible options are available to support the proposed plan change. The concept design is not prescriptive and does not preclude alternative solutions. Detailed design, including assessment of any viable alternatives, will be undertaken at the development stage.

Similarly, the concept which has been assessed assumes that, in addition to the plan change request, a future reserve exchange process will enable approximately 0.6ha of land at Kekeno Park to be repurposed as part of the proposed expansion of the mixed-use area in Area H.

An addendum has been provided to the report outlining the stormwater design considerations that would apply should the reserve exchange not proceed. Refer to Appendix E.

5.0 POST-DEVELOPMENT STORMWATER CONDITION

The proposed stormwater system within the site performs satisfactorily during 10-year and 100-year events. Most of the flooding occurs in the existing gullies and stormwater detention areas, with minor

¹ As discussed further below, this element would be subject to a future reserve exchange process.

flooding on the roads. However, due to downstream constraints, water backs up in the system and floods the area immediately north of the state highway, as shown in Figures 7 and 8.

The proposed stormwater mitigation reduces downstream flooding during 10-year and 100-year events. However, it does not eliminate flooding across the state highway. It seems impractical to completely remove flooding on the state highway due to the downstream level and backwater effect.

The depth of floodwater increases significantly in existing gullies where new embankments and detention basins are present, as illustrated in Figures 9 and 10. However, it does not have any adverse effects on the development.

The ecological impacts of the proposed development and its associated stormwater management measures should be evaluated.

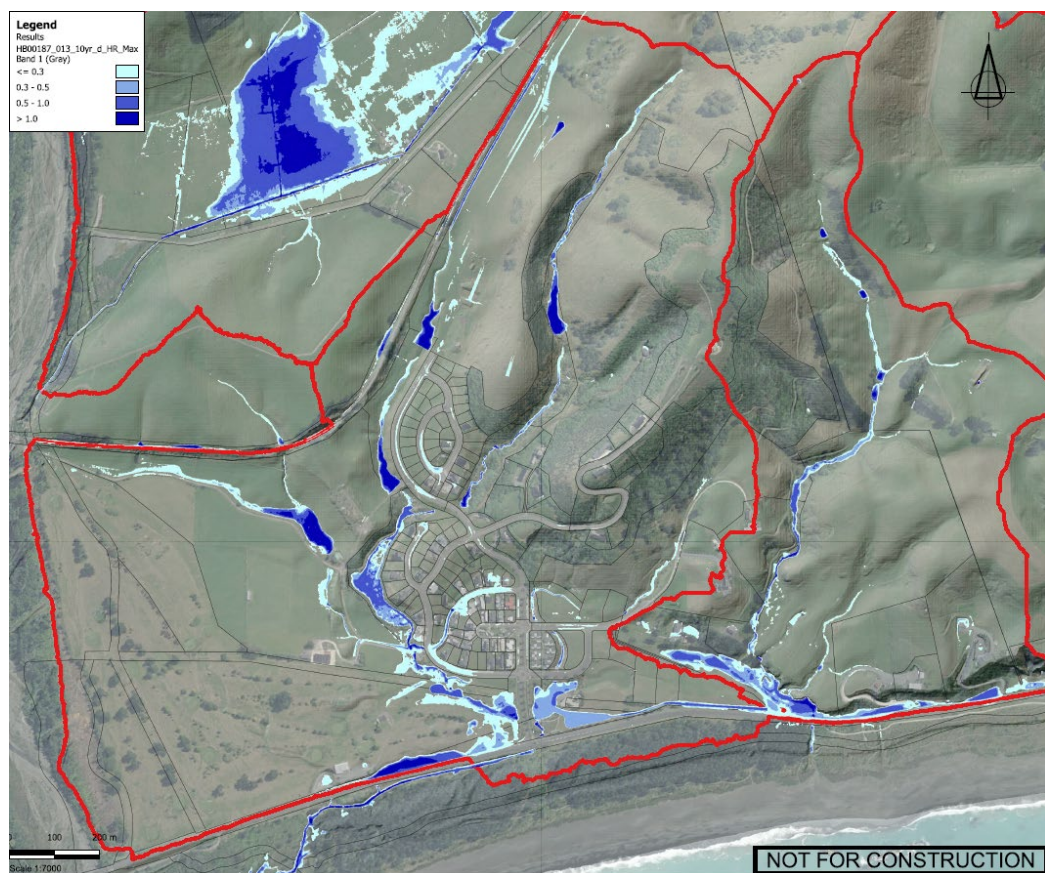


Figure 7: 10-year Post-development Flood Extent

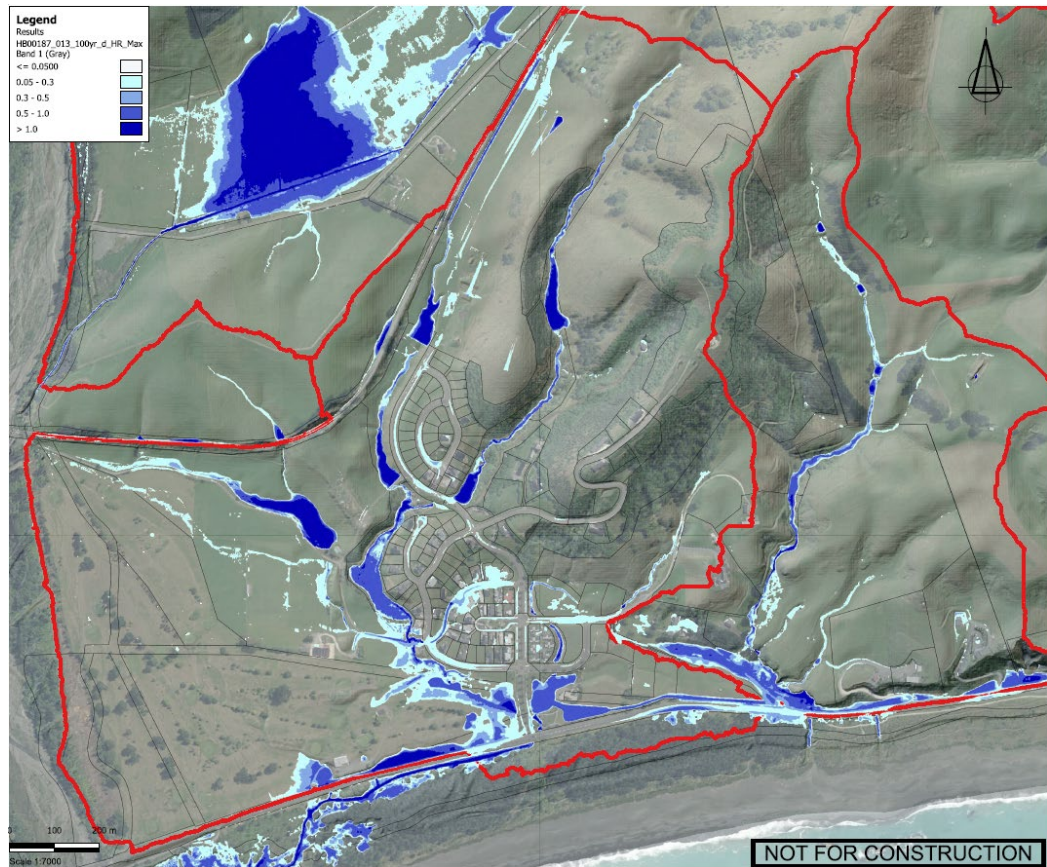


Figure 8: 100-year Post-development Flood Extent

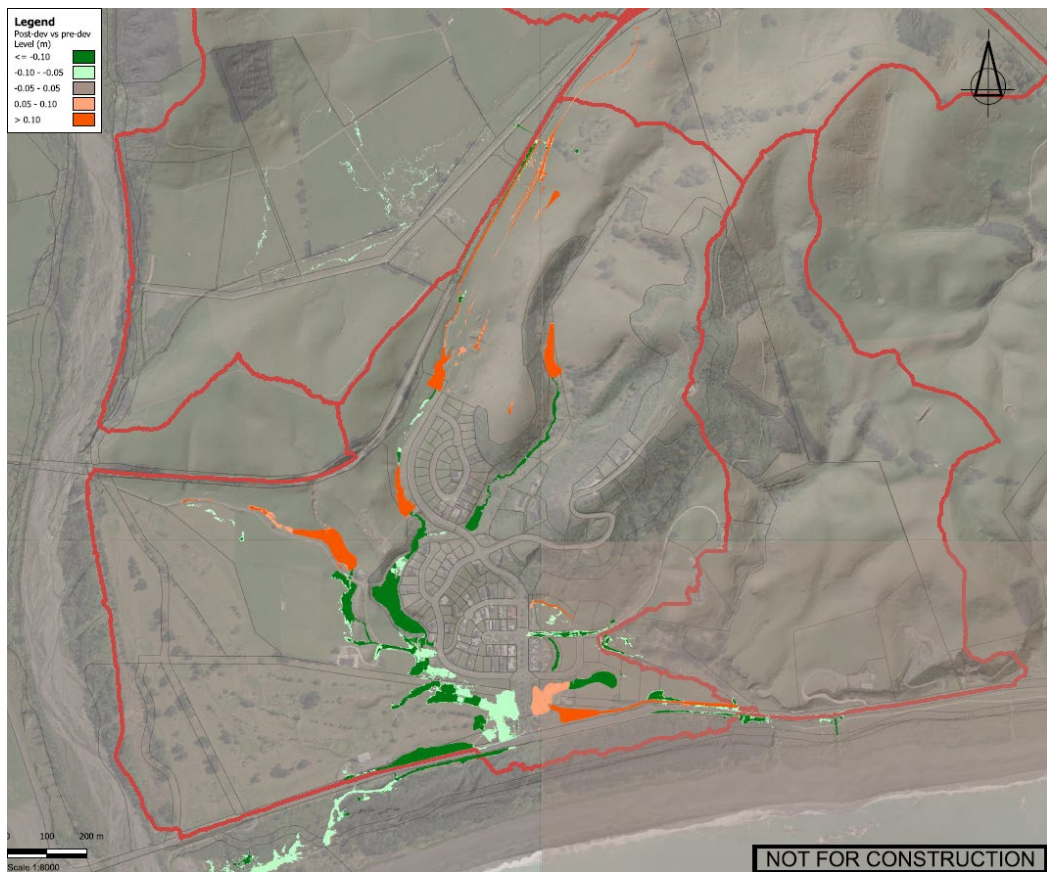


Figure 9: 10-year Flood Level Difference

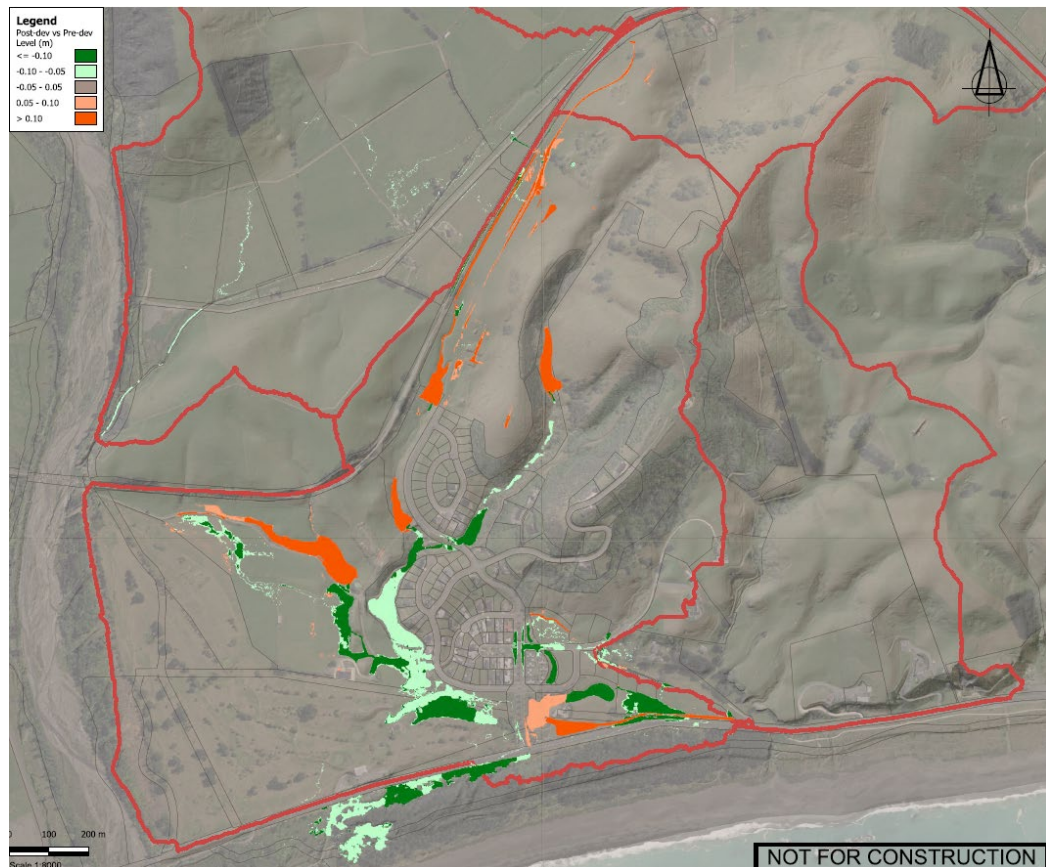


Figure 10: 100-year Flood Level Difference

6.0 CONCLUSION

Based on the site investigation and the flood modelling results, it is our opinion that:

1. The current stormwater system within the site performs satisfactorily during 10-year and 100-year events. Most of the flooding occurs in the existing gullies and stormwater detention areas, with minor flooding on the roads. However, due to downstream constraints, water backs up in the system and floods the area immediately north of the state highway. During a 100-year event, it is predicted that flooding will occur on the state highway.
2. The proposed stormwater mitigation reduces downstream flooding during 10-year and 100-year events. However, it does not remove flooding across the state highway. It appears impractical to completely remove flooding in SH1 due to the downstream level and backwater effect.
3. The depth of floodwater increases significantly in existing gullies where new embankments and detention basins are present. However, it does not have any adverse effects on the development.
4. If the design and recommendations outlined in this report are implemented, development can be adequately serviced regarding stormwater management, and the proposed mitigation measures will ensure that any adverse downstream effects are alleviated.

7.0 RECOMMENDATION

Based on the site investigation and the flood modelling results, we recommend that:

1. The proposed stormwater mitigation outlined in this report should be implemented as part of the development to mitigate any potential adverse downstream effects.
2. The ecological impacts of the proposed development and its associated stormwater management measures should be evaluated.
3. The proposed stormwater embankments are between 1.0m and 3.0m deep. A geotechnical assessment should confirm the feasibility of the proposed embankment location.
4. The flood model should be updated during the detailed design phase to incorporate the final landform and stormwater infrastructure.
5. There is minor flooding within the proposed development area, where the flood water is generally less than 0.3m deep. It should be addressed as part of the detailed design.

8.0 DISCLAIMER

The professional opinion expressed herein has been prepared solely for and is furnished to our client, Cargill Station Ltd and for the information of Council, on the express condition that it will only be used for the purpose for which it is intended.

No liability is accepted by this firm or by any Principal, or Director, or any servant or agent of this firm, in respect of its use by any other person, and any other person who relies upon any matter contained in this report does so entirely at its own risk. This disclaimer shall apply notwithstanding that this report may be made available to any person by any person in connection with any application for permission or approval, or pursuant to any requirement of law.

We do not assume any liability for misrepresentation or items not visible, accessible or present at the subject site during the time of the site inspection; or for the validity or accuracy of any information provided by our client or third parties that have been utilised in the preparation of this report.

The conclusions and recommendations expressed herein should be read in conjunction with the remainder of this report and should not be referred to out of context with the remainder of this report.

APPENDIX A

STORMWATER

SITE INVESTIGATION

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MEMORANDUM

Date: 14 May 2024
To: Developer and consultant team

Project: CH01912 - Ocean Ridge
From: Jan De Kock / Arnold Marquez

OCEAN RIDGE SUBDIVISION STORMWATER SYSTEM SITE INVESTIGATION

1.0 BACKGROUND

Ocean Ridge is a partially completed residential subdivision 2.5km east of Kaikoura township. Ocean Ridge is situated north of State Highway 1 (SH1) and is located in the rural hills west of Kaikoura township and east of the Kowhai River.

Stormwater runoff is collected through a network of pipe reticulated drains and gutters, which discharge into a series of channels and in-line stormwater basins/ponds, followed by discharge off-site, south, below SH1.

The Kaikoura earthquake changed levels in the region, and Kaikoura 2016 earthquake data corrections were subsequently made.



Figure 1: Ocean Ridge Subdivision – Site Location

2.0 STORMWATER SYSTEM INVESTIGATION

Desktop and site investigations were undertaken for the Ocean Ridge subdivision to confirm the existing surface water management system characteristics, including catchments, flow routes, channels and basin infrastructure, and identify infrastructure requiring obvious maintenance, i.e. overgrown ponds, that could contribute to potential reduction in the performance of the stormwater management system.

Refer to the following drawings detailing the Ocean Ridge stormwater system as of April 2024:

- CH01912-SK-001_Rev B Existing site stormwater layout overview
- CH01912-SK-002_Rev B Existing stormwater ponds 1, 2 and 3 layout
- Ocean Ridge - SW survey markups Site survey markup illustrating stormwater details



Figure 2: Overall Ocean Ridge Stormwater System

2.1 KAIKOURA GOLF CLUB AND MARSHLAND DISCHARGE

Stormwater runoff from the adjacent Kaikoura Golf Club greens discharges off-site, below SH1, through 450NB RCRRJ culvert pipes at the three locations shown in Figure 3. These culverts were not actively flowing.

A portion of the Ocean Ridge subdivision north and adjacent to the golf club flows south and discharges through the golf course drainage pipelines below SH1.

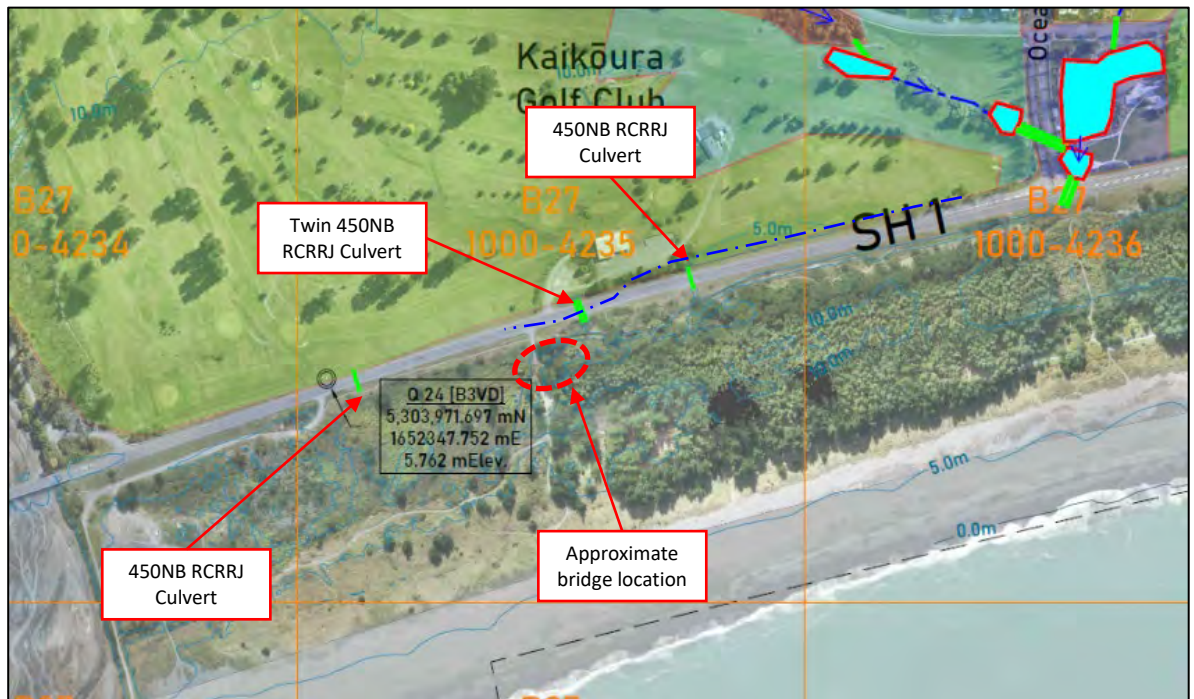


Figure 3: 450NB RCRRJ Culvert Pipes Discharging from the Golf Course and Crossing SH1

The 450NB SH1 culvert outlets (at the south side of SH1) were found to be either partially broken or obstructed by vegetation. All culverts were angled eastwards, which follows the expected drainage path trend.



Figure 4: 450NB RCRRJ Culvert Outlet Discharging from the Golf Course (Crossing SH1)



Figure 5: Twin 450NB RCRRJ Culvert Inlets Discharging from the Golf Course (Crossing SH1)



Figure 6: 450NB RCRRJ Culvert Inlet Discharging from the Golf Course (Crossing SH1)

2.2 NORTH AND WEST CATCHMENTS (UPPER CATCHMENT) STORMWATER FLOWS

Stormwater from the upper reaches of the Ocean Ridge follow two paths from the top of the subdivision down towards SH1.

Runoff from the northern part of Ocean Ridge flows southwards through Ponds 8, 7, 6, 5 and then onto Pond 4.

Pond 9 flows south east to Pond 4, and was found to be heavily vegetated at the time of the site investigation.

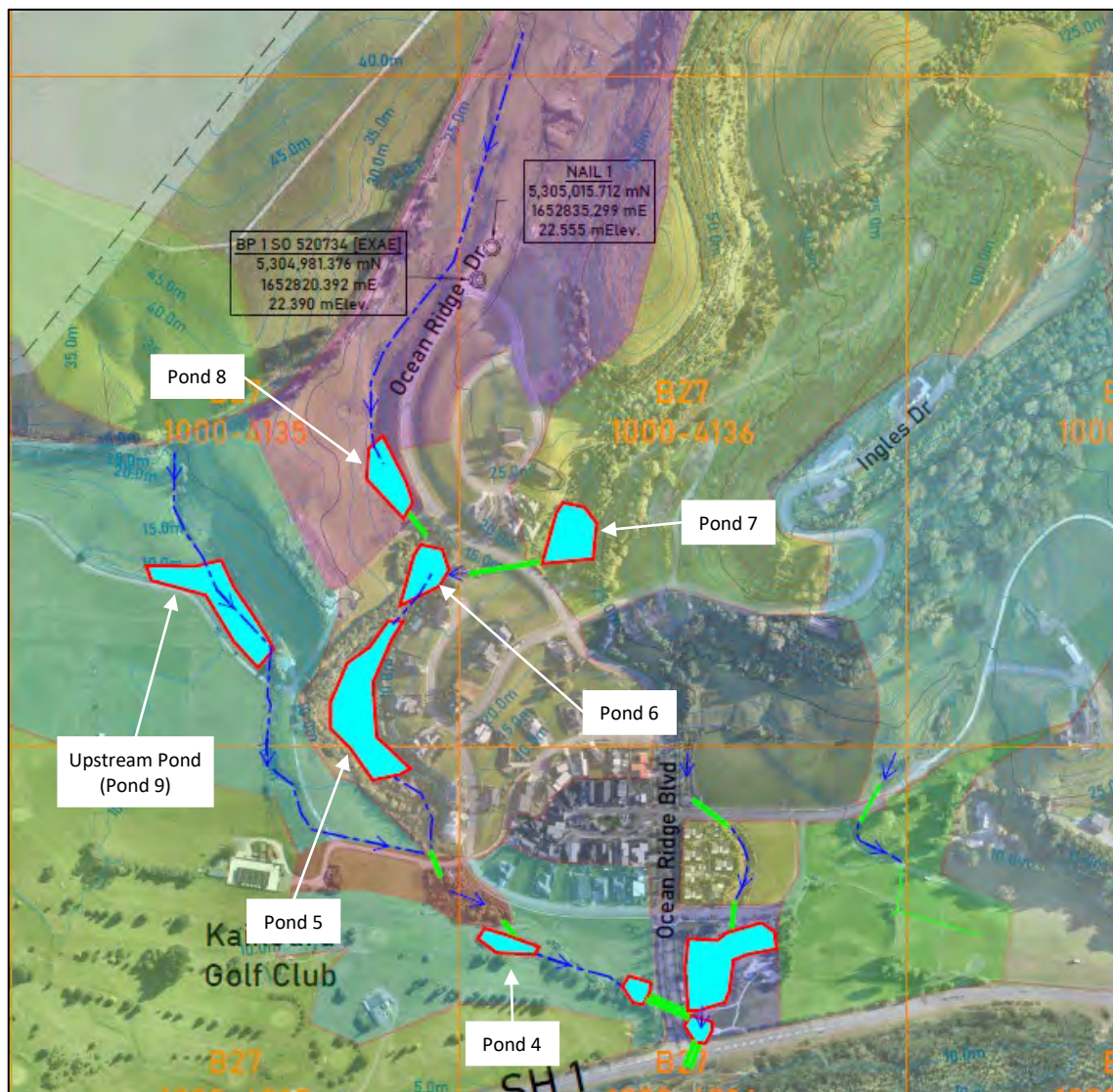


Figure 7: Stormwater Drainage Network Showing Ponds 9 to 4



Figure 8: Pond 6 Inlet with Dense Vegetation Downstream of Inlet of Pond 8



Figure 9: Pond 6 900NB RCRRJ Inlet from Pond 7



Figure 10: Pond 6 225 PVC Outlet Connection to Pond 5 Under Access Track



Figure 11: Pond 6 Outlet Connection to Pond 5 (with Additional Pipelines)

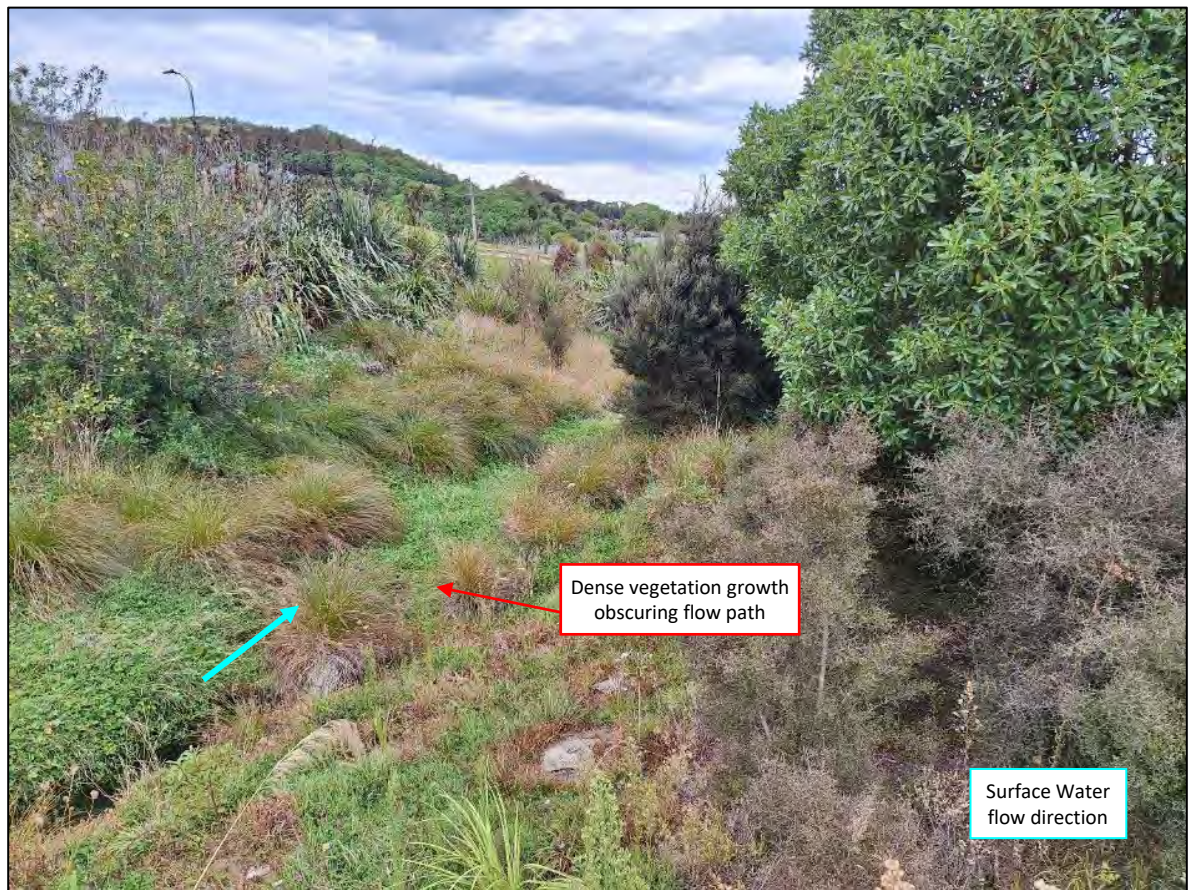


Figure 12: Pond 6 Channel Connection to Pond 5 Showing Vegetation Growth



Figure 13: Pond 5

Pond 5 had a slow moving flow through to Pond 4 at the time of the site investigation.

Water from Pond 5 to Pond 4 via an open channel as shown in Figure 14.



Figure 14: Open Channel from Pond 4 to Pond 3



Figure 15: Pond 3

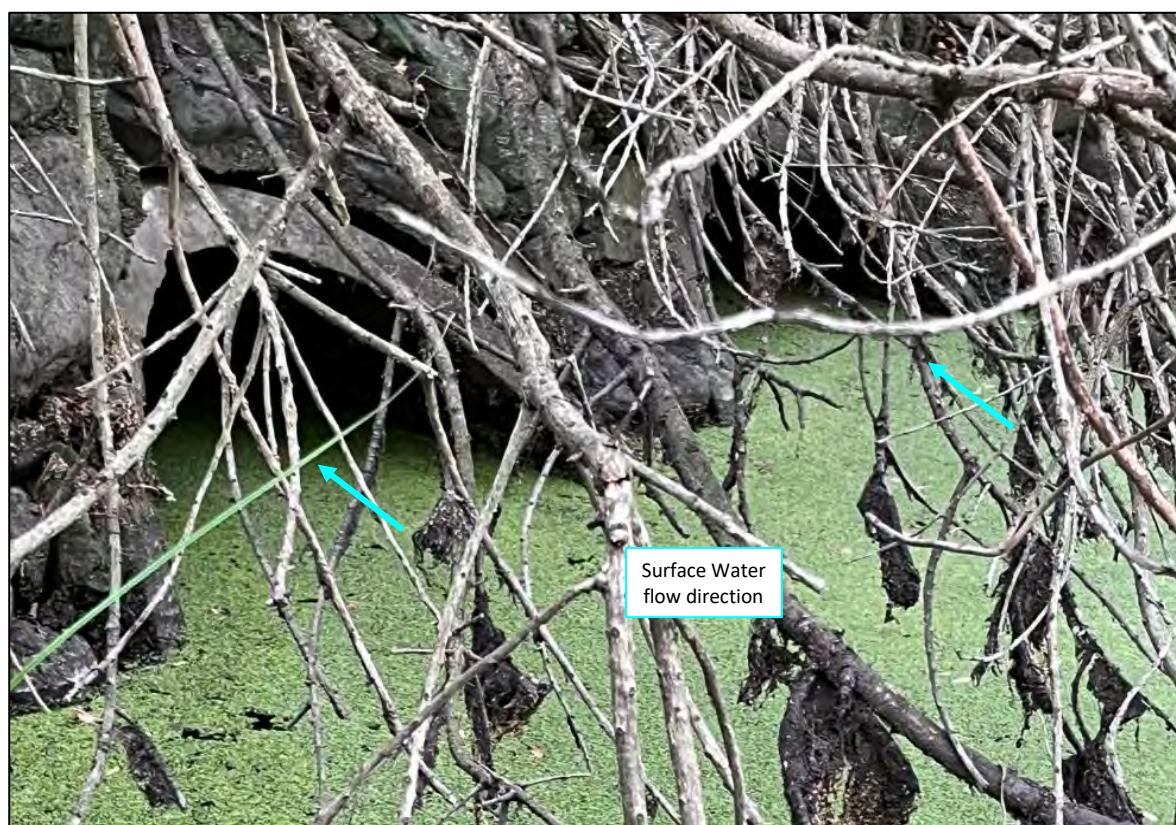


Figure 16: Pond 3 Twin 900NB RCRRJ Culvert Pipe Outlets (to Pond 1)

Twin 900NB RCRRJ culvert pipes situated underneath Ocean Ridge Boulevard carry stormwater runoff from Pond 3 to Pond 1. Both pipes were be partially submerged at the time of the investigation. Pond 1 inlets were also submerged at the time of the investigation. The partially submerged pipelines were observed on multiple visits. Historic design drawings denote these ponds as wet ponds.



Figure 17: Twin 900NB RCRRJ Culvert Pipe Inlets to Pond 1 (from Pond 3)



Figure 18: Pond 1 Outlet to Twin 900NB RCRRJ Culvert Pipelines Crossing SH1

Pond 1 was partially dry at the time of the site investigation. Upstream Pond 3 900NB RCRRJ outlet pipes were partially submerged, however the inlet to Pond 1 had no surface water present.



Figure 19: Pond 1 Outlet - Twin 900NB RCRRJ Culvert Pipes Crossing SH1

The invert level of the twin 900NB RCRRJ culvert pipes crossing Ocean Ridge Boulevard is 220mm lower than the invert of the twin-pipe crossing SH1. The invert level for the twin 900NB RCRRJ culvert pipes crossing SH1 is 0.4m higher than that shown on the as-built drawings.



Figure 20: Pond 1 twin 900NB RCRRJ culvert pipe outlets crossing SH1 (closer look)



Figure 21: Twin 900NB RCRRJ culvert pipe south of SH1

Stormwater flows discharging from the site to the south side of SH1 are directed by an (expected) overgrown channel, east alongside the highway, and under a timber footbridge, prior to flowing to the sea.

It is understood from local knowledge, that smaller flows generally infiltrate to ground, and that only major stormwater flows discharge directly to the sea.

The exact extent and geometry of the expected channel was unable to be confirmed due to dense vegetation growth obscuring access to, and views of, the channel.

The extent and flow direction of the channel is generally determined from LiDAR contours and the observed topography generally falling south and east of SH1.

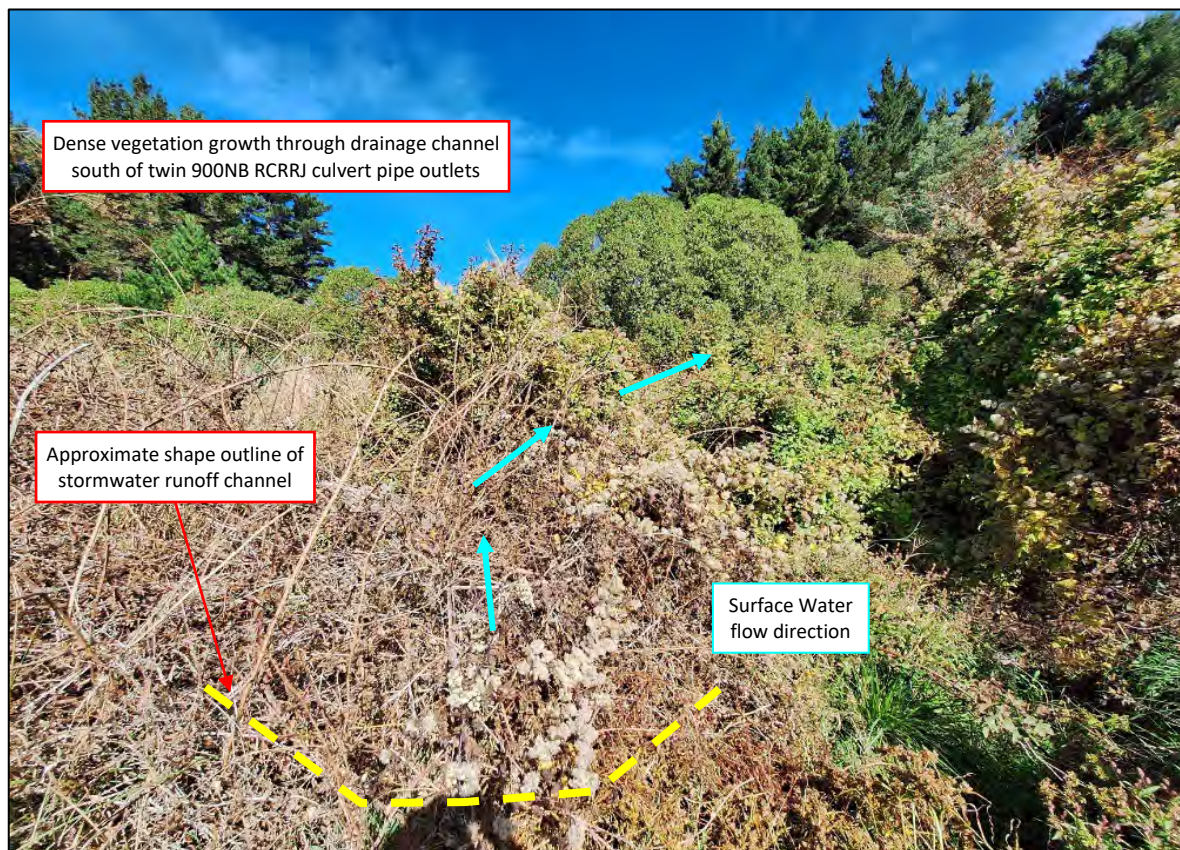


Figure 22: Drainage Channel South of Twin 900NB RCRRJ Culvert Pipes Heading Eastwards



Figure 23: Bridge Crossing Flow Path/Channel from Twin 900NB and 450NB RCRRJ Culvert Pipes

2.3 MID-CATCHMENT

The mid-catchment area of Ocean Ridge drains to Pond 2 and Pond 1, and discharges off-site to the south side of SH1 via the twin 900NB RCRRJ culvert pipe outlets via a combination of swales, channels and connecting stormwater pipes.

This catchment includes an overflow dry basin east, and upstream of Pond 2, which has connecting pipelines from upstream culverts crossing Ludley Drive.

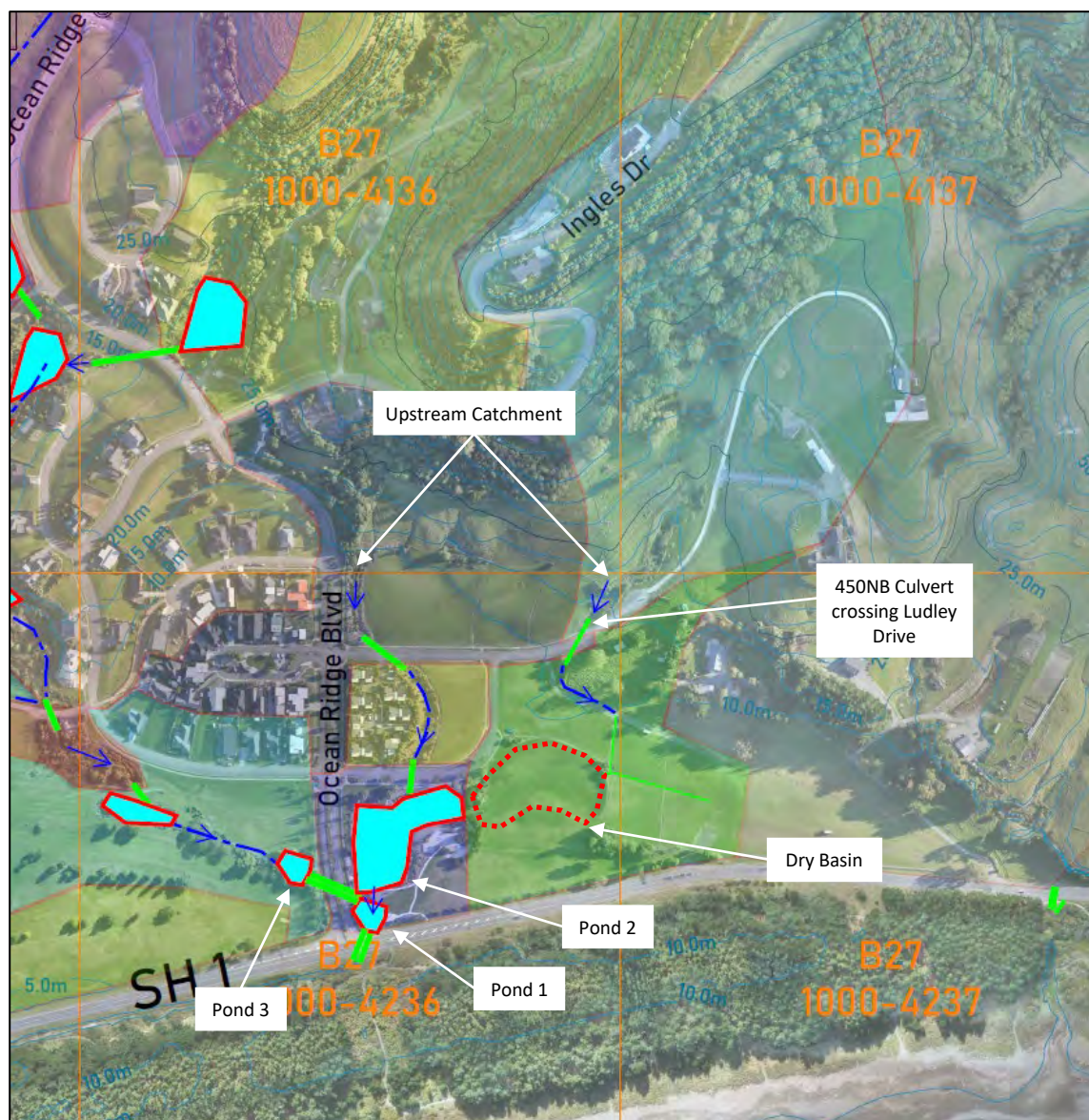


Figure 24: Stormwater drainage network showing Ponds 1 to 3 locations



Figure 25: 500NB RCRRJ Culvert Pipe Connecting to Wetland



Figure 26: Discharge Channel from Wetland (Adjacent to Fidele Lane)



Figure 27: Discharge from wetland to Dry Basin (Adjacent to Fidele Lane)



Figure 28: Upstream Catchment Flow Through 450NB then 900NB RCRRJ Culvert Pipes to Pond 2

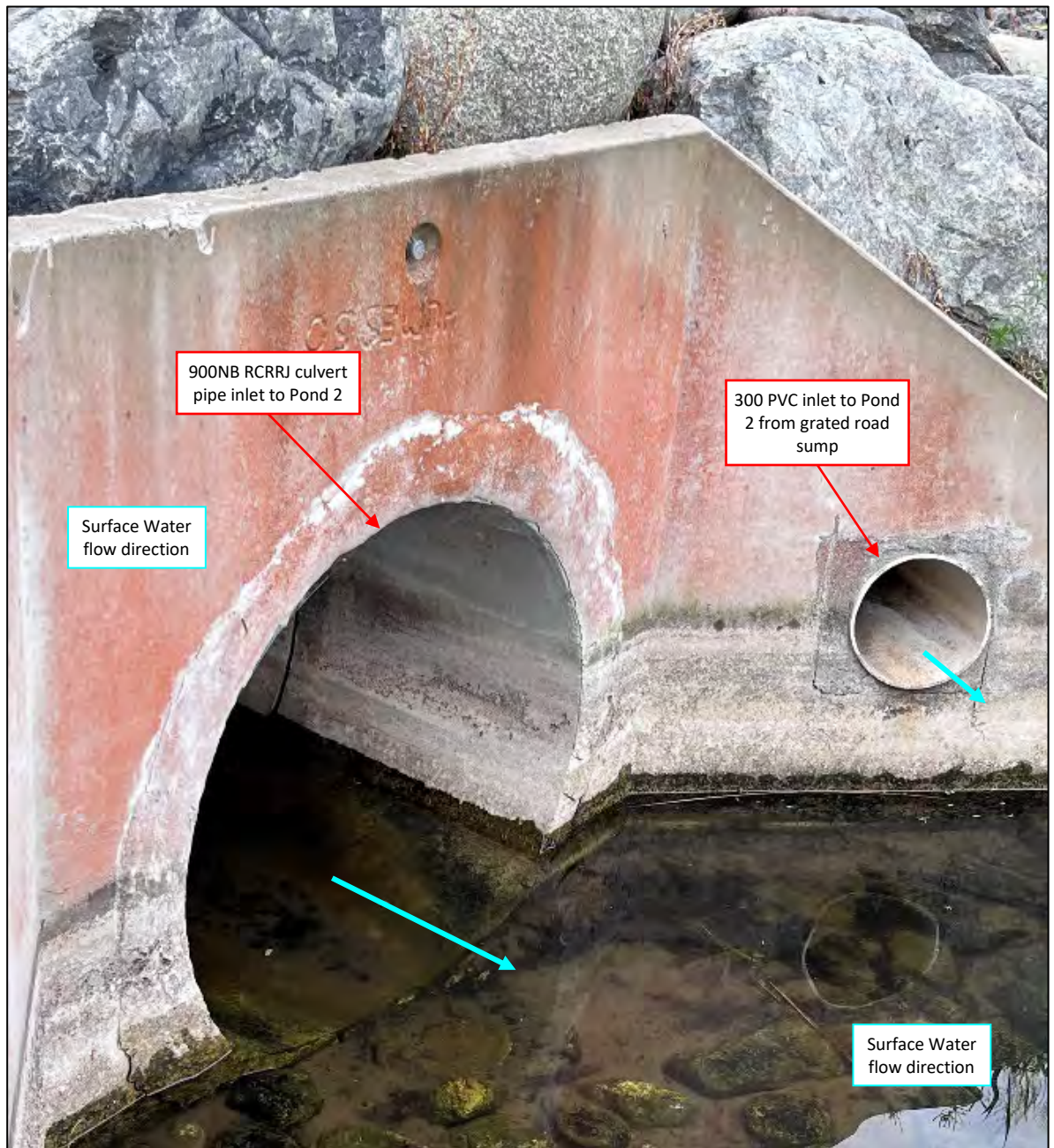


Figure 29: Pond 2 Inlet Through 900NB RCRRJ and 300 OD PVC Pipelines



Figure 30: Pond 2 Outlet Through Concrete Box Weir



Figure 31: Pond 2 Outlet Through Concrete Box Weir Flowing to Pond 1

2.4 EASTERN CATCHMENT ADJACENT TO OCEAN RIDGE SUBDIVISION

The eastern catchment area, adjacent to the Ocean Ridge Subdivision, drains to a 900NB RCRRJ culvert pipe, with a 300NB RCRRJ overflow pipe, laid below SH1, and discharges stormwater runoff towards the sea.

A small wet pond was observed at the outlet of the two pipelines during the site visits.

A 450NB RCRRJ pipeline was observed to discharge to the same pond, however, the pipeline inlet was not observed.

Surface Water flow direction

450NB RCRRJ Overflow Pipe

900NB RCRRJ Culvert Pipe

J:\ CH Series\CH01912 - Ocean Ridge\Civil\Reports\2. SW system site investigation\Ocean Ridge Stormwater System Site Investigation - JDK AM 240514.docx



Figure 34: Stormwater Flowing to Sea, and Crossing Access Track

APPENDIX B

INFILTRATION TEST

MEMORANDUM / FILENOTE

Date: 6 August 2024

Project No. CH01912

To: Justin Adamson and Son Nguyen

Subject: Results of ground infiltration testing at Ocean Ridge

From: Sam Gladwin

As requested, Fraser Thomas has undertaken ground infiltration testing as part of the assessment of the proposed stormwater management system for the Ocean Ridge Subdivision, Kaikoura. Then testing was undertaken on 22 July 2024.

It is understood that there had been light rainfall overnight prior to the site visit.

Potential Infiltration Basin Testing

A single machine excavated test pit, numbered TP1, measuring approximately 0.9 m long by 0.7 m wide, was excavated to a depth of approximately 1.5 m below the existing ground surface, generally at the location of a potential infiltration basin. The test pit location is shown on the appended marked-up plan.

It is understood that the potential infiltration basin will likely be founded at a depth of approximately 1.5 m below the existing ground surface.

A surficial layer of topsoil, generally comprising silty sands, was encountered at the location of TP1, to a depth of approximately 0.3 m below the existing ground surface.

The surficial topsoil was generally underlain by gravelly sands, inferred to be beach deposits of Holocene age. These sediments were encountered to the extent of TP1.

Groundwater was not encountered during the investigation reported herein. A review of the New Zealand Geotechnical Database and Canterbury Maps, for borehole logs in the vicinity of the site, did not provide any reliable indication of the depth to groundwater in the vicinity of the potential new infiltration basin location.

The log of Test Pit TP1 is appended to this memo.

Ground infiltration testing was undertaken at the location of TP1, generally using the methods suggested by E1/VM1, in order to determine the infiltration rate for the gravelly sands.

Due to the flow rate from a Kaikoura District Council (KDC) hydrant being too high (approximately 19L/sec) to maintain a constant head, a falling head test was performed at the location of TP1. The results of the testing are presented in Table 1.

TABLE 1: SOIL INFILTRATION TESTING RESULTS- TP1

Test Pit	Depth (mbgl)	Type of Infiltration Test	Infiltration Rate (mm/hr)	Assumed Groundwater Level (mbgl)
TP1	1.5	Falling Head Test	1,500	>1.5m ⁽¹⁾

1. Groundwater was not encountered in the shallow test pit put down at the site. No reliable information is currently available to determine accurate groundwater depth.

Surface Infiltration Testing

Four double-ring infiltration tests were undertaken at the ground surface, at the locations of DR1, DR2, DR3 and DR4, shown on the appended marked-up plan, in order to test the infiltration characteristics of the surficial soils for flood modelling purposes.

Tests DR1, DR2 and DR3 were undertaken from the existing ground surface.

Test DR4 was undertaken from the base of an existing drain. The invert of the drain, at the location of DR4, was approximately 1.3 m below the surrounding ground surface.

Shallow hand augers were put down next to test locations DR1, DR2 and DR3 (to depths ranging between approximately 0.3 m to 1.7 m below the existing ground surface), to determine the nature of the subsoil conditions in these areas.

Due to the ground conditions encountered, the hand auger was unable to be progressed at the location of DR4, so a shallow hand dug test was excavated.

Surficial layers of topsoil and topsoil/fill, generally comprising silty gravelly sands and sandy silts were encountered to depths ranging between approximately 0.3 m and 0.4 m below the existing ground surface, at the locations of Tests DR1, DR2, DR3 and DR4.

The surficial layers of topsoil and topsoil/fill were generally underlain by sandy silty gravels inferred to be either subdivisional bulk fill (DR1), or natural sandy gravels and sandy silts, inferred to be alluvial sediments (DR2 and DR3). At the location of DR4, material comprising silty sandy gravels was encountered. It is not reliably known if this is natural ground or localised fill material.

Groundwater was not encountered at the locations of Tests DR1, DR2 and DR3, during the investigation reported herein. However, information presented in a previous geotechnical report, dated June 2005, prepared by URS for the Ocean Ridge Development, indicates that the depth to groundwater is likely to range between 1.3 m and 2.0 m below the existing ground surface, in the vicinity of Test DR2.

At the location of Test DR4, standing groundwater was encountered at a depth of approximately 0.15 m below the base of an existing drain, which corresponds to a depth of approximately 1.45 m below the surrounding ground surface.

The logs of the subsurface conditions at the locations of DR1, DR2, DR3 and DR4 are appended to this memo.

Double-ring infiltration testing was undertaken generally using the method suggested in Appendix 6.2 of Christchurch City Council's "Waterways, Wetlands and Drainage Guide", at the locations of DR1, DR2, DR3 and DR4.

The results of the testing are presented in Table 2.

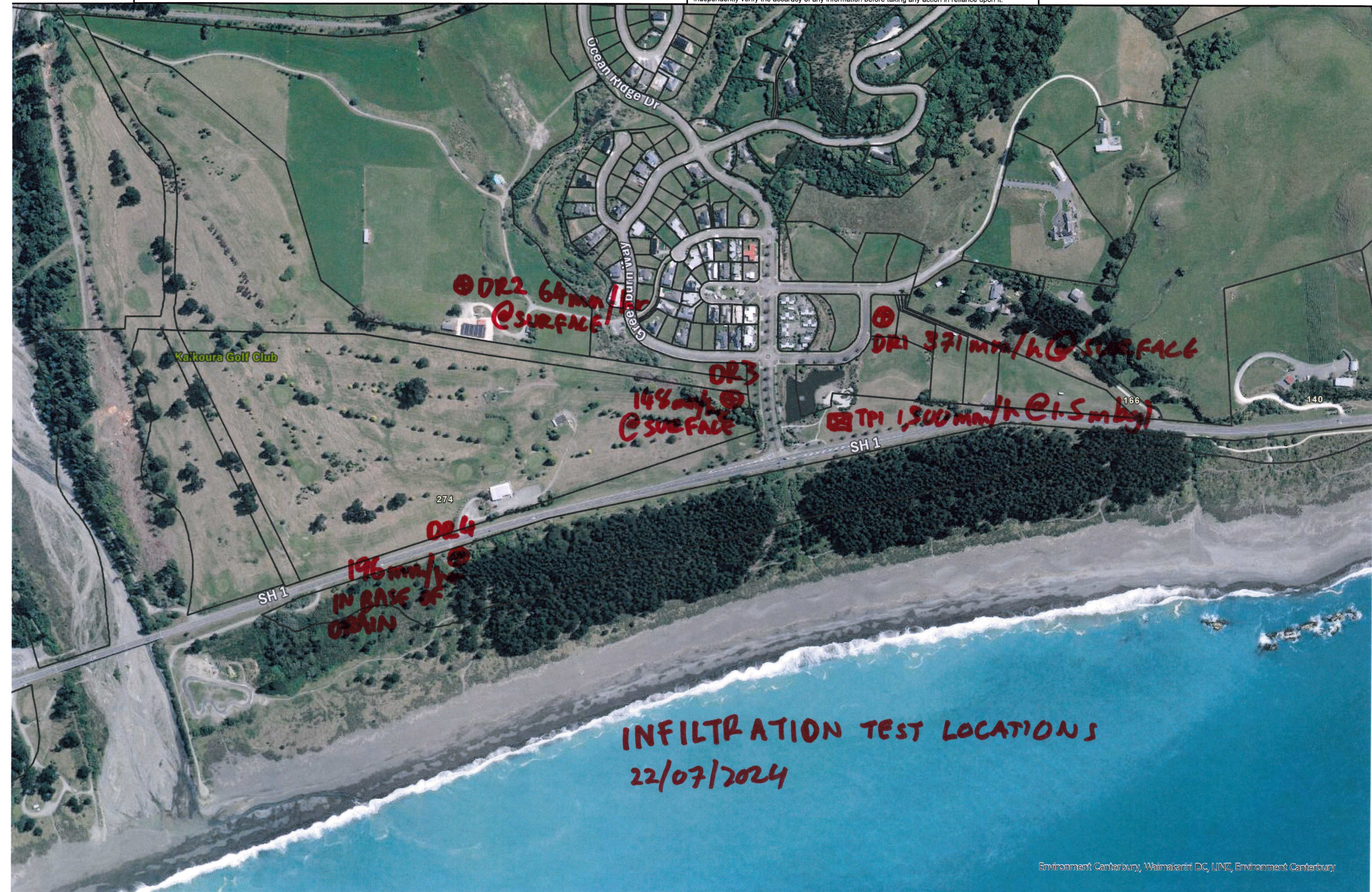
TABLE 2: DOUBLE-RING INFILTRATION TESTING RESULTS

Test ID	Depth (mbgl)	Type of Infiltration Test	Infiltration Rate (mm/hr)	Assumed Groundwater Level (mbgl)
DR1	Ground Surface	Double-Ring	371	>1.5 ⁽¹⁾
DR2	Ground Surface	Double-Ring	64	1.3 – 2.0 ⁽²⁾
DR3	Ground Surface	Double-Ring	148	>1.7 ⁽¹⁾
DR4	Ground Surface	Double-Ring	196	0.15 (1.45) ⁽³⁾

1. Groundwater was not encountered in the shallow hand augers put down next to the double-ring test locations DR1 and DR3. No reliable information is currently available to determine accurate groundwater depth.
2. The “assumed” groundwater level has been inferred from information presented in a previous geotechnical report, dated June 2005 prepared by URS for the Ocean Ridge Development.
3. Depth to groundwater below base of existing drain, based on the results of a shallow hand dug test pit put down next to DR4.

This memorandum only provides the results of soil infiltration testing, at the locations of TP1, DR1, DR2, DR3 and DR4, and does not relate to any other geotechnical aspects of the site.

Site Plan Mark-Up



Test Pit and Hand Auger Logs



**Fraser
Thomas**

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HAND AUGER LOG

Hole No:

DR1

Project No:
CH01912

Project: **Cargill Station Ltd
Ocean Ridge, Kaikoura**

Shear Vane:

Date Drilled:
22/07/2024

Logged By:
SG

Checked By:

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)		Depth (m)	Dynamic Cone Penetrometer											Groundwater
				Shear Vane	Residual Shear Vane		Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 100 mm)											
				50	100	150	200	Values	2	4	6	8	10	12	14	16		
0.2	SAND, silty, gravelly, black, moist [TOPSOIL/FILL]	Fill															GWNE	
0.4	GRAVEL (fine, subrounded), sandy, silty, brown, moist [INFERRED BULK FILL]																	
0.6	EOH: 0.60 m TARGET DEPTH																	
0.8																		
1.0																		
1.2																		
1.4																		
1.6																		
1.8																		
2.0																		
2.2																		
2.4																		
2.6																		
2.8																		
3.0																		
3.2																		
3.4																		
3.6																		
3.8																		
4.0																		
4.2																		
4.4																		
4.6																		
4.8																		
Remarks: 1. Hand auger put down next to location of double ring infiltrometer test. 2. Groundwater not encountered on 22/07/2024									Datum:									
									Coordinates:									



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HAND AUGER LOG

Hole No:
DR2

Project No: CH01912	Project: Cargill Station Ltd Ocean Ridge, Kaikoura	Shear Vane:	Date Drilled: 22/07/2024	Logged By: SG	Checked By:
-------------------------------	---	-------------	-----------------------------	------------------	-------------

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)		Depth (m)	Dynamic Cone Penetrometer										Groundwater
				Shear Vane	Residual Shear Vane		Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 100 mm)										
				50	100	150	200	Values	2	4	6	8	10	12	14	16	
0.2	SILT, sandy, dark brown, moist [TOPSOIL]	T/S															GWNE
0.4	EOH: 0.30 m TOO DIFFICULT TOO AUGER - GRAVELS ENCOUNTERED																
0.6																	
0.8																	
1.0																	
1.2																	
1.4																	
1.6																	
1.8																	
2.0																	
2.2																	
2.4																	
2.6																	
2.8																	
3.0																	
3.2																	
3.4																	
3.6																	
3.8																	
4.0																	
4.2																	
4.4																	
4.6																	
4.8																	
Remarks: 1. Hand auger put down next to location of double ring infiltrometer test. 2. Groundwater not encountered on 22/07/2024									Datum:								
									Coordinates:								



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HAND AUGER LOG

Hole No:

DR3

Project No:
CH01912

Project: Cargill Station Ltd
Ocean Ridge, Kaikoura

Shear Vane:

Date Drilled:
22/07/2024

Logged By:
SG

Checked By:

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)		Values	Depth (m)	Dynamic Cone Penetrometer									Groundwater	
				Shear Vane	Residual Shear Vane			Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 100 mm)										
				50	100	150	200		2	4	6	8	10	12	14	16		
0.2	SILT, sandy, dark brown, moist [TOPSOIL]	T/S																
0.4	SILT, sandy, trace gravels (medium, subrounded), brown, moist [ALLUVIAL SEDIMENTS]	Holocene Alluvial Deposits																
0.6																		
0.8	SILT, sandy, light brown, moist																	
1.0																		
1.2																		
1.4																		
1.6																		
1.8	EOH: 1.70 m TOO DIFFICULT TO AUGER - POOR RECOVERY																	
2.0																		
2.2																		
2.4																		
2.6																		
2.8																		
3.0																		
3.2																		
3.4																		
3.6																		
3.8																		
4.0																		
4.2																		
4.4																		
4.6																		
4.8																		

Remarks:

1. Hand auger put down next to location of double ring infiltrometer test.

2. Groundwater not encountered on 22/07/2024

Datum:

Coordinates:



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TEST PIT LOG

Hole No:
TP1

Project No: CH01912	Project: Cargill Station Ltd Ocean Ridge, Kaikoura	Shear Vane:	Date Excavated: 22/07/2024	Logged By: SG	Checked By:
------------------------	---	-------------	-------------------------------	------------------	-------------

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)					Depth (m)	Dynamic Cone Penetrometer								Groundwater	
				Vane readings corrected as per BS 1377						Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm)									
				● Shear Vane	○ Residual Shear Vane														
				50	100	150	200	Values		2	4	6	8	10	12	14	16		
0.2	SAND, silty, dark brown, moist, rootlets [TOPSOIL]	T/S							0.2									GWNE	
0.4	SAND (medium to coarse), gravelly (fine to medium, subrounded), brown, moist [INFERRED BEACH DEPOSITS]	Holocene Beach Deposits							0.4										
0.6																			
0.8																			
1.0																			
1.2																			
1.4									1.4										
1.6	EOTP: 1.50 m TARGET DEPTH								1.6										
1.8									1.8										
2.0									2.0										
2.2									2.2										
2.4									2.4										
2.6									2.6										
2.8									2.8										
3.0									3.0										
3.2									3.2										
3.4									3.4										
3.6									3.6										
3.8									3.8										
4.0									4.0										
4.2									4.2										
4.4									4.4										
4.6									4.6										
4.8									4.8										

Profile:	Excavation Method:
 	Remarks: 1. Hand auger put down next to location of double ring infiltrometer test. 2. Groundwater not encountered on 22/07/2024.
	Datum:
	Coordinates:

TEST PIT LOG

Hole No:
DR4

Project No:
CH01912

Project: Cargill Station Ltd
Ocean Ridge, Kaikoura

Shear Vane:

Date Excavated:
22/07/2024

Logged By:
SG

Checked By:

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)		Depth (m)	Dynamic Cone Penetrometer										Groundwater
				Vane readings corrected as per BS 1377			Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm)										
				● Shear Vane	○ Residual Shear Vane		2	4	6	8	10	12	14	16			
0.2	SILT, sandy, dark brown, saturated, roots and leaves [TOPSOIL/DETRITUS]	T/S				0.2											▼ 22/07/2024
0.4	EOTP: 0.40 m TOO DIFFICULT TO EXCAVATE HAND DUG TEST PIT - SANDY SILTY GRAVELS ENCOUNTERED					0.4											
0.6						0.6											
0.8						0.8											
1.0						1.0											
1.2						1.2											
1.4						1.4											
1.6						1.6											
1.8						1.8											
2.0						2.0											
2.2						2.2											
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3.2						3.2											
3.4						3.4											
3.6						3.6											
3.8						3.8											
4.0						4.0											
4.2						4.2											
4.4						4.4											
4.6						4.6											
4.8						4.8											

Profile:



Excavation Method:

Remarks:
1. Hand dug test pit put down next to location of double ring infiltration test in base of drain running parallel to and south of SH1.
2. Base of drain is approximately 1.3m below surrounding ground surface.
3. Standing groundwater was encountered at a depth of approximately 0.15m below the base of drain on 22/07/2024.

Datum:

Coordinates:

Site Photos



P1: TP1 after completion of infiltration test.



P2: View looking east across potential infiltration basin. TP1 in middle ground.

Site Walkover Photographs – July 2024



P3: Double-ring test DR1 located approximately in centre of this lot.



P4: DR1 topsoil/fill.



P5: Double-ring test DR3 was located approximately in centre of this photo.



P6: Double-ring test DR4 and hand dug test pit located in base of open drain.



P7: View of side wall of open drain showing sandy silty gravels.



P8: View of open drain south-west of DR3. This section of open drain is not as well formed and has been used as a four wheel drive track, note the standing water in ruts. This section terminates at walking track that runs along the tops of the sand dunes to the south, and does not appear to outlet to the ocean.



P9: View of beach showing clean sand beach deposits, which were also encountered in the dunes along with some gravelly sands similar in nature to those encountered in TP1



P10: View of open drain south-west of twin 900 outlet. A hand dug test pit was also excavated here, which found topsoil.



P11 View of topsoil in hand dug test pit from P9.



P12: View of twin 900 outlets at eastern end of open drain.



P13: View of open drain just down stream of outlets showing standing surface water.



P14: View of weir located south of 21 Greenburn Way.

APPENDIX C

MODELLING MAPS

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Legend

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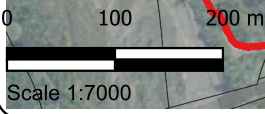
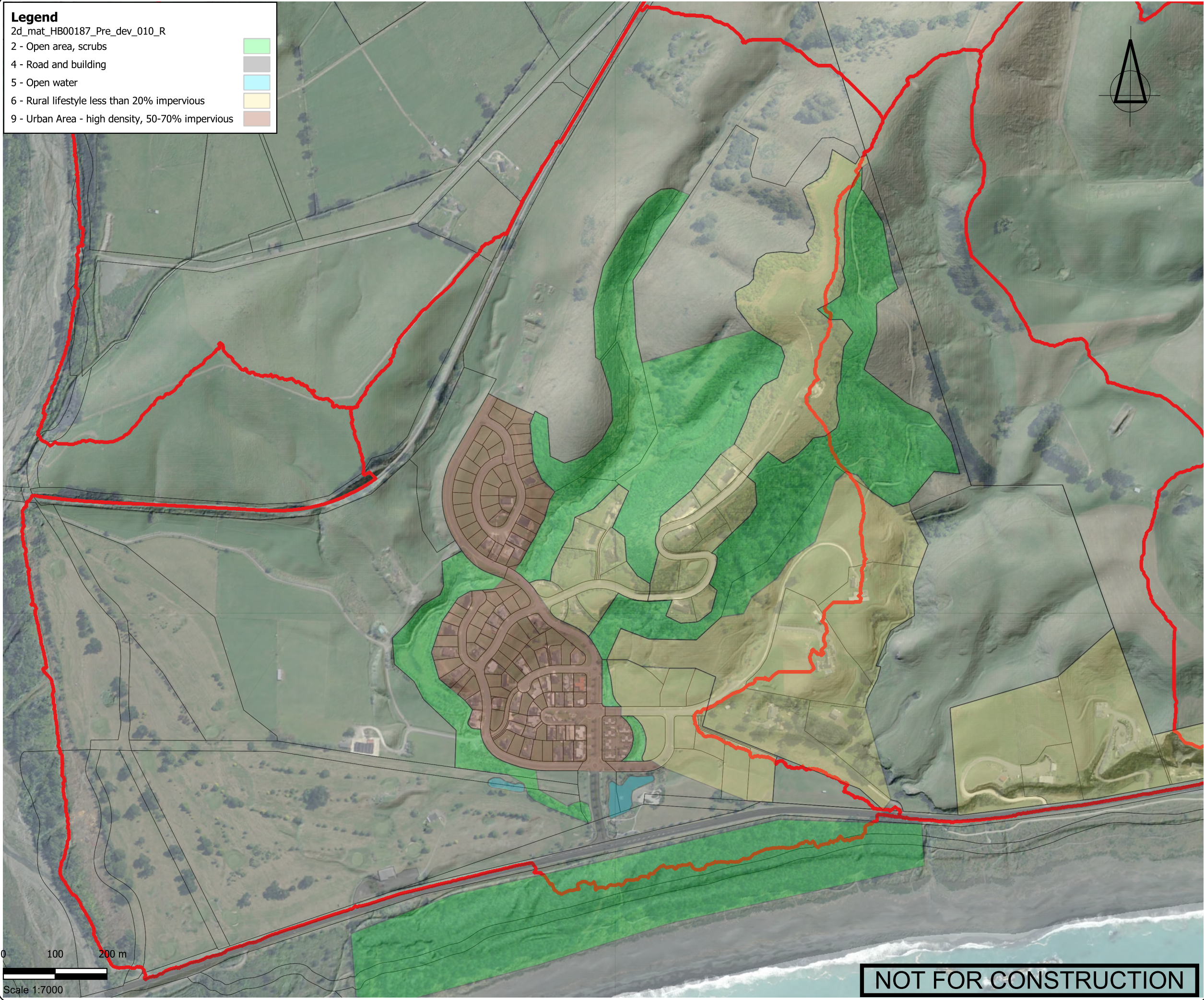
2 - Open area, scrubs

4 - Road and building

5 - Open water

6 - Rural lifestyle less than 20% impervious

9 - Urban Area - high density, 50-70% impervious



NOT FOR CONSTRUCTION

SURVEYED			APPROVED	DATE
DESIGNED	SN	03/06/2025		
DRAWN	SN	03/06/2025		
CHECKED				
REVISION	CHANGES		CHECKED	DATE

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SCALE 1:7000	A3	
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Legend

2d_mat_HB00187_PC_013_R

2 - Open area, scrubs

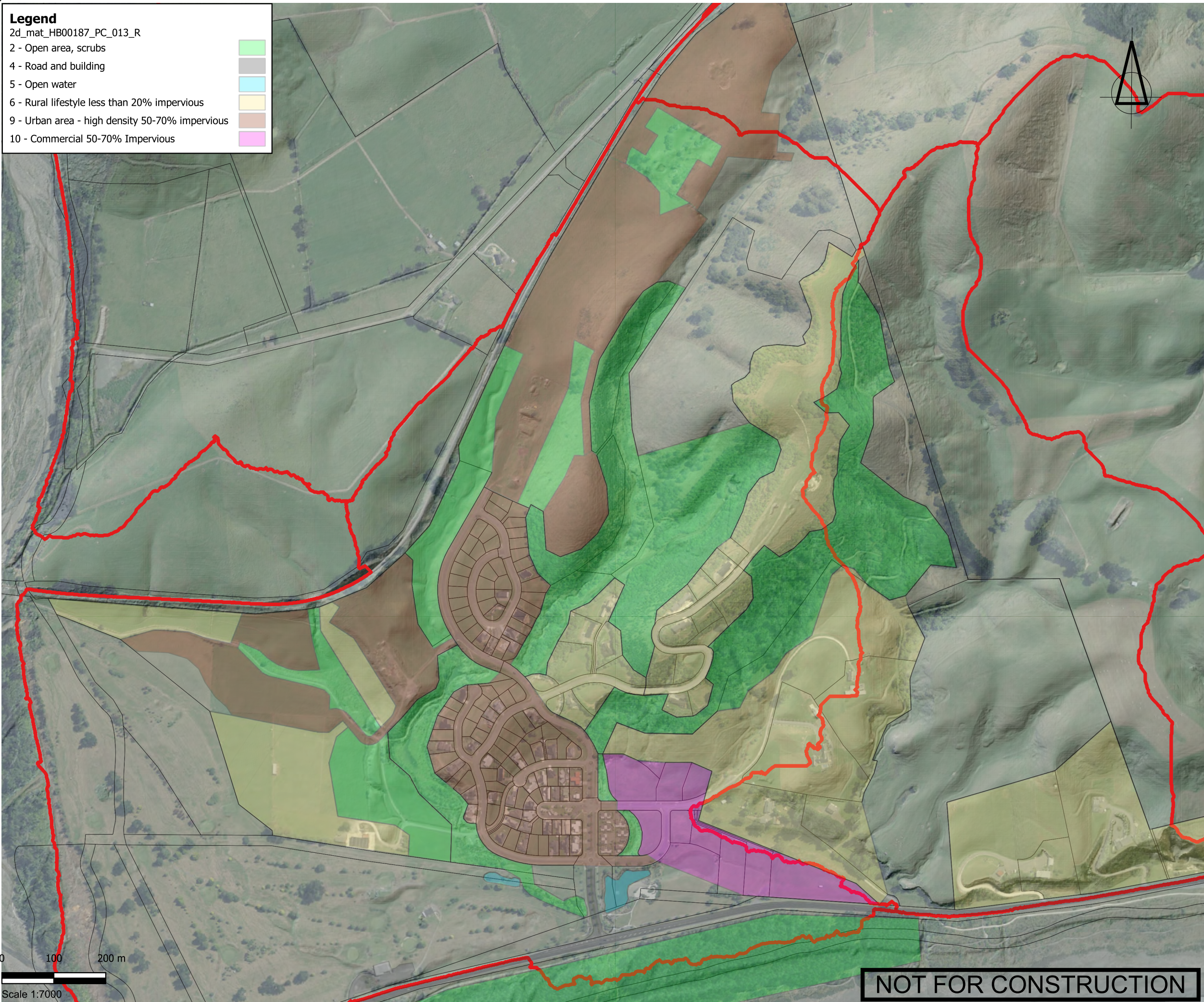
4 - Road and building

5 - Open water

6 - Rural lifestyle less than 20% impervious

9 - Urban area - high density 50-70% impervious

10 - Commercial 50-70% Impervious



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Legend

Results

HB00187_010_10yr_d_HR_Max

Band 1 (Gray)

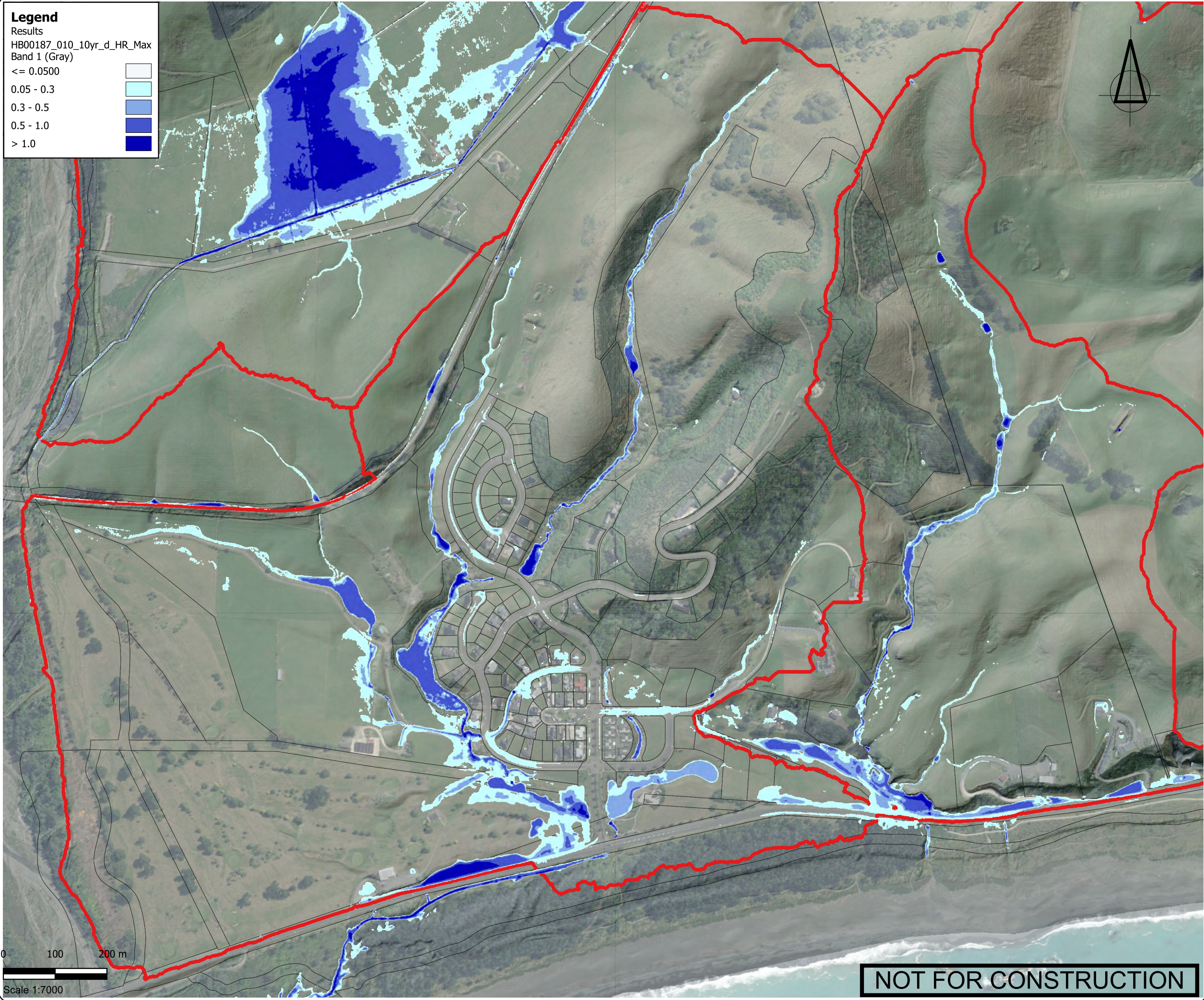
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0.5 - 1.0

> 1.0



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CHECKED				
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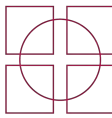
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HB00187-PC-003		

Legend

Results

HB00187_010_100yr_d_HR_Max

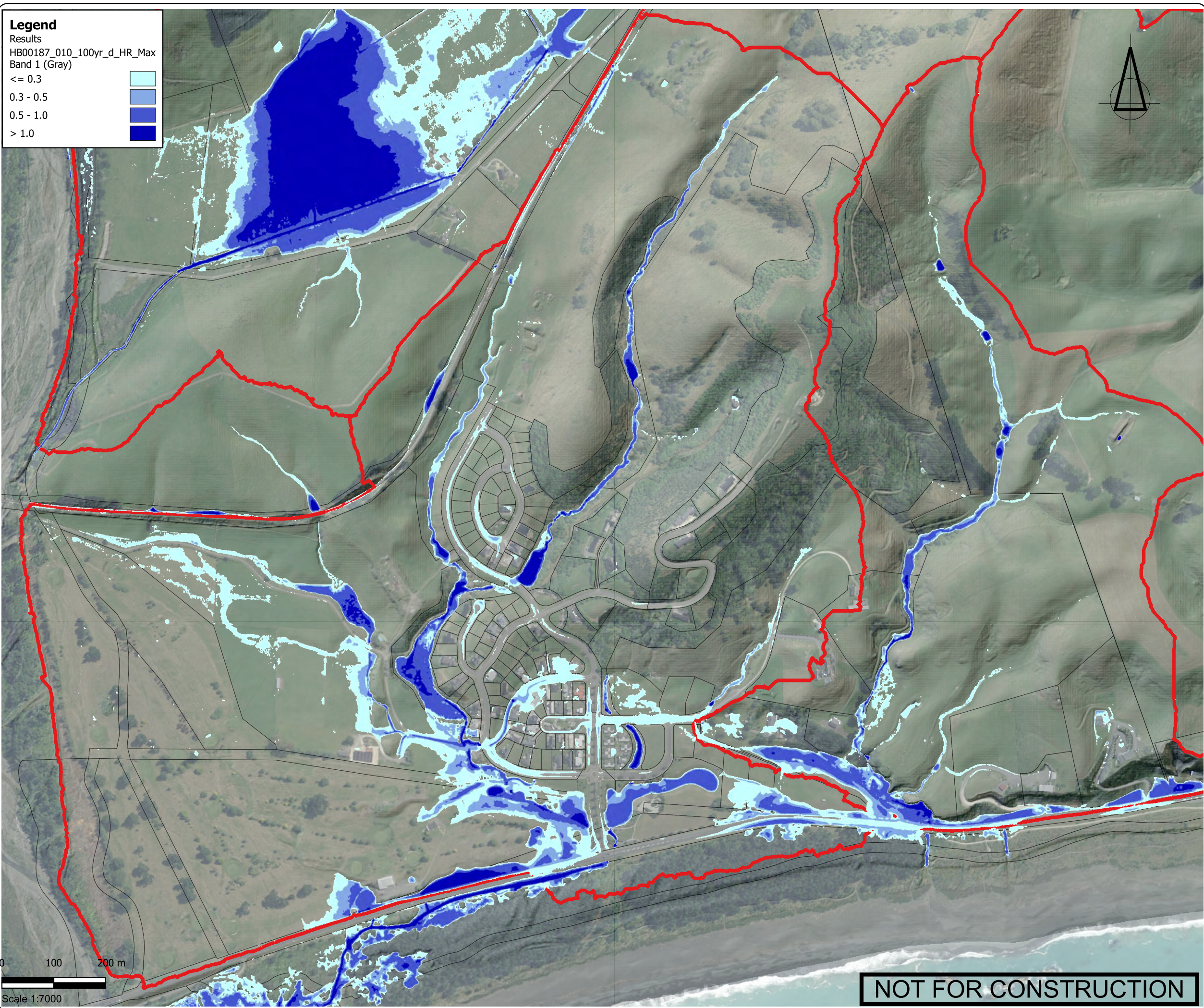
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<= 0.3

0.3 - 0.5

0.5 - 1.0

> 1.0



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DESIGNED	SN	03/06/2025		
DRAWN	SN	03/06/2025		
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DRAWING No HB00187-PC-004	REVISION

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Legend

Results

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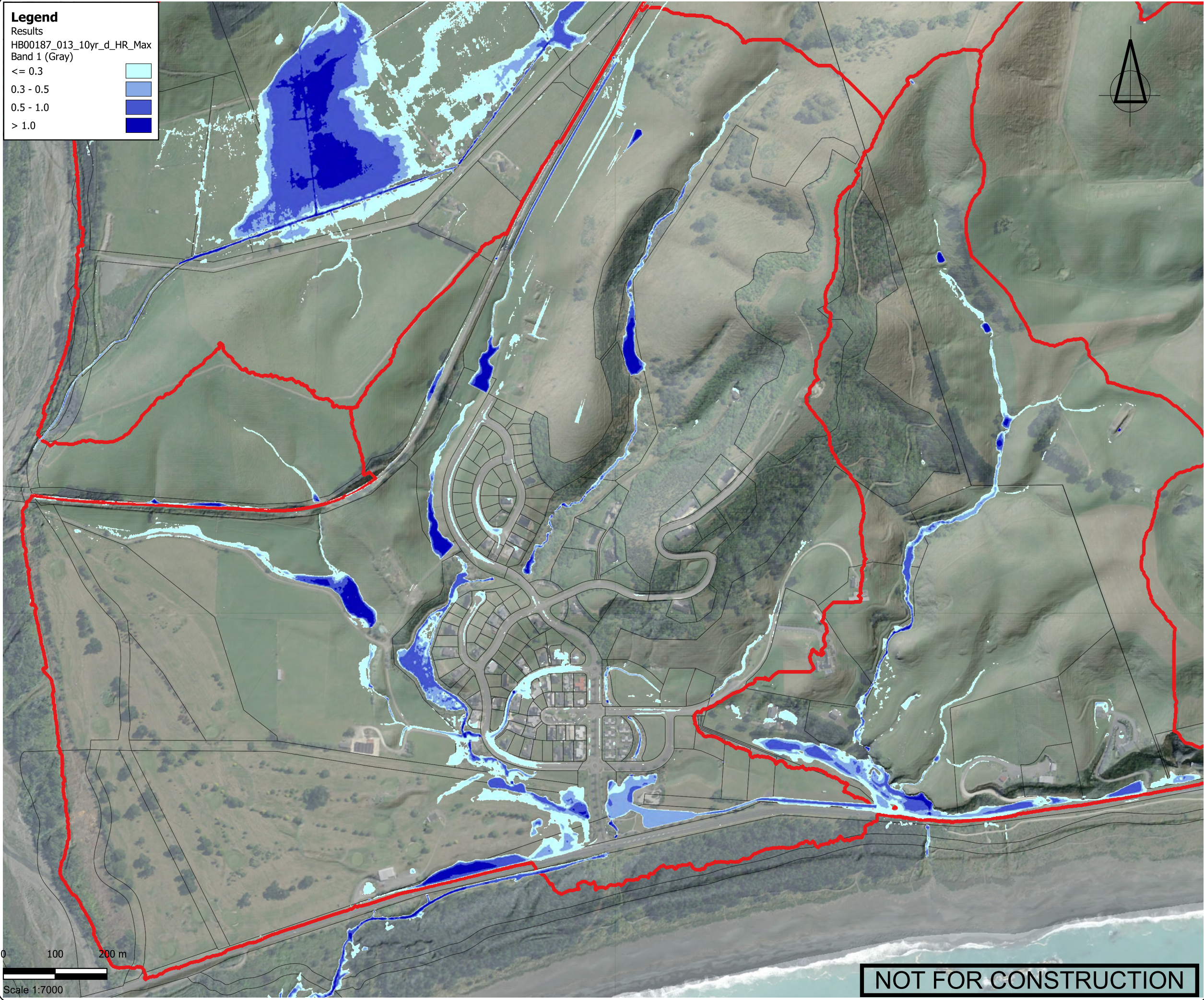
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> 1.0



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KAIKOURA

TITLE

POST-DEVELOPMENT
10 YEAR FLOOD EXTENT

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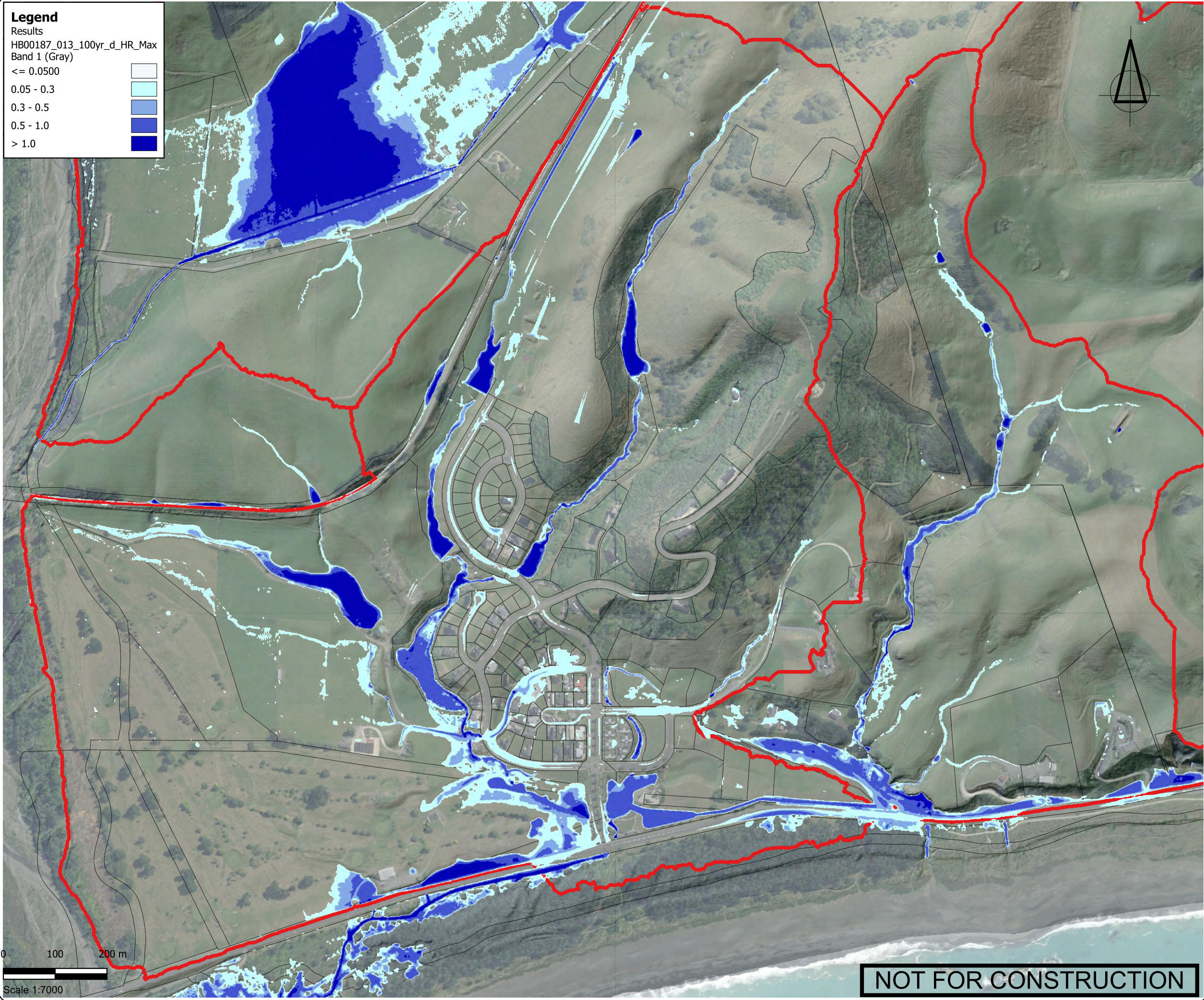
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Legend

Results

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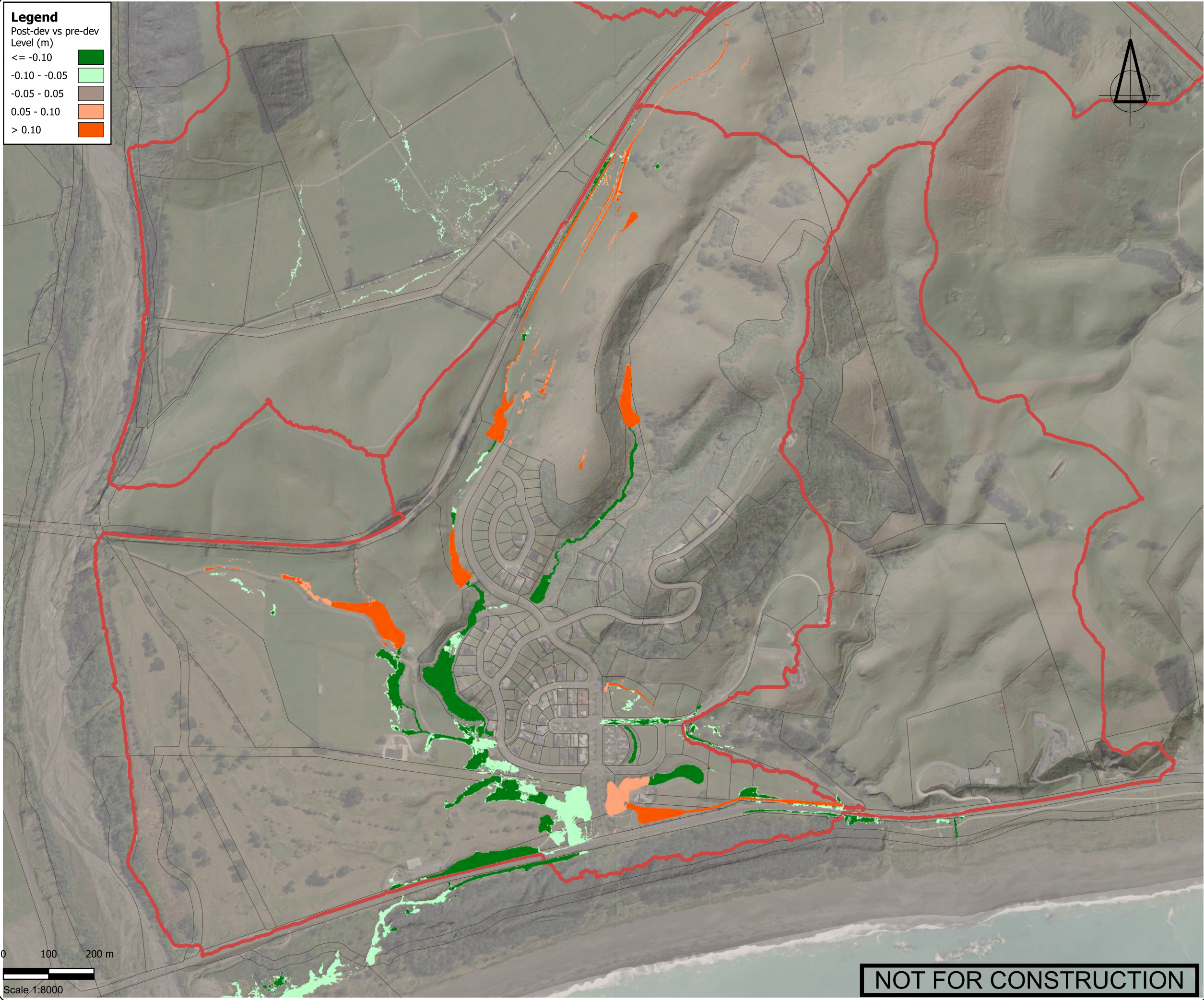
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Legend
Post-dev vs pre-dev
Level (m)

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0.05 - 0.10	
> 0.10	

0 100 200 m

Scale 1:8000

NOT FOR CONSTRUCTION

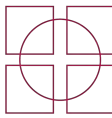
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DIFFERENCE

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Legend

Post-dev vs Pre-dev
Level (m)

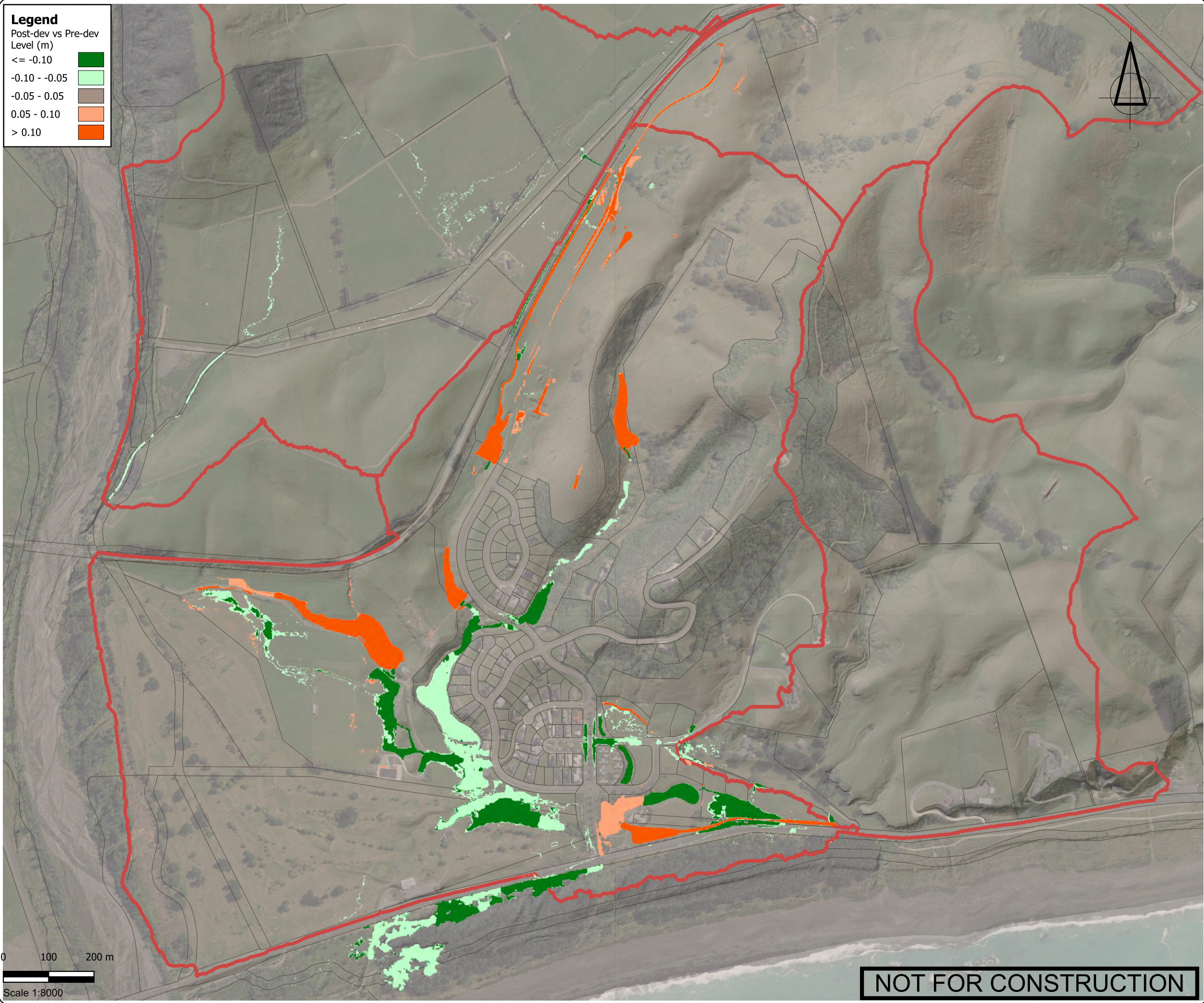
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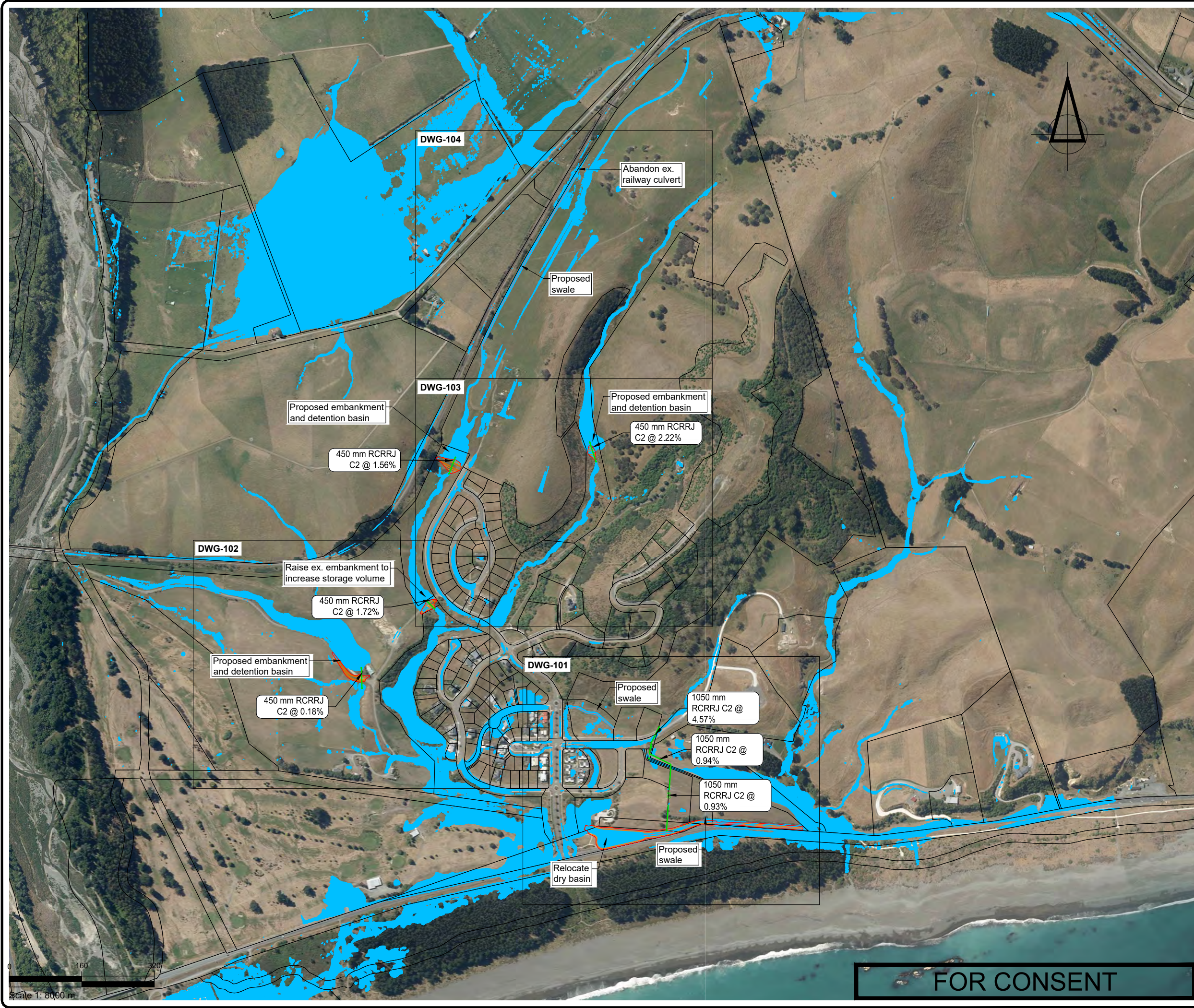
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DRAWING No HB00187-PC-008	REVISION

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REVISION	CHANGES		CHECKED	DATE

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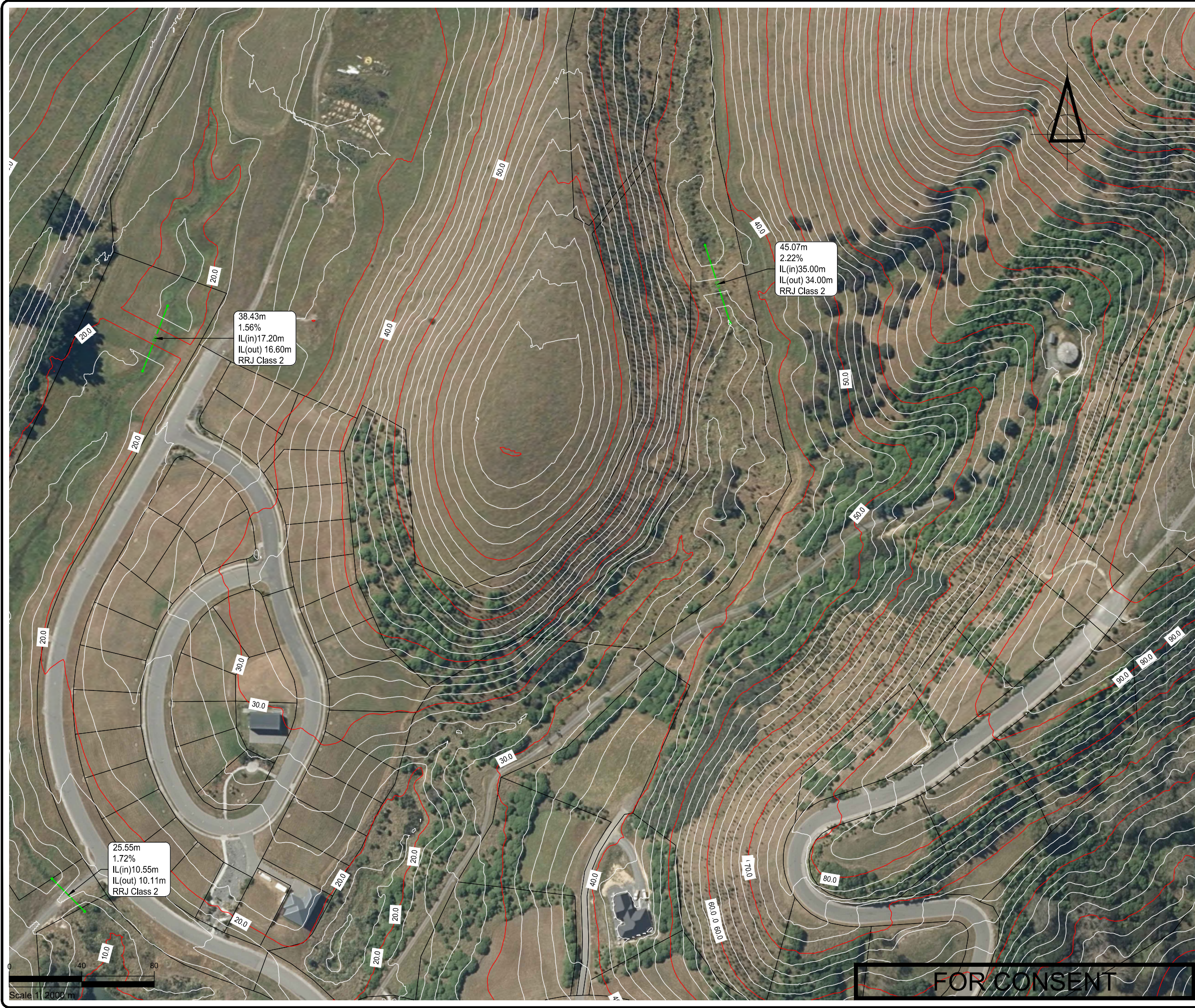
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DESIGNED	SN	03/06/25		
DRAWN	SN	03/06/25		
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SCALE	1:2,000	(A3)
DRAWING No	HB00187-PC-103	REVISION
		-

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APPENDIX D

PROPOSED MASTER PLAN

Novo Group Limited
Level 1, 279 Montreal Street
PO Box 365, Christchurch 8140
0 - 03 365 5570
info@novogroup.co.nz

 SITE BOUNDARY
 INDICATIVE AREAS
 LOW DENSITY RESIDENTIAL AREAS
 RESIDENTIAL AREAS
 MULTI-USE AREA

☒ A RESIDENTIAL AREA
☒ B RESIDENTIAL AREA
☒ D(i) LOW DENSITY RESIDENTIAL AREAS

- C THE CREST
- D THE DOMAIN
- E THE ESTATE
- F THE FARM
- G GREENLANE
- H THE HUB

 PRIMARY ROADS
 SECONDARY ROADS
 LOCAL ROADS

..... COASTAL PACIFIC TRAIN LINE

 OPEN SPACE
 EXISTING PRE-DEVELOPMENT NATIVE PLANTING
 EXISTING DEVELOPMENT NATIVE PLANTING

----- WALKING TRACKS (NOT SUBJECT TO ST. 4 EASEMENTS)

----- WALKING TRACKS (SUBJECT TO EASEMENTS)

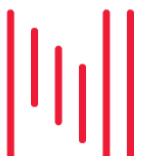
 DISTRIBUTION LINE AND 20M OFFSET
 40M & 100M ACOUSTIC RAILWAY OFFSET

D	9/05/2025	FOR COORDINATION
C	24/02/2025	FOR COORDINATION
B	24/01/2025	FOR INFORMATION
A	9/09/2024	FOR COORDINATION
REV	DATE	STATUS



PROJECT
OCEAN RIDGE

PROJECT NO. 132001	DRAWN OC/ECF
SCALE 1:7500	DATE
SHEET NO. L1.01	REVISION NO. D
STATUS FOR COORDINATION	



DRAFT

KEY

EXISTING AREAS (RETAINED)

PROPOSED AREAS

TRANSPORT NETWORK

ACCESS

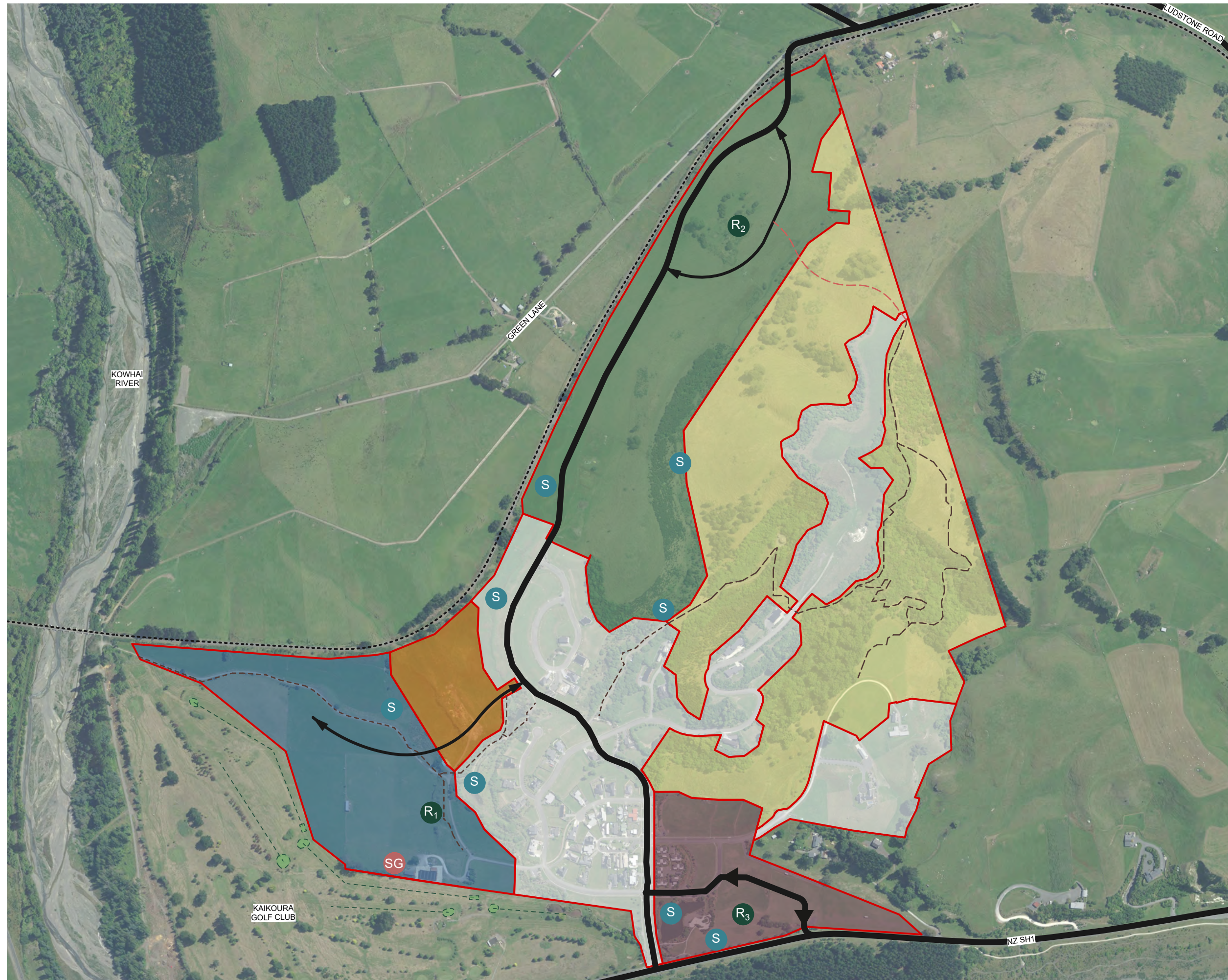
RESERVES & OPEN SPACES

RESERVE AREA	INDICATIVE AREA
R1	2.29Ha approx.
R2	1.45Ha approx.
R3 (Proposed Kekeno Park Extension)	0.53Ha approx.

 STORAGE / GARAGE AREA (INDICATIVE)

CLIENT
OCEAN RIDGE DEVELOPMENT

DRAFT



APPENDIX E

ADDENDUM 1

MEMORANDUM / FILE NOTE

Date: 6/11/2025 HB00187

From: Son Nguyen

Subject: Ocean Ridge, Kaikoura - Proposed Plan Change
Stormwater Modelling Study
Addendum 1 – Dry Basin Options

To: William Loppe (KPMO) <william.loppe@kpmo.co.nz>
Jason Jones <jason@rmgroup.co.nz>

1.0 INTRODUCTION

This document is an **addendum** to the Fraser Thomas Stormwater Modelling Study for the proposed Plan Change at Ocean Ridge, Kaikoura (July 2025). The original study assumed that the existing dry stormwater basin in Kekenok Park would be repurposed for the mixed-use Hub Area, with a replacement basin provided elsewhere within the park. Although this arrangement is subject to a future reserve exchange, it formed the basis of the original model.

This addendum compares the existing and replacement basins to confirm that exchanging them would not affect the function of the stormwater system.

2.0 BASIN VOLUME AND DRAINAGE PATH

The existing and replacement dry basins have the same invert and top water levels, with total storage volumes differing by less than 3%, as shown in the table and figure below. Accordingly, they can be interchanged without materially affecting the model results.

The original model includes an open drain, directing runoff from the Hub into the replacement basin. An alternative drain connection to the existing basin is also feasible, as illustrated in the figure below.

Table 1: Basin Levels and Volumes

	Top of the water level	Bottom of the basin	Volume
Existing dry basin	RL 6.84 m	RL 6.0 m	2,450 m ³
Replacement dry basin	RL 6.84 m	RL 6.0 m	2,520 m ³



Figure 1: Existing and Replacement Dry Basin Volumes and Drainage Path

3.0 LAND USE

The July 2025 flood model assumes that a portion of the existing dry basin will be filled and redeveloped as a mixed-use area, as shown in the figure below. This area is approximately 5,800 m², and is assumed to have an impervious coverage of 70%, compared with the existing dry basin's 0% imperviousness. The model, therefore, represents the worst-case scenario between the two development options in terms of the site coverage.



Figure 2: Assumed Land-use in Stormwater Model

4.0 **CONCLUSION**

Based on the above assessment, it is our opinion that the hydraulic characteristics of the existing and replacement dry basins are effectively equivalent. Therefore, they can be interchanged without adversely affecting the overall performance of the stormwater system. It is anticipated that the stormwater model will be reviewed and refined during the detailed design stage of the subdivision.