occurs in both the Ō2NL Project and baseline models (in events larger than 1:10 AEP current climate). The presence of the new highway embankment will not change the depth or duration of flooding over the existing road.

156. Downstream of the existing Manakau Stream bridge, the model shows that minor changes dissipate well within the proposed designation for the main design event.

157. Peak velocities in some parts of the modelled realignments for the main design event reach approximately 3m/s compared to 2m/s peaks in the baseline situation. This is the result of realigning the channel.

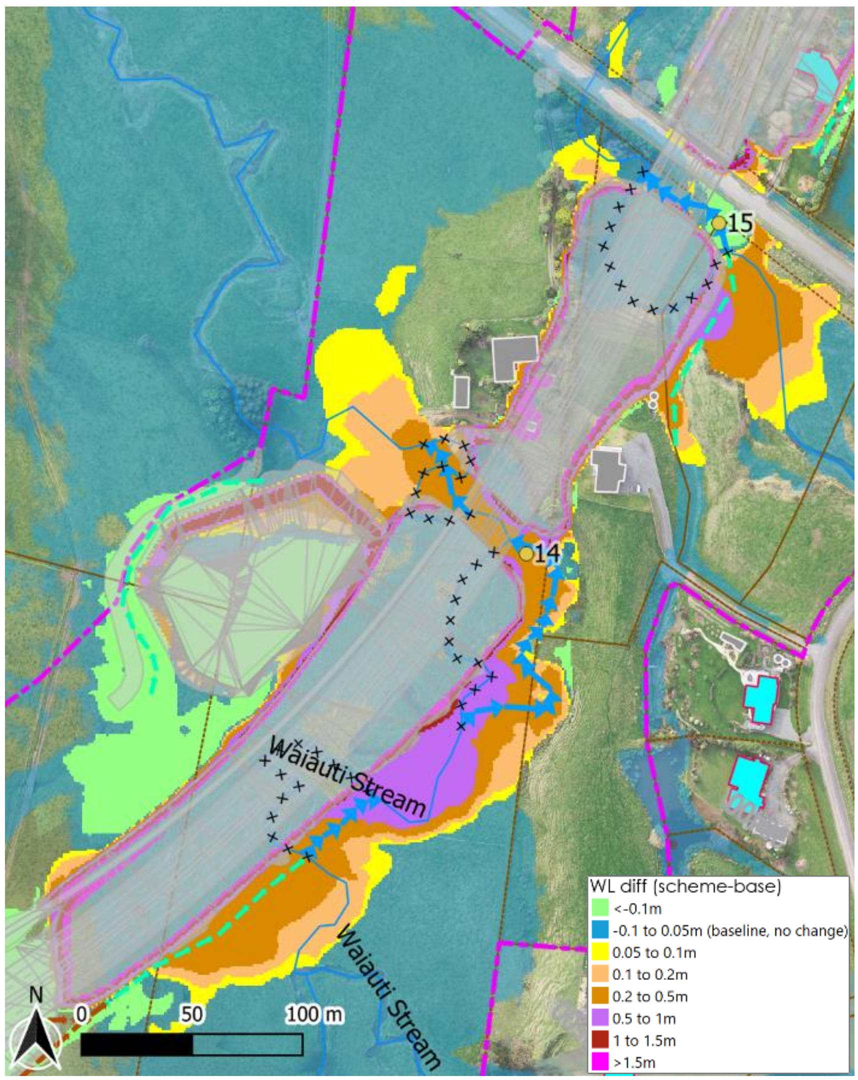

Figure F.28: Peak level differences Manakau and Waiauti streams in a 1:100 AEP flood event with climate change

158. The Waiauti Stream (ID 14) is a small meandering stream (**Figure F.29**) with a relatively wide floodplain (approximately 250m).

Figure F.29: Waiauti Stream looking downstream toward proposed crossing

159. The indicative embankment occupies a considerable proportion of the floodplain, with a new bridge towards the true right (northeast) corner of the floodplain. Constructed meanders will be designed to mitigate the loss of fluvial environment under the embankment.

160. The modelled opening between the bridge abutments is approximately 20m (**Figure F.30**). The upstream peak water levels in the throat of the bridge increase by approximately 0.3m relative to the baseline, although differences of up to 0.7m are noted near the embankment. This may be the result of a combination of shortening of the stream, and slight under-sizing of the modelled realignment channel in this area; however, any effect is considered to still be within acceptable limits.

161. Downstream of the Waiauti Stream bridge, the slight increase in water levels that is caused by the lateral concentration of flow through the bridge rebalances within the proposed designation. There is no difference in flood flows or water levels downstream of the proposed designation.

162. Peak velocities in the throat of the bridge range from 2-2.5m/s, compared to around 1.6m in the baseline (noting that the route of high velocities changes because of the realignments). Any effects from velocity caused by the final design can be mitigated by scour protection within the proposed designation.

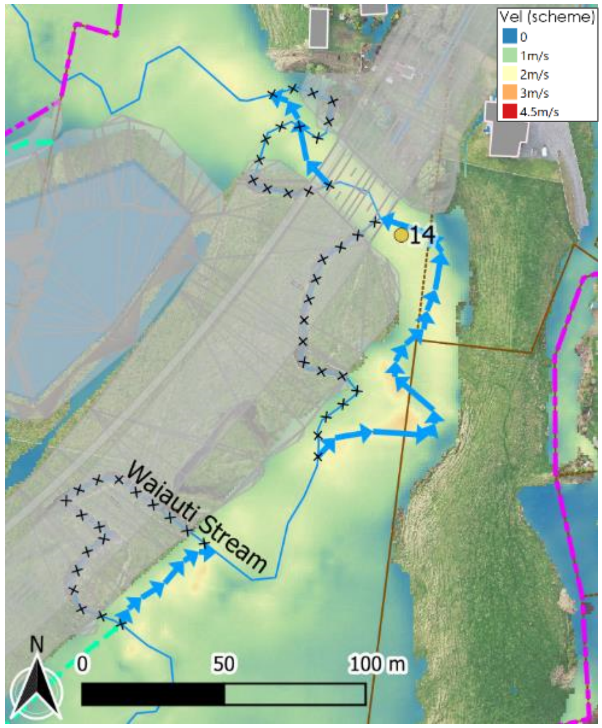

Figure F.30: Waiauti Stream peak velocity in a 1:100 AEP flood event with climate change (concept design stream diversions shown as blue arrows and stream to be removed indicated with black dashed line)

163. In the 1:10 AEP event, slight changes in lateral distribution between the Manakau and Waiauti floodplains dissipates to original patterns (<0.05m) within 100m downstream of the proposed designation. There are no cumulative effects passed further downstream and no existing buildings with discernible increases in flood risk.

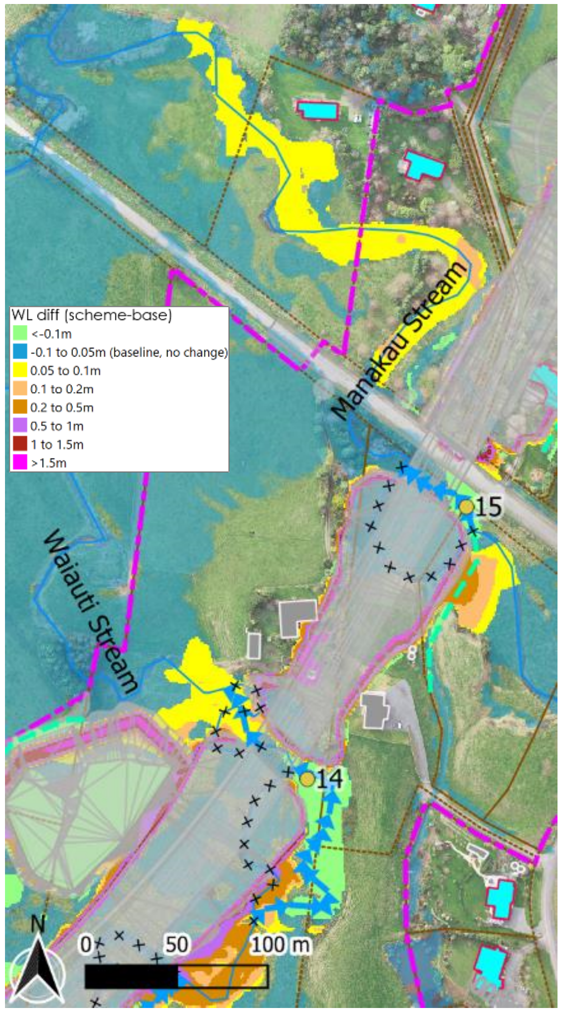

Figure F.31: Peak level differences Manakau and Waiauti streams in a 1:10 AEP flood event

164. Given the above, the effects of the Manakau and Waiauti Stream bridges on hydrology and flooding are considered less than minor.

Smaller streams and overland flow management

165. Ephemeral overland flows that approach the Ō2NL Project at the top of cut sections or toes of fill embankments are captured in clean open channels to avoid excessive ponding or erosion against the Project. The small flows captured by these channels are routed to the most appropriate stream or culvert so that prevailing flow paths downstream are maintained.

166. The catchments leading to indicative culvert locations tend to have relatively steep narrow valleys. As a result, the localised increases in peak flood levels at the inlets to most culverts are predominantly contained within the proposed designations. Only a few modelled culverts show peak level increases

approaching or marginally exceeding 0.5m at the upstream proposed designation boundaries, which dissipate rapidly to <0.1m within less than 50m upstream of the boundaries, which is considered less than minor given the land use and topographic context.

167. Any lateral changes in flow distribution at the culvert outlets are rebalanced within the proposed designations. There are no cumulative increases in peak flood flows or peak water levels downstream of the proposed designations.

Queen Street East

168. During extreme events (e.g., the 1:100 AEP RCP 6.0 2130), there is a substantial overland flow path adjacent to Queen Street that carries runoff from the east toward Levin.

169. East of the Ō2NL Project, most of the flow is on the northern side of Queen Street. This flow is therefore taken underneath the proposed new Queen Street East raised embankment and then underneath the Ō2NL highway. The downstream flow distribution toward Levin will remain essentially unaltered and consequently the Ō2NL Project will have less than minor effects on the existing situation.

Material supply sites and spoil sites

170. The exact volumes and final form of the material supply and spoil sites are not known and will remain somewhat uncertain until detailed design has occurred, and construction commences, because of potential material variability on site.

171. On a conservative basis, the potential hydraulic behaviour and effects of the full development of these sites and their consequent rehabilitation (as shown in the CEDF) have been inferred from the available model results.

172. **Site #34a (Koputaroa tributary)**: There would be no adverse effects on flooding either upstream or downstream because of material extraction. The indicative material supply boundary is set back from the (ephemeral) stream floodplain, although it is understood that the site will be integrated into an online wetland (i.e., not separated from the stream). The site could represent a very slight advantage in terms of flood risk in some events (from the additional storage of flood water on a wider floodplain).

173. **Site #36 North Ohau**: There would be no adverse effects on flooding upstream or downstream because of material extraction. The site could represent a small advantage in terms of flood risk, because of the additional storage of flood water on a wider floodplain during some events. The indicative outline currently shows a small clash with an overland flow path on the north-western side of the site (near Ō2NL chainage 22250). This could be addressed easily by either modifying the proposed outline or by re-alignment of the overland flow path within the proposed designation extent. Parts of the terrace site does have some overland flow in major events (larger than 1:10 AEP current climate). Whilst the river is relatively stable currently, there remains the possibility that future injections of gravel from earthquakes or severe storms could increase aggradation, lateral erosion and avulsion. The Ō2NL Project and this material supply site will not change this risk, but if this scenario eventuates then deep-seated scour protection on the upstream face of the Ō2NL highway embankment will mitigate effects by steering flows toward the flood relief bridge. Periodic monitoring and maintenance may be required, which will be further specified through detailed design and may also be influenced by the final form.

174. **Site #19 North of Waikawa (east and west)**: There would be no adverse effects on flooding upstream or downstream because of material extraction. The site has the potential to offer a very slight flood benefit if allowed to fill with flood water during major events by offering additional floodplain storage potential. The velocities in this area will be low because of the Ō2NL highway embankment. Integration of the stormwater pond into the western side will be an important design consideration to maximise material recovery and legacy outcomes.

175. **Site #15 South of Waikawa**: There would be no adverse effects on flooding upstream or downstream because of the material extraction. The site could provide a slight benefit in terms of flood risk, by the capture and attenuation of overland flow toward the west of the site. There is a small, discontinued water race across the site. The material supply site will capture and route a similar size catchment to the outlet, so that flows will closely mimic the existing situation. The route of the historical water race will still be used as an overland flow path to pass excess runoff. The design of attenuation will ensure that the future peak discharge would be similar to or less than the baseline. The site outline currently shows a small extension onto the Waikawa floodplain near the proposed O2NL bridge. This is understood to

be a potential access corridor rather than forming part of material supply. Care should be taken in detailed design to protect the riverbank in this location, to avoid increasing the potential risk of accelerated lateral scour from the highly mobile Waikawa Stream.

176. In summary, the material supply sites represent no adverse effects on flooding and may reduce existing flood risk very slight because of additional flood storage during.

177. Spoil sites will be subject to detailed design so that they avoid or minimise affecting flood storage or flow paths. Spoil site boundaries, shape and/or the route of open collector channels will be designed ensure no aggravation of the existing flood risk outside the proposed designations. With reasonable care during the design process, the magnitude of effects arising from the spoil sites will be less than minor.

Longitudinal stormwater management features

178. Stormwater management devices have been proposed to capture, convey, treat, and attenuate runoff from the road surface and cuts. During the 1-hour and 4-hour events simulated in the hydraulic model, the stormwater management system is effective in preventing increases in flood levels downstream. The final shapes of the devices, landscaping and optimising of outlet dimensions to mimic existing pre-development peak runoff rates will be completed through detailed design.

179. An illustration of the attenuation performance of stormwater pond 4 (chainage 13400, near the proposed SH57 roundabout) is provided in **Figure F.32**. Additional detail is provided in the Stormwater Management Design Report (Appendix 4.2 to the DCR).

180. Pond 4 discharges toward the Koputaroa Stream catchment. Post development outflows are throttled to pre-development rates during a 24-hour storm. All shorter duration events display a substantial reduction in pond peak outflow compared to the same duration pre-development peak flow.

181. The stormwater management features of the Ō2NL Project therefore provide a benefit of reducing downstream flood risk in most storms compared to the baseline scenario.

182. Koputaroa Stream is constrained by stop banks that limit its flow capacity and cause it to overtop the stop banks in most years. During these frequent events, the proposed stormwater management devices will significantly reduce peak flows from the Project's contributing area compared to the baseline situation. This represents a slight reduction in the existing flood risk to areas behind the Koputaroa stop banks. The magnitude of the benefit will be small because the Ō2NL Project's contributing area is only 13 hectares (0.13km^2). This is only 0.2% of the total 54km^2 Koputaroa catchment area upstream of its confluence with the Manawatū River.

183. The Koputaroa Stream is occasionally hindered from draining during high levels in the Manawatū River, because of long duration storms higher up in the Manawatū catchment. During these events, flooding of the land behind the stop banks is common, despite the lower intensity rainfall over the Koputaroa catchment. Flooding is caused by local rainfall and inflows from the Koputaroa and Waoku streams which are prevented from draining effectively to the Manawatū River.

184. While the stormwater management devices slightly reduce the magnitude of the peak discharge in the Koputaroa Stream, the Ō2NL Project may result in a very slight net increase in hydrograph volume because of the runoff from new paved areas. The effects of this volume increase are less than minor as explained below using two example events:[12]

(a) During a 24-hour 1:10 AEP event, the lower Koputaroa floodplains are already heavily inundated in the baseline situation over an area of approximately 3km^2. The additional (delayed) volume contributed by the four stormwater management ponds constructed for Ō2NL in the Koputaroa catchment could be about 13,000m^3. Distributed over 3km^2, the potential accumulated increase in depth would be only 4mm after 72 hours (if high levels in the Manawatū prevent gravity drainage). This change is considered less than minor because the area would already be flooded for many hours.

(b) During a 24-hour duration 1:100 AEP event, the lower Koputaroa floodplains are already heavily inundated in the baseline situation over an area of approximately 3km^2. The additional (delayed) volume contributed by the four stormwater management ponds constructed for

[12] Areas of potential inundation estimated using GIS. Volume estimates obtained from Mr Nick Keenan, author of the Stormwater Management Design Report (Appendix 4.2 to the DCR).

Ō2NL in the Koputaroa catchment could be about 19,000m³. Distributed over 3km², the potential accumulated increase in depth would be only 6mm after 72 hours (if sustained high levels in the Manawatū prevent gravity drainage). This change is considered less than minor because the area would already be flooded for many hours.

185. Koputaroa Stream has sensitive water quality and ecological constraints, which are addressed in Technical Assessment H (Water Quality) and Technical Assessment K (Freshwater Ecology).

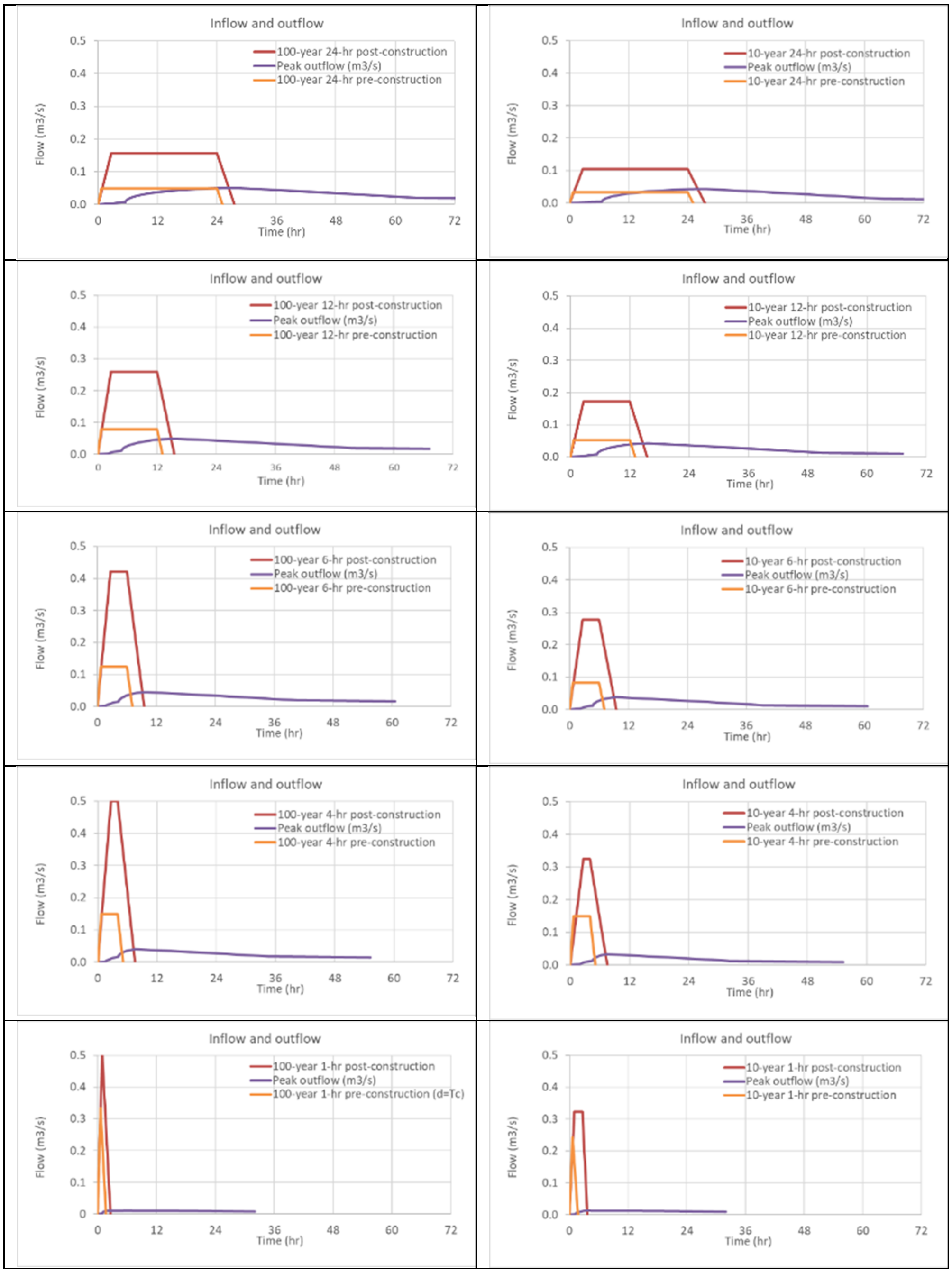

Figure F.32: Stormwater pond 4 attenuation performance illustration

186. For the remainder of the Ō2NL Project area that does not drain toward the Koputaroa Stream, the same stormwater attenuation performance principles have been followed to ensure sufficient space for stormwater management in the proposed designations. The final design of the Project will ensure that the peak pond discharges do not exceed 80% of the pre-development peak flow rate during a critical 4-hour storm up to 1:100 AEP. The 80% factor is intended to avoid any cumulative hydrological effects that could increase the peak flow downstream.[13]

187. Where ground conditions allow, such as south-east of Levin, some treated stormwater will be discharged to ground. These treatment devices have been sized to allow soakage of all runoff during the 1:100 AEP event. A factor has been included to allow for change in efficiency over time. Excess flows in events above this threshold will be routed to mimic existing overland flow paths that otherwise exist in the without Project scenario.

188. As a result of this design philosophy, it is anticipated that the Ō2NL stormwater system will have effects on flooding and hydrology that are less than minor.

189. Some existing receptors may experience marginally less flooding during some events because of the attenuation of highway runoff.

190. Further discussion on stormwater design philosophy and volumetric performance is provided in the Stormwater Management Design Report (Appendix 4.2 to the DCR).

191. Further discussion on the water quality effects of the Ō2NL Project is provided in Technical Assessment H (Water Quality).

Tararua Road / existing SH1 / NIMT improved intersection

192. The proposed improvements to the Tararua Road intersection with existing SH1 include a new level crossing of the NIMT. The proposed works will be essentially ‘at grade’ with the existing terrain. The NIMT railway line is the highest local hydraulic control at this location, and the works will not change the elevation or drainage of the NIMT railway.

[13] Stormwater Treatment Standard for State Highway Infrastructure, Waka Kotahi NZ Transport Agency, 2010.

193. The detailed design of the new road works will minimise any modification of existing overland flows in flood events and therefore any effect on hydrology and flooding will be less than minor.

Summary of net effects downstream of the Ō2NL Project

194. Results of computational hydraulic modelling have been used to test an indicative concept design for the Ō2NL Project. Slight changes in lateral distribution downstream of some of the bridges redistribute over a short distance and no cumulative effects are passed further downstream. No existing buildings beyond the proposed designations are subject to discernible increases in flood risk. Any effects of the Ō2NL Project on hydrology and flooding are considered less than minor.

195. In addition, the Ō2NL Project will provide a more resilient highway during heavy rainfall that is predicted to increase with climate change. The proposed Ō2NL Project will lower risk exposure and provide greater regional resilience benefits to emergency responders, operators, and users of the road network, compared to the existing SH1.

Construction phase considerations

196. Early construction activities (as discussed in the DCR) will typically include the following activities that are pertinent to hydrology and flooding (some may occur in parallel or in a different order, or broken into project areas):[14]

(a) Preparation of Temporary Works Areas ("**TWAs**"): most TWAs proposed on the drawings have low flood risk, however TWA 4 (north of Tararua Road) and TWA 7b (south of South Manakau Road) both have overland flow paths through the site that could occasionally flow (estimated 1:5 to 1:10 AEP current climate). Since the flows are relatively small and shallow, site layouts could be designed to minimise risk to occupants or equipment and without changing flow patterns downstream.

(b) Preparation of erosion and sediment control measures[15] include open collector channels (to reduce runoff into the main works areas), silt fences, decanting earth bunds, sediment retention ponds, construction water storage ponds, etc. Most of these features are aligned with or will form part of the permanent works associated with the Ō2NL Project.

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[14] Refer to Design Construction Report, Volume II, for more detail on construction methodology.
[15] Refer to Erosion and Sediment Control report, Appendix 4.3 to DCR Volume II.

Since these involve only small flows from a very small percentage of each catchment, any changes in flood hydrology will be less than minor.

(c) Temporary access and haul roads created for construction purposes may include some temporary culverts with facilities for controlled overtopping. The design of temporary culverts will ensure that the effects on flooding that are similar or smaller than those of the permanent culvert, ie effects that are less than minor. Fish passage will be maintained on permanently flowing streams.

(d) Placement of permanent culverts on smaller streams or ephemeral overland flow paths, which may involve temporary and/or permanent flow path realignments as discussed in the DCR.

(e) Bridge piles, abutments, and piers will be designed and constructed to avoid increasing flood risk to inhabited buildings, scour or obstructing fish passage. Temporary works design and construction methodology are discussed in the DCR.

(f) Construction water abstraction, storage and application (e.g., for dust suppression). The small abstraction and application rates will not have an impact on flood hydrology. The proposed maximum abstraction rates are within catchment allocations in the Regional Plans,[16] in order to avoid or minimise effects. Abstraction will be reduced or ceased during periods of low flow. The management of water takes to avoid or minimise potential effects during periods of low flow is discussed in Technical Assessments K (Freshwater Ecology) and Technical Assessment G (Hydrogeology and Groundwater).

197. Many of these activities will take place relatively early, prior to much of the bulk earthworks, and under current climate rather than future climate. Therefore, the effects of the construction phase on hydrology and flooding will be less than those of the operational stage. In summary, construction phase effects will be less than minor.

[16] Discussed in Chapter 4.7.6.8 of the DCR, Table 4.4.

Operational phase considerations

198. The effects discussed throughout this report are applicable to a medium-high RCP 6.0 climate scenario after 100 years. This is considered a conservative case in terms of hydrological boundary conditions applied to the assessment.

199. The detailed design stage will develop monitoring and maintenance plans for managing scour, debris and sediment where required (especially after significant floods). This will include regimes to clear debris arrestors upstream of culverts, sediment traps and stilling basins.

SUMMARY RATING OF EFFECTS

200. Overall effects of the Ō2NL Project on hydrology and flooding will, in my opinion, be less than minor.

201. **Upstream** changes in peak water levels greater than 0.05m relative to baseline (for 1:100 AEP with climate change RCP 6.0 to 2130) have been mapped and evaluated, with the following findings:

    (a) Increases in flood levels upstream of bridges and culverts are generally contained within the proposed designation boundaries. Modelled increases dissipate to less than 0.1m within 50m upstream of the proposed designation boundaries (70m in the case of the Ohau River) and are commensurate with the landscape and land-use context and the extreme nature of the design event. The short durations of increased water levels are considered unlikely to have a material effect on sediment deposition or crop recovery.

    (b) No buildings outside the proposed designations are impacted by the modelled increase in flood levels for the 1:100 AEP with climate change RCP 6.0 to 2130.

    (c) In more frequent flood events such as the 1:10 AEP current climate, the peak flood level changes are contained within the proposed designations, except for backwater effects on the Ohau River that dissipate to less than 0.1m approximately 50m upstream of the proposed designation.

    (d) Therefore, given the rural context, the extreme nature of the design event (1:100 AEP with climate change RCP 6.0 to 2130), and the short duration and small footprint of impacts, I consider these effects less than minor.

202. **Within the proposed designations,** the design philosophy for bridges and culverts allows for effective passage of water and sediment underneath the Ō2NL Project:

(a) Localised increases in velocity within the proposed designations are small and will be managed with scour protection.

(b) Flows redistribute laterally to confirm to their original floodplain pattern within a very short distance downstream of the structures, and generally within the proposed designations.

(c) Fish passage is provided except for some culverts on ephemeral flow paths where no fish are present, and no viable habitat exists upstream.

(d) Stormwater from the highway will be managed within the proposed designations, including treatment and attenuation of any discharge. Scour protection will be provided where necessary, and any effects on hydrology and flooding will be less than minor.

203. **Downstream** of the bridges and culverts:

(a) Flows redistribute laterally to confirm to their original floodplain pattern (<0.05m relative to baseline) within the proposed designations or a short distance downstream (<115m in the case of the Ohau River for the 1:100 AEP design event with climate change).

(b) In the 1:10 AEP event, the only locations to show modelled increased levels downstream of the proposed designations are the Ohau River, Waikawa Stream tributary and Manakau Stream. These are all because of small changes in lateral distribution that totally redistribute upon returning to the main channel a short distance downstream.

(c) There are no cumulative effects passed further downstream, and no existing bfuildings with discernible increases in flood risk.

Conclusion

204. Based on my detailed assessment, my professional opinion is any adverse effects of the Ō2NL Project on hydrology and flooding in the area will be less than minor.

205. Increase in heavy rainfall anticipated from climate change is predicted to exacerbate flooding along the existing SH1. The proposed Ō2NL Project will lower risk exposure and provide greater regional resilience benefits to emergency responders, operators, and users of the road network, compared to the existing SH1.

MEASURES TO REMEDY OR MITIGATE ACTUAL OR POTENTIAL ADVERSE HYDROLOGICAL OR FLOODING EFFECTS

206. My assessment above concludes that any adverse effects of the Ō2NL Project on hydrology and flooding in the area will be less than minor.

207. No measures to remedy or mitigate actual or potential adverse flood effects are required outside of the proposed designations.

208. Within the proposed designations, the potential effects of increased velocity (for example at bridges, culvert outlets, or steep open collector channels) will be mitigated by scour protection measures. Potential effects of increased runoff from paved surfaces will be remedied by stormwater management devices that capture, treat, attenuate, and discharge runoff in a manner that mimics pre-development rates.

209. Potential effects during construction will be mitigated by appropriate construction methodologies with managed overflow pathways that do not exacerbate flooding elsewhere. Construction water abstractions and application will not have an impact on flooding patterns.

210. In my opinion, no additional measures will be required to remedy or mitigate actual or potential hydrological or flooding effects.

CONCLUSION

211. Based on a detailed assessment, any adverse effects of the Ō2NL Project on hydrology and flooding in the area will be less than minor. The new highway will provide significant resilience benefits to operators and users of the road network.

Andrew Craig

Andrew Craig

14 October 2022

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APPENDIX F.1: BASELINE FLOOD REPORT

ŌTAKI TO NORTH OF LEVIN: BASELINE FLOOD ASSESSMENT REPORT

PREPARED FOR WAKA KOTAHI NZ TRANSPORT AGENCY

August 2022

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REVISION SCHEDULE

Rev No.	Date	Description	Signature or Typed Name (documentation on file)			
			Prepared by	Checked by	Reviewed by	Approved by
0	17/04/2020	Preliminary Baseline Flood Assessment Report	AC	JP		
1	12/02/2021	Baseline Flood Assessment Report: Draft final	AC	NK	JP	JP
1.2	26/10/2021	Baseline Flood Assessment Report: Draft Final	AC	NK	JP	JP
1.3	04/03/2022	Baseline Flood Assessment Report: Final (update selected figures)	AC	NK	JP	JP
1.4	18/08/2022	Baseline Flood Assessment Report: Final (update selected figures)	AC	NK	JP	JP

Abbreviations

Abbreviation	Full Name
AEE	Assessment of Environmental Effects
ARF	Aerial Reduction Factor
AEP	Annual Exceedance Probability
DBC	Detailed Business Case
DEM	Digital Elevation Model
FFA	Flood Frequency Analysis
FSL	Fundamental Soil Layer
GWRC	Great Wellington Regional Council
HDC	Horowhenua District Council
HIRDS	High Intensity Rainfall Dataset
HRC	Horizons (Manawatū-Whanganui) Regional Council
h	Hour(s)
IPCC	International Panel on Climate Change
KCDC	Kapiti Coast District Council
LiDAR	Light Detection and Ranging (airborne survey to prepare DEM)
LINZ	Land Information New Zealand
NES-F	Resource Management (National Environmental Standard for Freshwater) Regulations 2020
MfE	Ministry for the Environment
Ō2NL	Ōtaki to North of Levin Project
PMP	Probable Maximum Precipitation
PP2Ō	Peka Peka to Ōtaki Expressway
RCP	Representative Concentration Pathway (IPCC climate scenario)
SLS	Serviceability Limit State
SH1	State Highway 1
ULS	Ultimate Limit State
Waka Kotahi	Waka Kotahi New Zealand Transport Agency

WAKA KOTAHI NZ Transport Agency

**Ōtaki to North of Levin:
Baseline Flood Assessment Report**

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APPENDICES

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1. Introduction

1.1 Background to the Project

Waka Kotahi NZ Transport Agency ('Waka Kotahi') is investigating a new 24km offline highway from Ōtaki to north of Levin (Ō2NL). The Project extent is between the SH1 / Koputaroa Road intersection north of Levin, to south of Taylors Road near Ōtaki where it links with the Peka Peka to Ōtaki (PP2Ō) Expressway.

Previous stages of the project looked at various corridor routes, which led to selection of the preferred corridor passing east of Levin. The preferred corridor identified in the Indicative Business Case (IBC) was approximately 300m wide. This has since been refined to the corridor shown approximately in Figure 1-1 (the final Designation extents may differ). The project is currently progressing through Detailed Business Case and intermediate level design toward preparation of consent applications (Resource Consent and Notice of Requirements).

This baseline flood assessment report presents an improved understanding of the flood risk near the corridor, which will be used to test the potential effects and mitigation design for the Project. Therefore, this report is not sensitive to design changes, and any reference to the corridor or the potential design only serves to highlight the area of greater interest or focus for the baseline model. A separate report will discuss the scheme representation and assessment of effects post scheme.

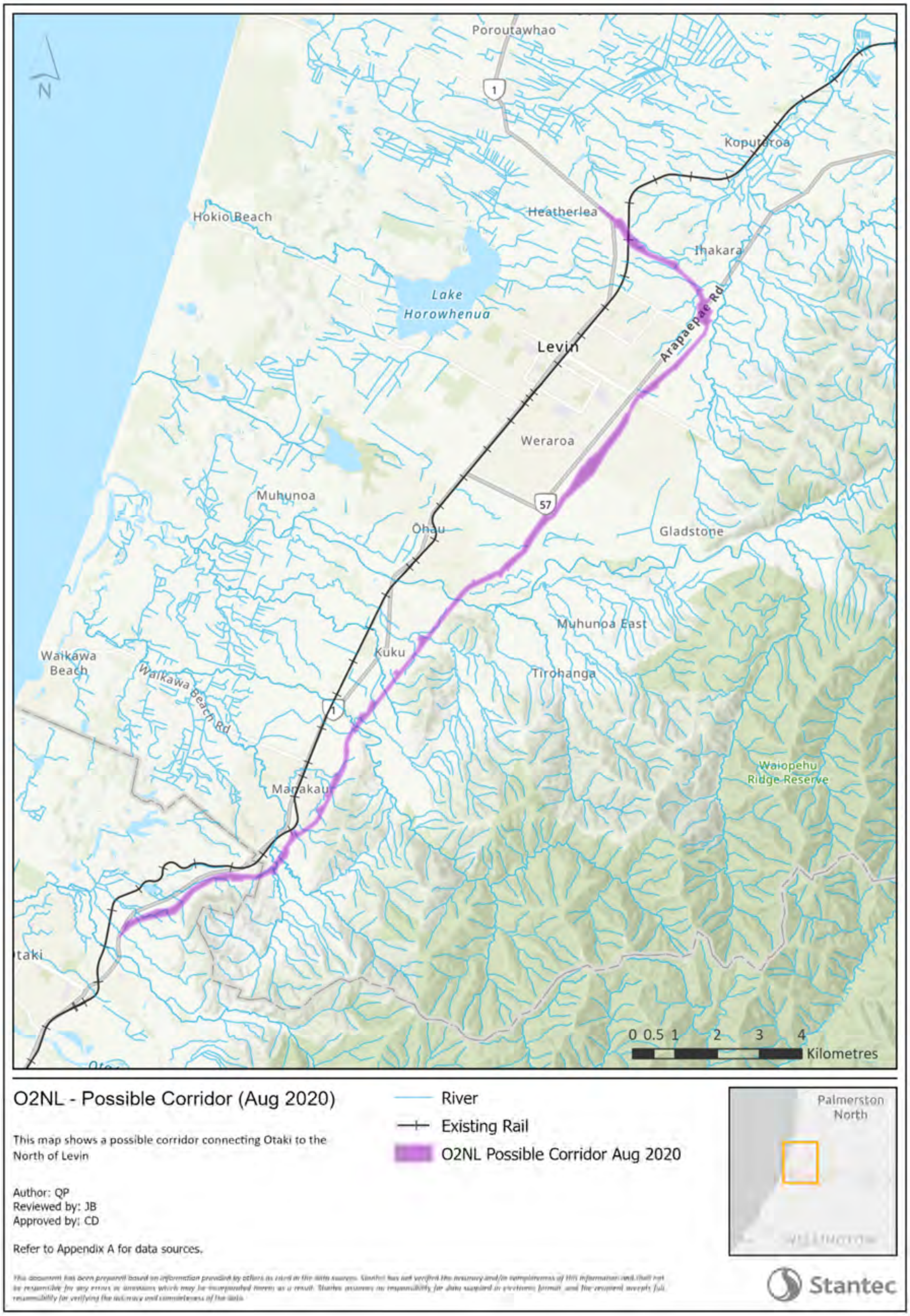


Figure 1-1: Location map and indicative Ō2NL Corridor

1.2 Purpose and Scope of this Assessment

This Baseline Flood Assessment Report presents a description of flood risk information in the vicinity of the Ō2NL Project corridor. It is based on existing topographical and asset information but includes an assessment of future climate change factors. This will provide Waka Kotahi and local councils with confidence that a sound baseline understanding will be used for the assessment of effects. The updated baseline information will also inform ongoing design work for the DBC.

There will necessarily be some assumptions and limitations, due to information gaps or modelling processes. These have been captured through the reporting process, and some may need to be revisited during Detailed Design to test whether the assumption has bearing on the final design or mitigation measures.

2. Reference Information

Key reference datasets obtained as foundation to this study are outlined below. The application of the datasets in analytical context is described later in the report.

2.1 Previous Ōtaki to Levin Studies Relevant to this Report

During the previous IBC Phase, flooding of the existing State Highway 1 (SH1) was identified as a key concern and resilience risk to the operation and availability of the existing highway. Flooding has caused the closure of SH1 numerous times in recent years. There is no possible alternative route for much of the project length, so providing an alternative and resilient transport corridor was a key project objective identified during the IBC stage.

The Multi-Criteria Analysis undertaken as part of the IBC included an Engineering Considerations criterion, part of which considered potential flooding risk of the corridor options. Horizons Regional Council provided advice on flood risk and known problematic locations during the IBC. The District Plan flood risk areas were also utilised previously to inform early stages of the project.

No ‘new’ flood modelling assessments were completed in previous phases of the project.

2.2 Regional Information Sources and GIS

2.2.1 Horizons Regional Council

The following relevant datasets have been obtained from Horizons Regional Council (HRC):

- A large proportion of the LiDAR derived DEM dataset towards the North and West of the study area. This data was captured over the course of multiple surveys between 2005 and 2018 at 1m resolution, and in the Wellington 1953 vertical datum. See Figure 2-1 below for coverage extent.
- Indicative modelled 1:200 AEP flood extents for the larger rivers in the region.
- Flow data for the Ōhau river at the Rongomatane river level gauge. A flood frequency analysis was also provided.
- Flow data for Koputaroa, Manakau and Wakawa streams, as discussed in Section 4.3.
- Rainfall data as discussed in section 4.2.1.
- Aerial imagery for the Horizons Region, captured in the summer of 2016-17, obtained via the LINZ data service.
- Existing surveyed cross section data for the lower Ōhau River (June-July 2018), from just upstream of the Rongomatane river level gauge to just downstream of the existing SH1.
- Background reports on the Koputaroa Drainage Scheme (scheme reviews 1997, 2007, 2014).

2.2.2 Greater Wellington Regional Council

The following relevant datasets have been obtained from Greater Wellington Regional Council (GWRC):

- LiDAR derived DEM covering the southern portion of the route, captured in 2013 and 2016 at 1m resolution, and in the NZD2016 vertical datum. See Figure 2-1 for coverage extent. We intend to adjust this data to Wellington 1953 vertical datum to provide a consistent vertical datum for the project.

- Aerial imagery for the Greater Wellington Region, captured in the summer of 2016-17, obtained via the LINZ data service.
- Rainfall data for the Ōtaki River at Depot, East Waitewaewae at Oriwa and Waitatapia Stream at Taungata gauges.

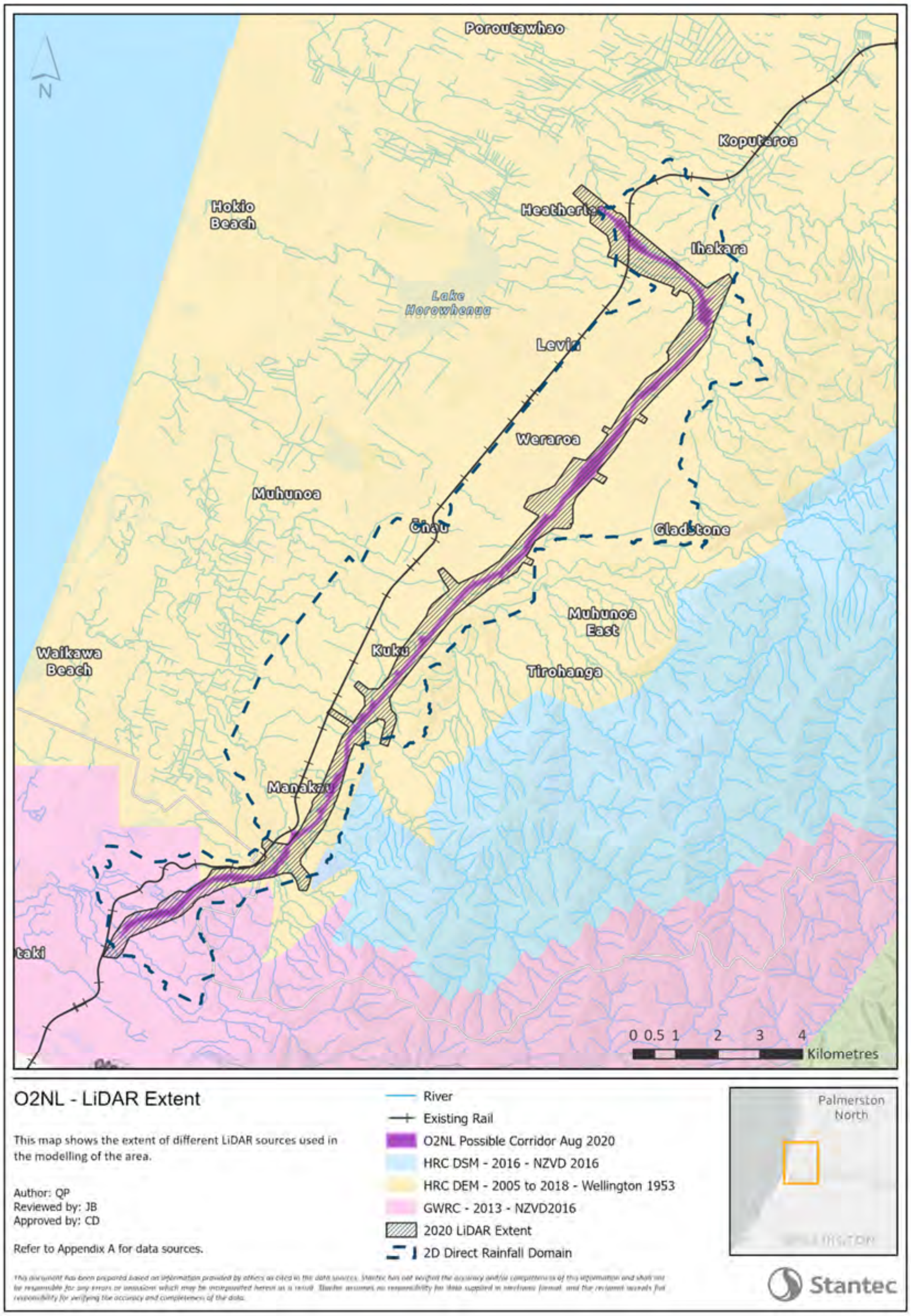


Figure 2-1: Extents of LiDAR datasets (source authority, year captured, and vertical datum)

2.2.3 Horowhenua District Council

The following relevant datasets have been obtained from Horowhenua District Council (HDC):

- Aerial imagery captured in the summer of 2015-16, obtained via the LINZ data service.
- Existing bridges and drainage asset information.
- Existing stormwater models for the built-up areas of Levin and Ōhau.
- Background reports on Catchment Management Plan modelling for the urbanised areas of Levin and Ōhau.
- Interim information on the proposed development of Tara-Ika east of Levin.

2.2.4 Kapiti Coast District Council

The following relevant datasets have been obtained from Kapiti Coast District Council (KCDC):

- Aerial imagery captured for the Kapiti Coast District Council area, captured in the summer of 2016-17, obtained via the LINZ data service.
- KCDC stormwater asset data.

2.2.5 Waka Kotahi NZ Transport Agency

The following relevant datasets have been obtained from Waka Kotahi:

- Existing State Highway bridges and drainage pit asset information
- Construction drawings and hydraulic model for the northern end of Peka Peka to Ōtaki Project (PP2Ō). The PP2Ō–Post Project Hydraulic Model files were used to update the northern end of PP2Ō construction details that represent the starting baseline conditions for assessment of the southern part of the Ō2NL project. The PP2Ō DHI MIKE model files were utilised to modify the proposed highway embankment north of the Waitohu River, including the proposed Greenwoods culvert.

2.2.6 KiwiRail

The following relevant datasets have been obtained from KiwiRail:

- KiwiRail existing bridge and culvert datasets.

2.2.7 Landcare Research

The following relevant datasets have been obtained online from Landcare Research using the Land Resource Information Systems (LRIS) portal[1]:

- Land Environments of New Zealand (LENZ) soil drainage layer – national level dataset providing a key component of the hydrology.
- Land Cover Database version 5.0 (LCDB v5) was extracted from the LRIS portal to classify the land cover on the hydraulic modelling extent. The LCDB v5 is a multi-temporal, thematic classification of New Zealand's land cover. It identifies 33 mainland land cover classes (35 classes once the offshore Chatham Islands are included). LCDB v5 was released in January 2020.

2.2.8 NIWA

The following relevant datasets have been obtained from NIWA:

- HIRDS v4 rainfall digital dataset.
- Levin MAF and Levin AWS rain data

2.2.9 2020 high-resolution aerial imagery and DEM

The present study included a new drone-based survey which captured aerial imagery and generated a DEM using photogrammetric processing. The survey campaign was flown by Cardno in July 2020 and

[1] https://lris.scinfo.org.nz/

covered the indicative project extents. The final point cloud has a point spacing of approximately 0.1m. The coordinate reference system is NZGD 2000 Transverse Mercator and the datum is Wellington 1953.

2.3 Key Guidance and Design Standards Referenced

The following standards are particularly relevant to this report:

- Adapting to Climate Change in New Zealand (MfE, 2018)
- HIRDS High Intensity Rainfall Design System (v4, NIWA, 2018)
- NZTA SP/M/022 Bridge Manual (3rd edition, NZTA, 2018)

2.4 Selection of Design Scenarios

Whilst this report focusses on baseline risk, it will be required to focus on scenarios that will be used to inform design and/or assessment of effects. For design purposes, the highway classification (under the NZTA One Network Road Classification) has been selected as “IL3+ National (High Volume)”. The associated serviceability and ultimate limit state design scenarios are provided in Table 2.1 of the NZ Bridge Manual, reproduced below:

Table 2-1: Design scenarios (reproduced from NZ Bridge Manual Table 2.1)

Bridge categorization	Importance level (as per AS/NZS 1170.0(4))	Bridge permanence*	Annual probability of exceedance for the ultimate limit state		Annual probability of exceedance for the serviceability limit state	
			ULS for wind, snow and floodwater actions	DCLS for earthquake actions	SLS 1 for wind, snow and floodwater actions	SLS 2 for floodwater actions
Bridges of high importance to post-disaster recovery (eg bridges in major urban areas providing direct access to hospitals and emergency services or to a major port or airport from within a 10km radius). Bridges with a construction cost (including associated ground improvements) exceeding $16 million (as at June 2018).	4	Permanent	1/2500	1/2500	1/25	1/100
		Temporary	1/1000	1/1000	1/25	1/100
Bridges on highways classified as National (High Volume in the One Network Road Classification (ONRC).	3+	Permanent	1/1500	1/1500	1/25	1/100
		Temporary	1/700	1/700	1/25	-
Bridges on highways classified as National, Regional, Arterial, Primary Collector or Secondary Collector in the ONRC.	3	Permanent	1/1000	1/1000	1/25	1/100
		Temporary	1/500	1/500	1/25	-

The design life (planning horizon) will extend to 100 years in accordance with the Bridge Manual. Assuming a start of operation around 2030, the design life will extend until at least 2130. This is particularly relevant for climate change. The allowances for climate change are based on HIRDS v4, noting this is based in turn on IPCC 5th assessment of 2014. The main SLS2 design scenario for flooding will be 1:100 AEP, with climate change scenario RCP6.0 extrapolated to 2130. HIRDS v4 only provides climate adjusted tables out to the epoch 2081-2100, and the extrapolation approach used to extrapolate out to 2310 is discussed in Section 4.2.7. RCP6.0 is a moderately conservative (medium-high) climate projection, which is considered suitable for the Ō2NL highway as the main SLS2 design case for operational use. The RCP6.0 scenario was also applied for the recent Te Ahu a Turanga; Manawatū Tararua Highway Project. The potential impacts of higher climate outcomes will be tested through the ULS case, namely the 1:1500 AEP with a more conservative RCP8.5 extrapolated to 2130. In summary, the following key scenarios will be used:

Table 2-2: Selected key scenarios

Annual Exceedance Probability		Climate Scenario	Description
1:10	10%	Current climate	Easier to relate to floods in recent history
1:100	1%	RCP6.0 2130	SLS2, operationally functional (at least one lane open in each direction)
1:1500	0.067%	RCP8.5 2130	ULS, resilience case (damage limitation, avoiding collapse)

Storm durations used are discussed further in Section 4.4.2. Adjustments for climate change are discussed in Section 4.2.7.

The modelling for future epochs is based on current topography, drainage assets and estimated infiltration rates. These may change slightly in future through natural morphological change (including earthquakes and associated debris load in addition to gradual erosion and aggradation processes), and/or anthropological changes in land-use with associated impacts on infiltration/runoff. For the purposes of the modelling, it is assumed that hydrological catchment net response to any depth-duration rainfall event will remain similar to historic performance to up 1:100 AEP with climate change, despite future anthropological or morphological change. This is considered a reasonable assumption because plans submitted under the RMA seek to ensure hydraulic neutrality to avoid or minimise potential adverse effects.

3. Existing Environment

3.1 Topographic Overview and Land Use

Along most of the route from Ōtaki to East of Levin, the topography slopes from the hills in the east towards the flatter coastal plains in the west, as shown in Figure 1-1. As the northern extent of the route wraps around the northern side of Levin, the drainage is generally north-eastwards toward the Koputaroa and Manawatū River floodplain. Over 30 significant catchments have been identified that drain across the corridor, although the final number will depend on the final alignment (work still in progress). The catchment definition process is described in Section 4.1. The Tararua Range forms the eastern boundary of most of the catchments, and with a maximum elevation (just outside the catchment) of 1570m this range generates significant orographic precipitation effect (refer Section 4 and Figure 4-8).

Catchment land use varies per catchment, but typically includes forested areas (approximately one third on average, notably on the hills and along the banks of some watercourses) and rural farmland with relatively low density scattered dwellings. As a result, all catchments are considered rural for the purposes of determining hydrology.

The geology is dominated by unconsolidated sediments of both fluvial and marine provenance, and as a result the soils are predominately medium to well drained as defined by Land Environments New Zealand (Soil Drainage, LRIS portal[2]).

3.2 Rivers and Streams

The largest watercourse crossing is the Ōhau River, which is a relatively high energy braided river with mobile bed material. Smaller watercourses include the Waikawa, Manakau, Waiauti and Kuku streams, plus several smaller unnamed streams and drains. These smaller drains were identified using LINZ online mapping plus LiDAR and high-resolution aerial imagery referenced in Section 2.2.

3.3 Existing Infrastructure

As described in Section 2.2, information was gathered from various local and national authorities on existing bridges and culverts, plus road and rail embankments, that will influence the progression of floodwaters in the area. The application of these in the hydraulic model is described in Section 5.

2 https://lris.scinfo.org.nz/

4. Hydrology

4.1 Catchments

4.1.1 Catchment boundaries for input into the hydraulic model

LiDAR and high-resolution aerial imagery were used to initially identify locations of watercourses that cross the indicative Project corridor. Catchments were then delineated above these proposed crossing points, to allow estimation of design flood flows by various methods. For the larger catchments above approximately 2km^2, point inflows will be used as inputs to the hydraulic model. Direct rainfall is then applied to the small catchments closer to and downstream of the corridor. This allows a baseline understanding of peak flood levels to be achieved everywhere in the vicinity of the corridor, irrespective of the eventual alignment (which was not known when the baseline model was first being developed).

Sub-catchments were delineated where necessary within the larger catchments to allow a more detailed representation of their respective contribution and timing in the hydrological modelling. Catchment and sub-catchment boundaries are shown in Figure 4-2. The application of the catchments within the modelling are discussed further in the subsequent sections of this report.

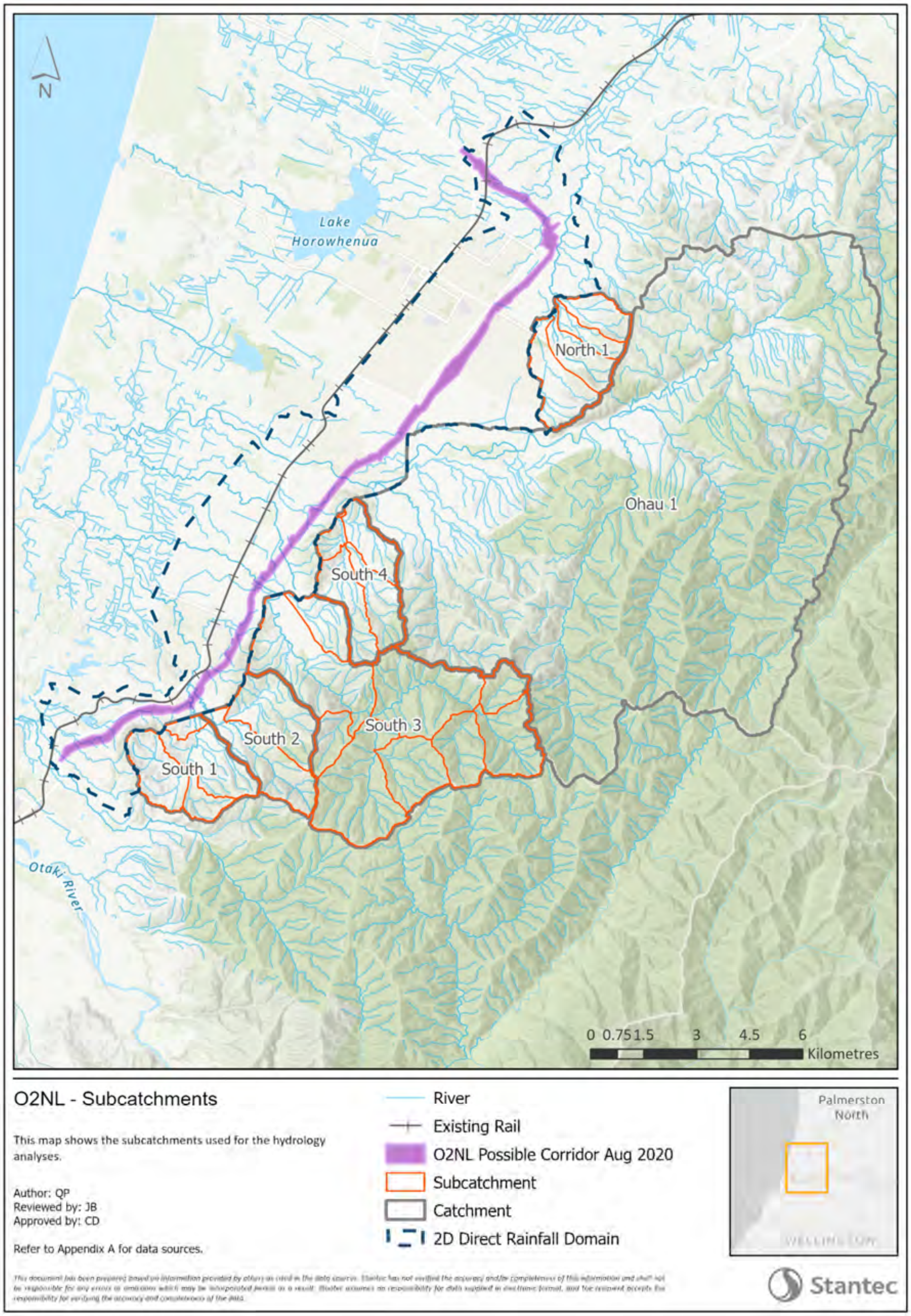


Figure 4-1: Catchment & sub-catchment delineation

4.1.2 Additional Validation Catchments

Manakau and Koputaroa catchments were also delineated further downstream of the Ō2NL corridor, where suitable flow gauge data was available to aid validation of the hydrological model. The Waikawa at North Manakau Rd flow gauge was also used to validate the Waikawa (South 3) catchment hydrological model. Refer to Section 4.3 for discussion regarding flow gauge & data suitability.

Catchment 39 (one of the catchments draining to a large culvert under the proposed Ō2NL highway, within the Koputaroa catchment) was used as an additional comparison point between the hydrological and hydraulic model results. These additional catchments are shown in Figure 4-2 below.

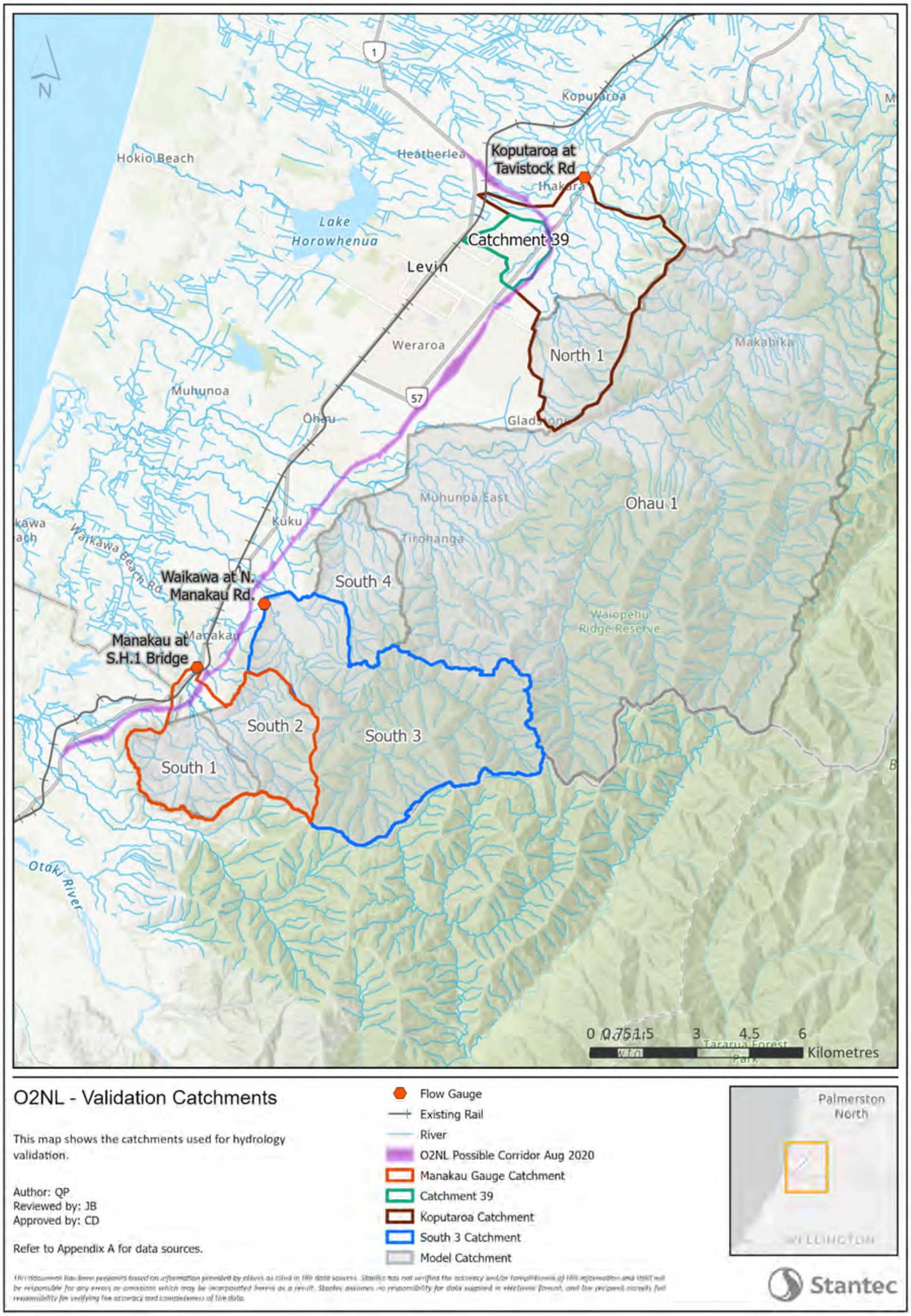


Figure 4-2: Catchments used for validation

4.2 Rainfall

4.2.1 Rain gauges

There are 8 rainfall gauges in or near the study area, where data has been collected for potential use in the calibration of rainfall runoff models and analysis of rainfall variability. These are summarised in Table 4-1 below. The temporal availability of data is shown in Figure 4-3, and the locations are shown on Figure 4-4.

Table 4-1: Rain gauges in or near the catchments

Name	Site Number	Latitude	Longitude	Recording Authority	Start of Data	End of Data	Altitude
Manawatū at Moutoa	55303	-40.4914	175.37207	HRC	Oct-99	Jun-20	5
Mangahao at No1 Dam	56403	-40.6252	175.47793	HRC	Jan-00	Jun-20	390
Ōhau at Makahika	56404	-40.6413	175.40065	HRC	Dec-09	May-20	240
Levin AWS	3275	-40.6199	175.2595	NIWA	Jan-95	Jan-13	15
Levin MAF	3277	-40.65	175.269	NIWA	Jan-86	Jan-91	46
East Waitewaewae at Oriwa	57302	-40.7496	175.34851	GWRC	Sep-91	Dec-20	1050
Waitatapia Stream at Taungata	58201	-40.8102	175.25687	GWRC	Sep-91	Dec-20	980
Ōtaki River at Depot	57106	-40.7693	175.14467	GWRC	Jun-92	Sep-20	17

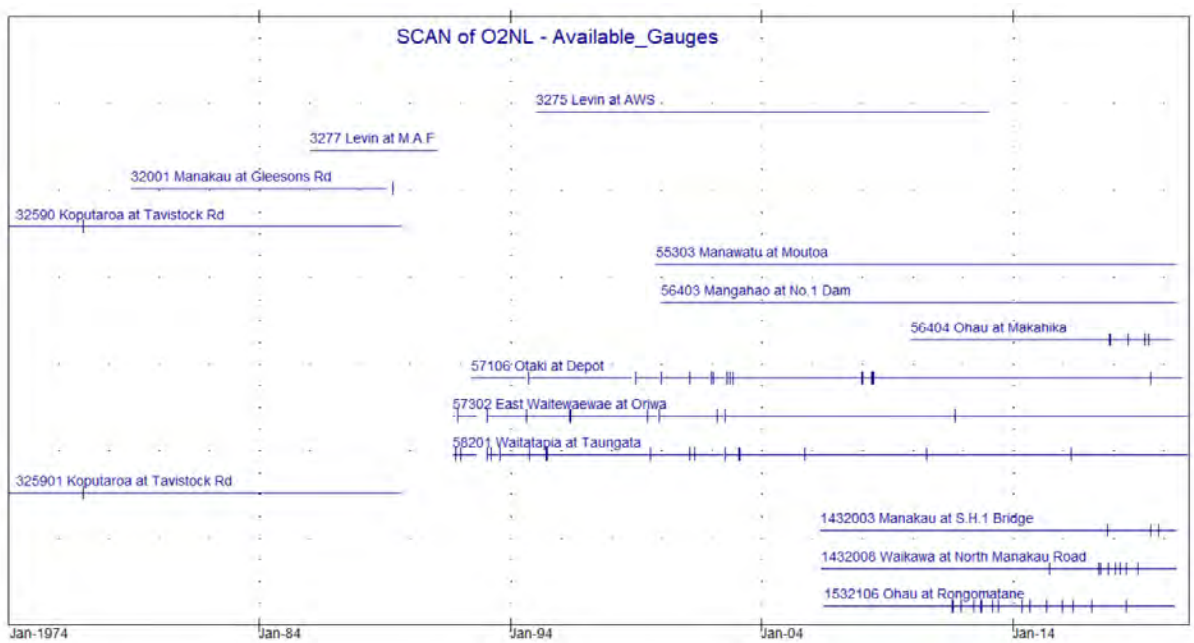


Figure 4-3: Graph showing availability of rainfall and flow gauge data

As illustrated in Figure 4-4 there are gauges at low elevations and at high elevation in the Tararua ranges which aid in estimation of the orographic precipitation gradient present in the larger catchments. For the gauges located in the ranges, the recording authority notes that they suffer from large evaporation discrepancies and are occasionally tampered by trampers.

Figure 4-5 shows cumulative rainfall plots for each gauge. This shows generally increasing rainfall gradient with elevation, although the gauge at Oriwa reports much more rainfall than Taungata which is at a relatively similar elevation and distance into the Tararuas. The disparity between these two gauges is discussed later in this chapter. The gauges on the coastal plain show a generally flatter gradient. There are no significant changes in cumulative rainfall gradient for these graphs, which confirms that there have been no significant changes in gauge exposure. Gaps in the data are indicated by squares.

Thiessen polygons were created based on the available gauge data for each observed event, to derive weighted catchment-average rainfall timeseries for use in the calibration process. An example of Thiessen polygons for the January 2008 event is shown in Figure 4-6. Given the sparse spacing of the rain gauges relative to the steep rainfall gradient, this method has limitations and might not accurately represent catchment average rainfall for some events. This application of catchment average rainfall is discussed further in 4.4.3 and Appendix C.

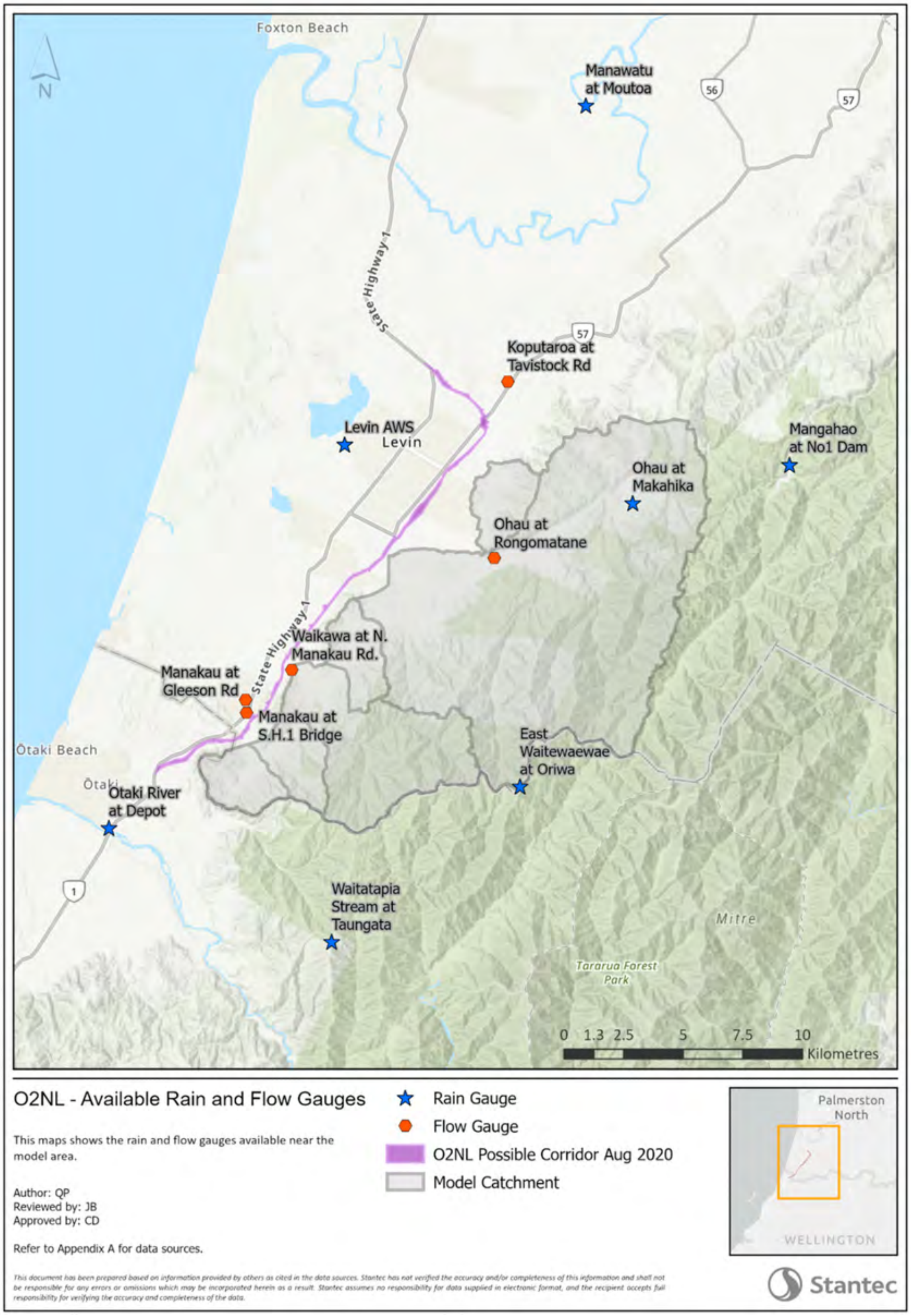


Figure 4-4: Available rain and flow gauges

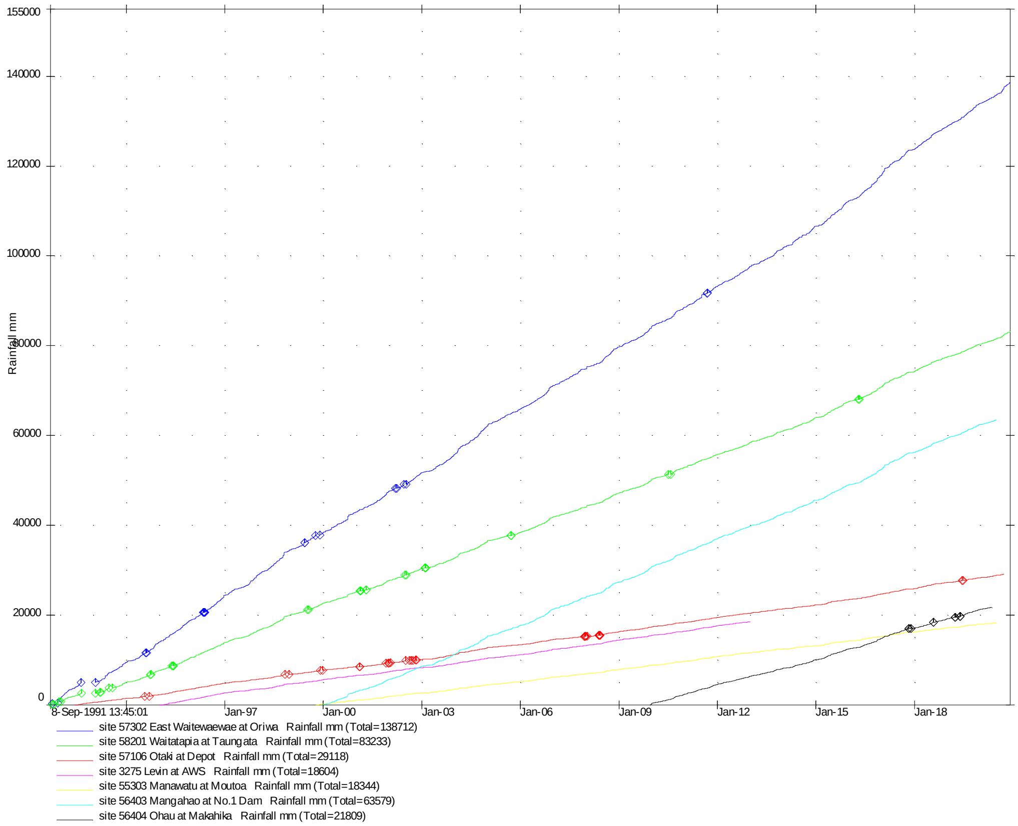


Figure 4-5: Cumulative rainfall plots

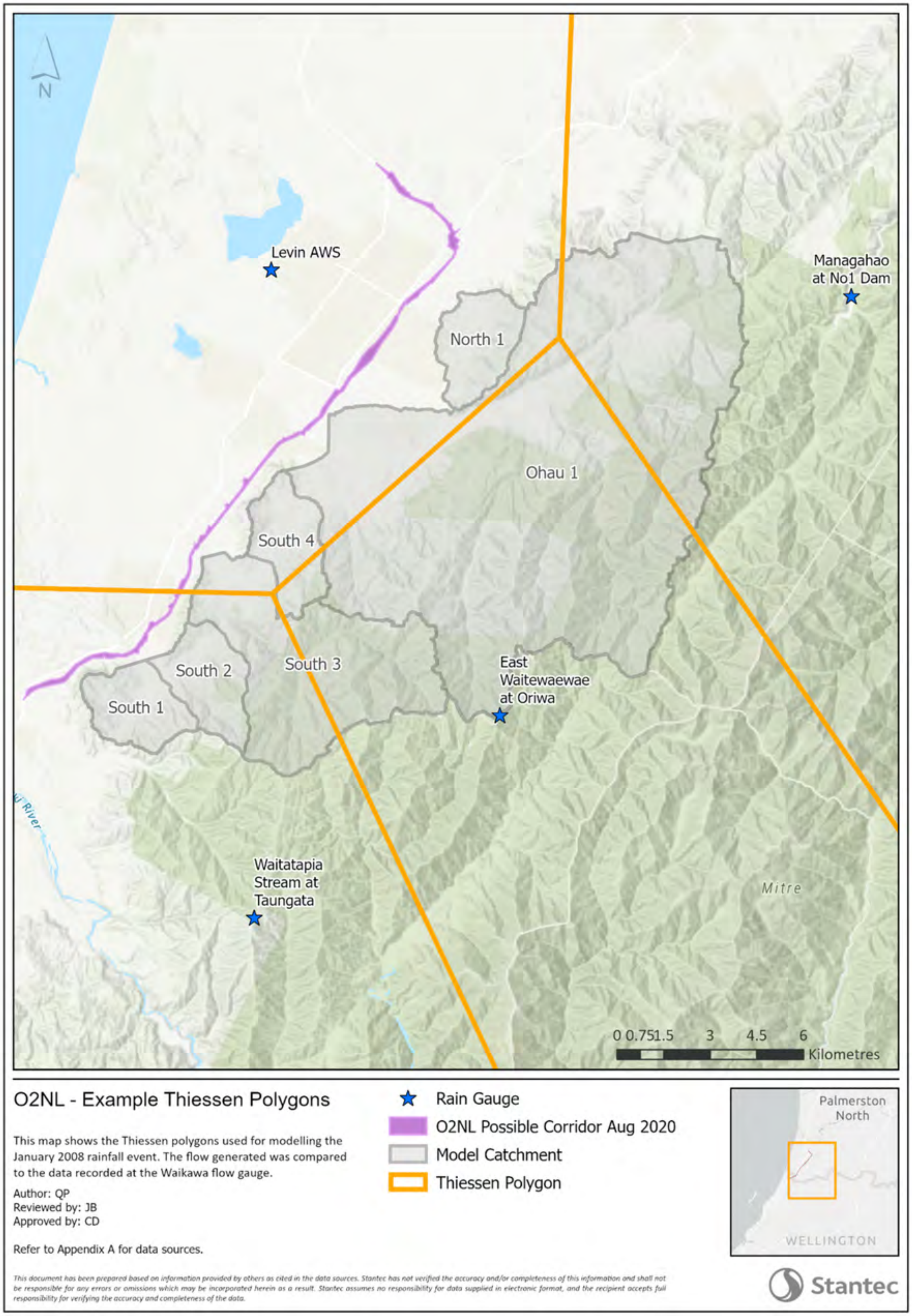


Figure 4-6: Thiessen polygons for the January 2008 event

4.2.2 Design rainfall up to 1:100 AEP

Design rainfall was required as input to the different hydrological approaches or processes that feed into the baseline flood modelling, notably:

- Design rainfall input to HEC-HMS rainfall runoff models (used to derive hydrograph shapes for medium-large catchments).
- Design rainfall inputs to the Rational Method for a subset of catchments as an independent check on the HEC-HMS and HEC-RAS modelling.
- Design rainfall for use in the 2D direct rainfall hydraulic model domain/s covering the smaller catchments, including near and downstream of the proposed Ō2NL corridor.

To prepare these rainfall inputs, HIRDS v4 design rainfall tables were downloaded for the centroid of each sub-catchment to account for spatial variation across the models. A further description of rainfall spatial variation is provided in Section 4.2.4.

Table 4-2: Example rainfall depths (mm) for Waikawa sub-catchment 1, current climate HIRDS v4

Event/Duration	10m	20m	30m	1h	2h	6h	12h	24h
1:10 AEP	13.1	17.9	21.5	29.5	40.1	63.5	82.8	105
1:100 AEP	20.8	28.0	33.5	45.3	61.0	95.1	123	154

4.2.3 Extreme rainfall (1:1500 AEP)

Whilst HIRDS v4 table downloads only extend to an AEP of 1:250, the HIRDS v4 online tool allows manual entry of a 1:1500 AEP probability to obtain associated rainfall depths. These values were checked by interpolation between the probable maximum precipitation (PMP), and HIRDs v4 1:50, 1:100 & 1:250 AEP events. Estimates of the PMP followed the method described in Tomlinson, (1993). An example interpolation check is shown in Figure 4-7 below.

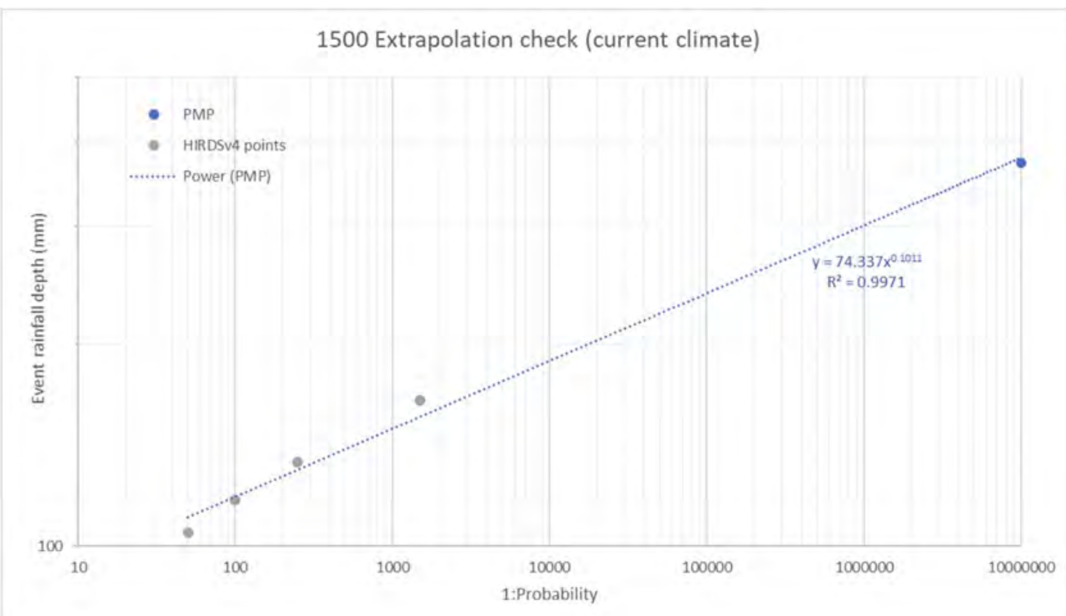


Figure 4-7: Example check on 1:1500 AEP rainfall (Waikawa catchment current climate)

4.2.4 Spatial Variation

As described in Section 4.2.2, the location of the catchments between the coast and the Tararua Ranges is within an area of significant orographic influence causing steep gradients in rainfall depths. Therefore, as a starting point for design rainfall, HIRDS v4 depths were calculated to the centroid of each modelled sub-catchment, including the 2D direct rainfall modelling zone. Figure 4-8 shows the spatial distribution based on HIRDS v4 2h 1:100 AEP. The 2h event is used for visual illustration only, as a compromise between the 1h and 4h storms that were analysed in the modelling.

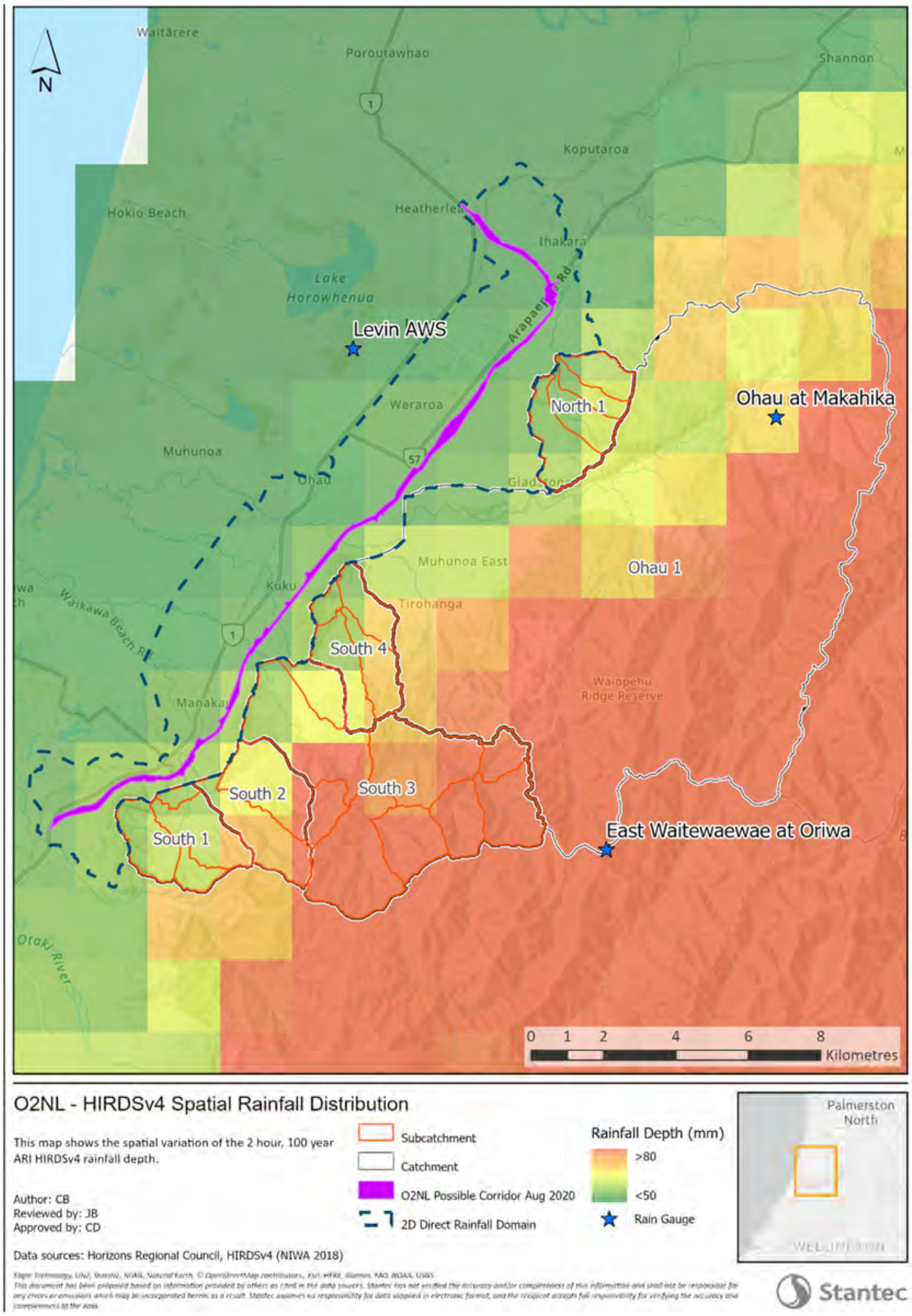


Figure 4-8: HIRDS v4 spatial rainfall distribution (2h 1:100 AEP)

4.2.5 Temporal Profile

Design rain temporal profiles were based on the method outlined in the HIRDS v4 report (Carey-Smith, Henderson & Singh, 2018) using the Western North Island curves. Multiple curves were produced for different storm durations - 1h, 2h, 3h, 4h, 6h. As the 3h and 4h profiles were not standard durations used in the HIRDs v4 method, a linear interpolation was used between the available four curve parameters to obtain temporal curve parameters for the 3h and 4h durations which could then be scaled to the required event rainfall depth. Figure 4-9 below shows the 4h 1:100 AEP temporal profile created from the Western North Island curves, prior to scaling for the various sub-catchment total rainfall depths (Table 4-3, in Section 4.2.8). Since the rise and recession limbs are separate parameter sets, it is not uncommon to see a shape change after the peak, but the correct total event rainfall depth is still applied.

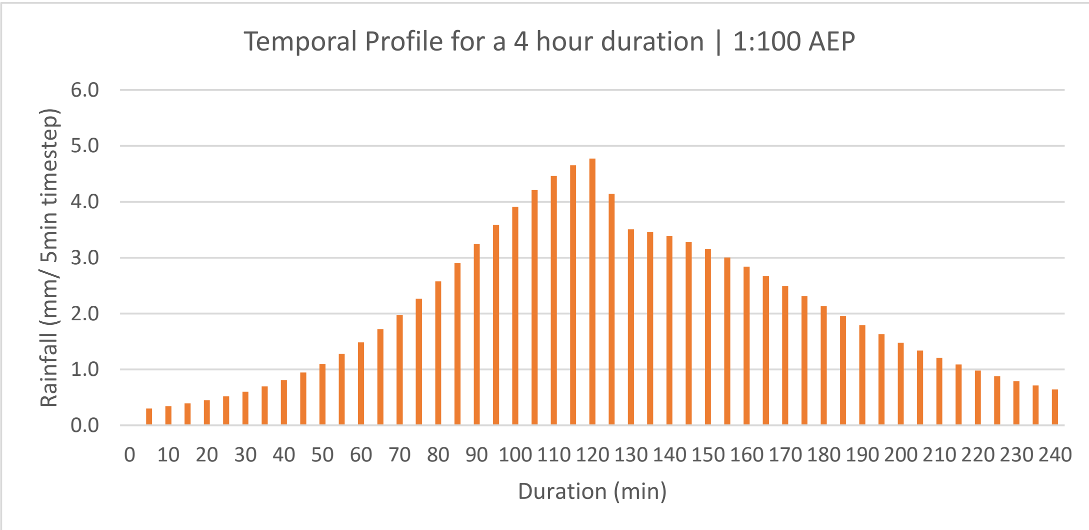


Figure 4-9: 4h temporal rainfall profile for 1:100 AEP

4.2.6 Areal Reduction Factors

An Areal Reduction Factor (ARF) was applied based on each catchment's area for the hydrological model events up to 1:100 AEP. The ARF estimation method was based on the method described in Carey-Smith et al (2018). ARF values ranged from 0.93 (30km^2 Waikawa 1:100 AEP) up to 0.97 (7km^2 catchments 1:10 AEP), and therefore only had a relatively minor impact on the rainfall totals.

4.2.7 Climate Change

HIRDS v4 provides climate change rainfall depths up to the 2081 – 2100 epoch (nominally 2090) and so depths for 2130 were extrapolated. The extrapolation was based on Table 8 in Carey-Smith et al 2018 which gives predicted temperature increase through to 2110. Values were extrapolated using a fitted curve to 2130 as shown in Figure 4-10. As described in section 2.4, RCP6.0 was used for 1:100 AEP event, and RCP8.5 was used for the 1:1500 AEP ULS scenario.

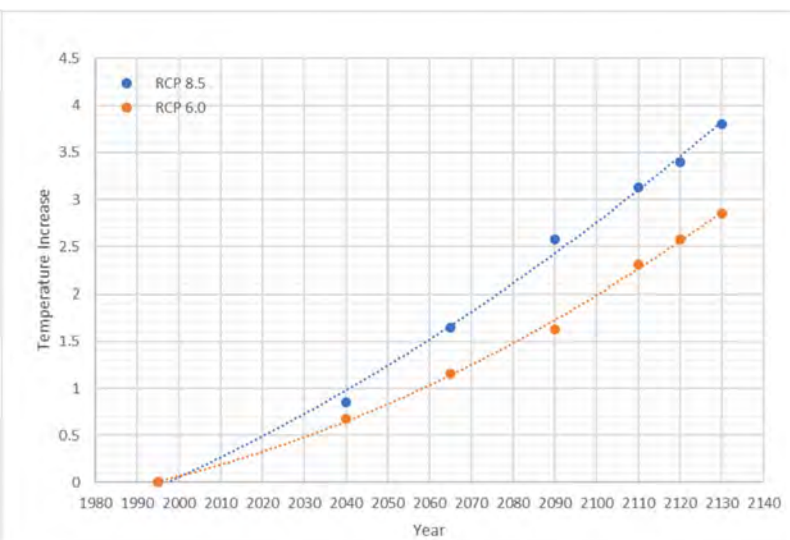


Figure 4-10: Estimated temperature increase to 2130

Percentage change factors per degree of warming were then based on Table 6 of Carey-Smith et al 2018.

To obtain the relevant climate change factor for the 1:1500 AEP RCP8.5 scenario, advice from Trevor Carey-Smith at NIWA (lead author of the HIRDS v4 report) was to use the percentage increase of warming given for the 1:100 AEP event (Table 6 of the HIRDS v4 report), and the extrapolated increase for RCP 8.5 (Table 8 of the HIRDS v4 report).

4.2.8 Summary of HIRDS-based Rainfall Design Event Totals

Table 4-3 below provides a summary of the total rainfall depths for the preliminary design events based on HIRDS and the preceding steps. The main differences in rainfall depths between the various catchments are the orographic effects (influence of higher elevation topography on rainfall) and the critical storm duration (for example the 3h and 1h durations has lower total rainfalls but higher intensity in mm/h compared to a 4h storm). The selection of critical storm duration is discussed in Section 4.4.2. For the 2D rain on grid area, both 1h and 4h durations are presented as these are used for different purposes as discussed later in the report. The conditional colour formatting helps to highlight trends, notably that the Waikawa catchment has substantially more rainfall compared to its neighbouring catchments of Kuku and Manakau. This HIRDS-based data is thought to be influenced by the rain gauge East Waitewaewae at Oriwa, which reports much higher rainfall than Waitatapia at Taungata just a little further south and at similar elevation. This disparity in observations is illustrated by the cumulative chart in Figure 4-5 and in table below.

Table 4-3: HIRDS-based rainfall depth (mm) for each catchment with ARF & climate change

Catchment name (ID)	Sub-catchment name	Critical Duration (h)	1:10 AEP current climate (mm)	1:100 AEP RCP 6 2130 (mm)	1:1500 AEP RCP 8.5 2130 (mm)
Koputaroa (North_1)	Nth_s_1	4	49	**99**	153
	Nth_s_2		47	**97**	150
	Nth_s_3		47	**97**	150
	Nth_s_4		49	**100**	154
Kuku (32)	S4_1	4	51	**103**	159
	S4_2		59	**119**	182
	S4_3		53	**107**	165
	S4_4		47	**95**	147
	S4_5		46	**94**	145
Waikawa (27)	S3_1	4	51	**102**	156
	S3_2		74	**147**	224
	S3_3		80	**159**	242
	S3_4		87	**173**	262
	S3_5		97	**192**	289
	S3_6		88	**175**	264
	S3_7		56	**113**	172
	S3_8		83	**165**	250
	S3_9		68	**135**	207
Manakau (15)	S2_1	4	62	**126**	192
	S2_2		57	**115**	176
Waiauti (14)	S1_1	3	43	**89**	137
	S1_2		44	**89**	138
	S1_3		48	**99**	152
	S1_4		46	**94**	145
2D model rainfall zones	2D_Mesh	1	26	**56**	84
		4	43	**89**	147

The very high design rainfall in the Waikawa catchment produced simulated flows from the calibrated HEC-HMS models (refer section 4.4) that appeared much higher than flood frequency analysis (refer section 4.3). Rainfall duration frequency analysis was therefore undertaken at each rain gauge, and compared to the corresponding HIRDS v4 cell data, for a 4h storm. The comparison is presented in Table 4-4 below. The fitted distributions at Taungata may be slightly low considering its elevation, but at Oriwa appear extremely high and may be unrealistic. This analysis helped to confirm adjustments to the simulated flows from the HIRDS rainfall, in line with the flow gauge flood frequency analyses which are anchored in real observations near the Ō2NL corridor.

Table 4-4: Comparison of HIRDS vs rain gauge frequency analysis for 4h storm

Gauge location (altitude)	4h 1:10 current climate			4h 1:100 current climate			4h 1:10 ratio		4h 1:100 ratio	
	HIRDS cell	Gumbel	Pearson	**HIRDS** cell	Gumbel	Pearson	Gumbel	Pearson	Gumbel	Pearson
Manawatu at Moutoa (5m)	41	38	38	63	53	53	93%	94%	85%	84%
Levin AWS (15m)	40	36	35	59	50	44	90%	89%	85%	74%
Levin MAF (45m)	40	47	48	59	66	75	118%	121%	111%	127%
Mangahao at No1 dam (390m)	86	79	81	130	107	116	92%	94%	82%	89%
Ohau at Makahina (240m)	63	69	70	95	97	102	109%	111%	102%	108%
East Waitewaewae at Oriwa **(1050m)**	**128**	**155**	**161**	**189**	**233**	**287**	122%	126%	124%	152%
Waitatapia at Taungata **(980m)**	**85**	**85**	**84**	**127**	**117**	**109**	99%	99%	92%	86%
Otaki River at Depot (17m)	50	47	46	75	64	55	94%	93%	85%	74%

4.3 Flow Gauges and Flood Frequency Analyses

Available flow gauging station records are summarised in Table 4-5 below. A graph of the data period of each gauge is shown in Figure 4-3 and a map of gauge locations provided in Figure 4-4 in Section 4.2.1.

The data was used to select flood events for calibration of rainfall-runoff models. Flood frequency analysis was also carried out as described below.

Advice from Horizons Regional Council is that the Koputaroa, Waikawa and Manakau stream gauges are primarily for water resource assessment and might not be rated with confidence for flood flows. This is taken into account in the discussions that follow. Accordingly, flood frequency results and calibrated runoff model results based on these gauges were not used on their own but were also compared with other methods including data transfer from donor sites. An overview of the available flow timeseries data is shown in Figure 4-11 below.

The gauging station for the Ōhau at Rongomatane provides a 43-year record with relatively few gaps. The station has a slack line cableway upstream of the site, where flows up to $250m^{3/s}$ have been gauged, and a model used to extrapolate above this. If future design decisions are considered very sensitive to the design flow, then further investigation could be carried out on the confidence of the rating curve and flow data for the highest peaks. Appropriate sensitivity testing and freeboard allowances should still be considered, depending on the level of conservatism required and tolerance of design decisions.

Flow data was also obtained for the GWRC gauge Waitohu at Water Supply Intake, since 1994 (27 years of record). This gauge is just south of the Ō2NL project extent, but serves as a useful gauge with a good record for analogous transfer of data into the project catchments.

Table 4-5: Flow gauges in or near catchments

Name	Site number	Latitude	Longitude	Recording Authority	Start of data	End of data
Koputaroa at Tavistock Rd	32590	-40.59663923	175.337058	HRC	Jan-74	Aug-89
Ōhau at Rongomatane	15321061	-40.66345725	175.3326475	HRC	Jul-78	Mar-20
Waikawa at N. Manakau Rd.	1432008	-40.70775371	175.2335565	HRC	May-06	Jun-20
Manakau at Gleeson Rd	32001	-40.71952056	175.2110066	HRC	Nov-78	May-89
Manakau at S.H.1 Bridge	1432003	-40.72439913	175.2116174	HRC	May-06	Jun-20

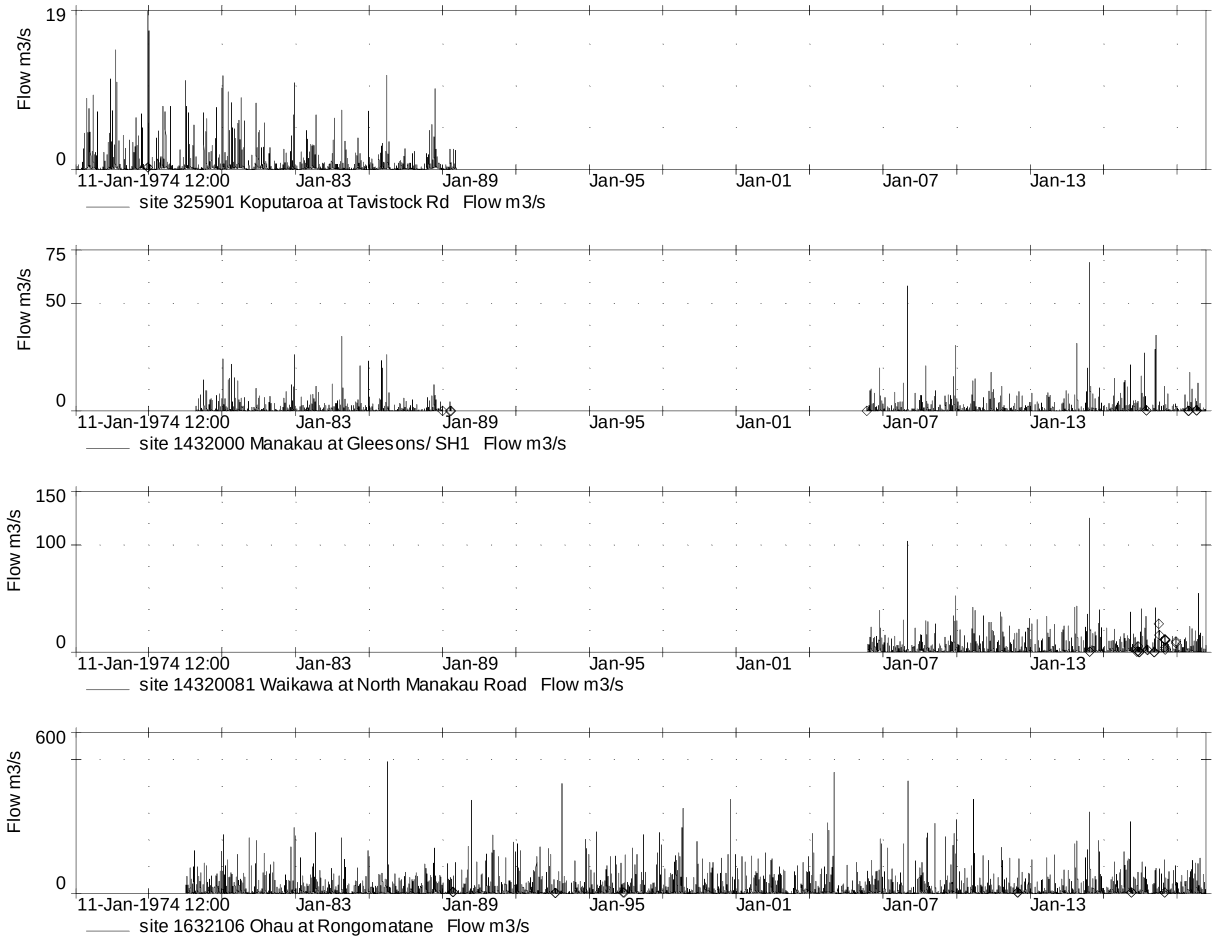


Figure 4-11: Overview of recorded flows

Flood Frequency Analysis (FFA) was carried out for the Ōhau River at Rongomatane. The best estimate for the Ōhau 1:100 AEP flood peak is 559 m^3/s using the full record and a Pearson3 curve. The Pearson3 was only slightly higher than Gumbel and was selected to provide the slightly more conservative peak estimate. The GEV curve appears unrealistically high for extreme (low probability) events and was therefore discarded. Pearson3 was also tested for shorter periods (for example after the 1986 flood to 2020, or 2006 to 2020 to coincide with the period of data at Waikawa). These showed a relatively small spread, with the reduced record producing slightly lower curves.

Previous flood frequency estimates by Horizons Regional Council for the period 1976 to 2006, including testing the impact of including four historic floods from the 1940's and 50's and using a range of fitting formulae gave 1:100 AEP estimates ranging from 545 m^3/s to 656 m^3/s.

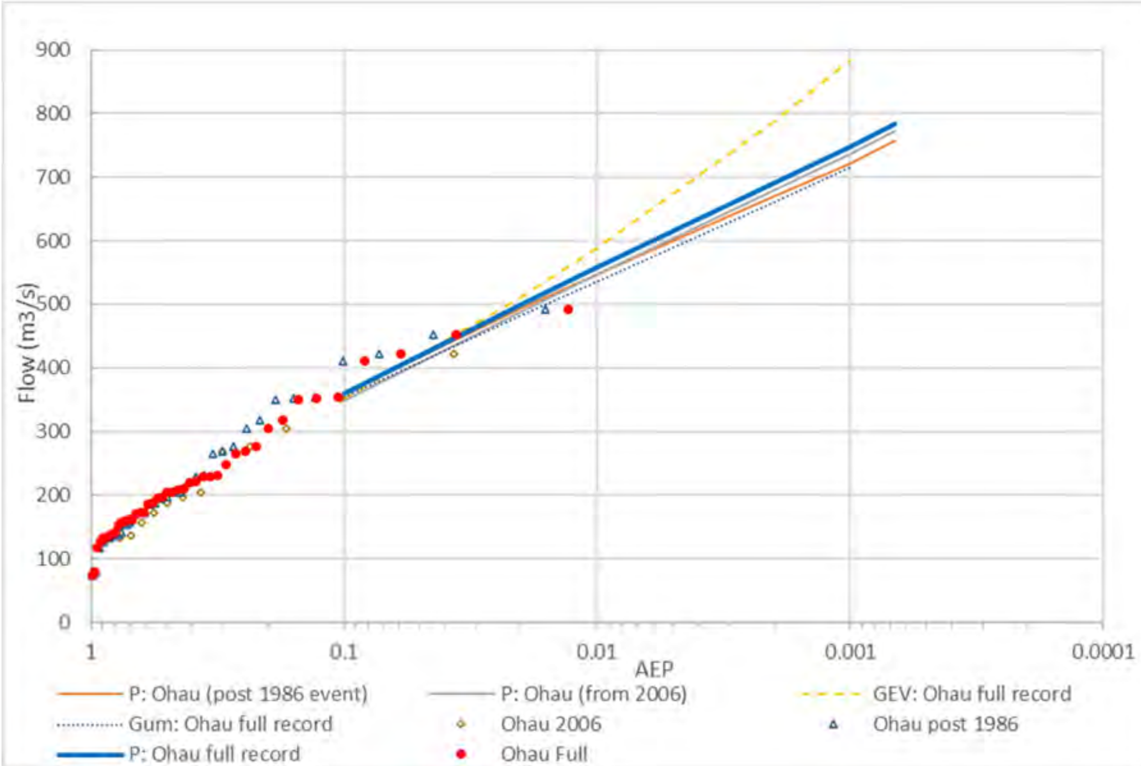


Figure 4-12: Ōhau at Rongomatane flood frequency analysis

The Ōhau peak flows were required for input to the hydraulic model near Muhunoa East Road, approximately 6km downstream of the Rongomatane gauging station. Over this distance, the catchment area increases from 104km^2 at the gauging station to approximately 120km^2 at Muhunoa East Road. The Rongomatane peak flows have therefore been increased by the ratio of 'catchment area to the power 0.9' in line with the North Island relationship in NIWA Regional Flood Estimation Tool for New Zealand Part 2 (Henderson et al, 2018). The same method was used to estimate the cumulative flow downstream of Makorokio Stream confluence (stream ID 33e) that joins just downstream of Muhunoa East Road, so that the correct total flow is found in the model just downstream of the confluence. Hydrograph shapes for the Ōhau at Muhunoa East Road and the Makorokio Stream (stream ID 33e) were both derived from the Waikawa 4h HEC-HMS model (as discussed in 4.4.2 scaled to the respective calculated peak. Climate change was applied based on a multiplier of 1.35 for RCP6.0 2130 and 1.47 for RCP8.5 2130, based on extrapolation of HIRDS data as discussed in 4.2.7.

The results will not be sensitive to slight changes in hydrograph shape or relative timing, because the storage and attenuation in the relatively short length of modelled domain is very small compared to the large design event flow rates.

Flood Frequency Analysis was also carried out on the available data for the Koputaroa, Waikawa and Manakau streams. As mentioned earlier, the primary purpose for these gauges is water resource assessment rather than flood monitoring. However, they are anchored in observations within the catchment close to the Ō2NL corridor and are therefore a valuable information source that is less susceptible to uncertainties in rainfall data (observed and/or HIRDS as discussed previously).

For the Waikawa at North Manakau Road, the Gumbel curve appears to correlate well with distributions scaled from the Ōhau River at Rongomatane and Waitohu Stream at Water Supply Intake. The Pearson3 appears too high as does the GEV. The calibrated HEC-HMS model (refer section 4.4) using the HIRDS design rainfall (refer section 4.2.8) significantly overestimates the preferred data sources (Waikawa 1:10 and transfers from Rongamatane and Water Supply Intake). The HEC-HMS design flows are therefore scaled down to fit the other distributions in a more balanced way and provide balanced yield per km^2 as presented in section 4.5. The scaled HEC-HMS hydrographs are used as inputs to the hydraulic model.

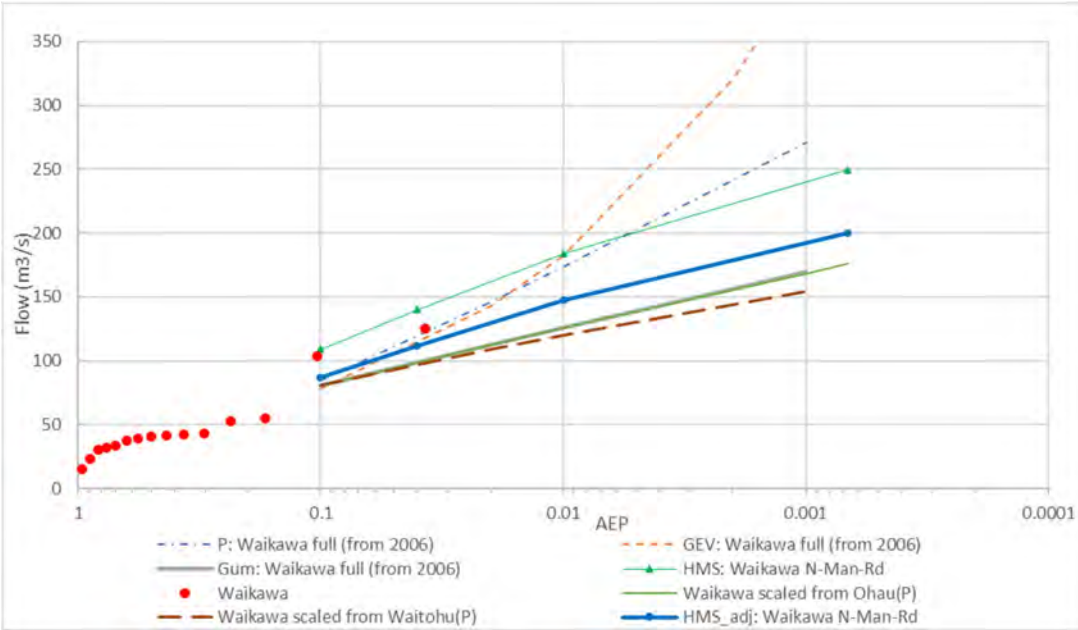


Figure 4-13: Waikawa at North Manakau Road flood frequency analysis

The Manakau Stream has a gauge at SH1, and an earlier period of record at Gleesons Road a short distance downstream. The FFA therefore tested the use of the merged record and the SH1 record only (from 2006). Visual inspection of the timeseries (Figure 4-11) and the FFA suggests that the earlier period at Gleesons Road may have been underestimating flows. The scaled distributions from Ōhau River at Rongomatane (Pearson), Waikawa at North Manakau Road (Gumbel) and Waitohu Stream at Water Supply Intake all show a reasonable correlation but might not account for the slightly lower rainfall within the slightly lower elevation Manakau catchment. The Manakau (from 2006) GEV is once again too high, especially for the low probability events. The Manakau (from 2006) Pearson distribution appears to correlate well with the preferred Pearson curve from Waitohu at Water Supply Intake, especially for the 1:100 AEP which is a key input to the design process. The calibrated HEC-HMS model (refer section 4.4) using the HIRDS design rainfall (refer section 4.2.8) significantly underestimates and has therefore been scaled up to match more closely to the Pearson distribution and the scaled distribution. The scaled HEC-HMS hydrographs are used as inputs to the hydraulic model.

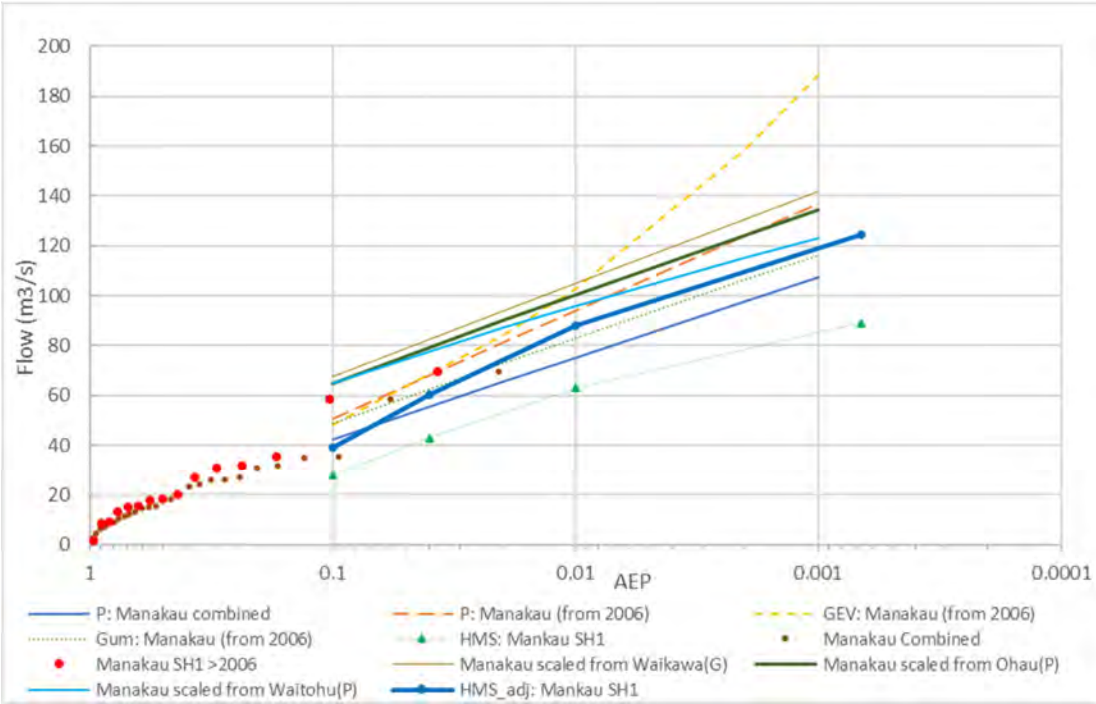


Figure 4-14: Manakau at Gleesons/SH1 flood frequency analysis

The FFA for the Koputaroa Stream at Tavistock appears to provide unrealistically low catchment yield (runoff per km^2) compared to other catchments and other methods (HEC-HMS model and the Regional and Rational methods). This may be on account of the drowning of the gauge due to the low channel capacity downstream. The maximum gauged flow at the gauging station is only 9m^3/s, which is in the order of 1:3 AEP. The FFA based on gauge data is therefore considered unreliable, and the HEC-HMS model with HIRDS design rainfall is adopted as the preferred method. The HEC-HMS model as discussed in section 4.4 and Appendix C is calibrates reasonably against low gauged events that are between 5-10m^3/s. Despite the disparity in observed rainfall frequencies between the two gauges in Levin (refer Table 4-4), in the absence of other more liable data it is reasonable to assume that HIRDS v4 is a reasonable approximation for design rainfall as it will be less susceptible to orographic influences for the Koputaroa catchment. The catchment yield (runoff per km^2) for the adopted HEC-HMS model appears reasonable by comparison to other methods and comparable locations (when adjusting for differences in rainfall).

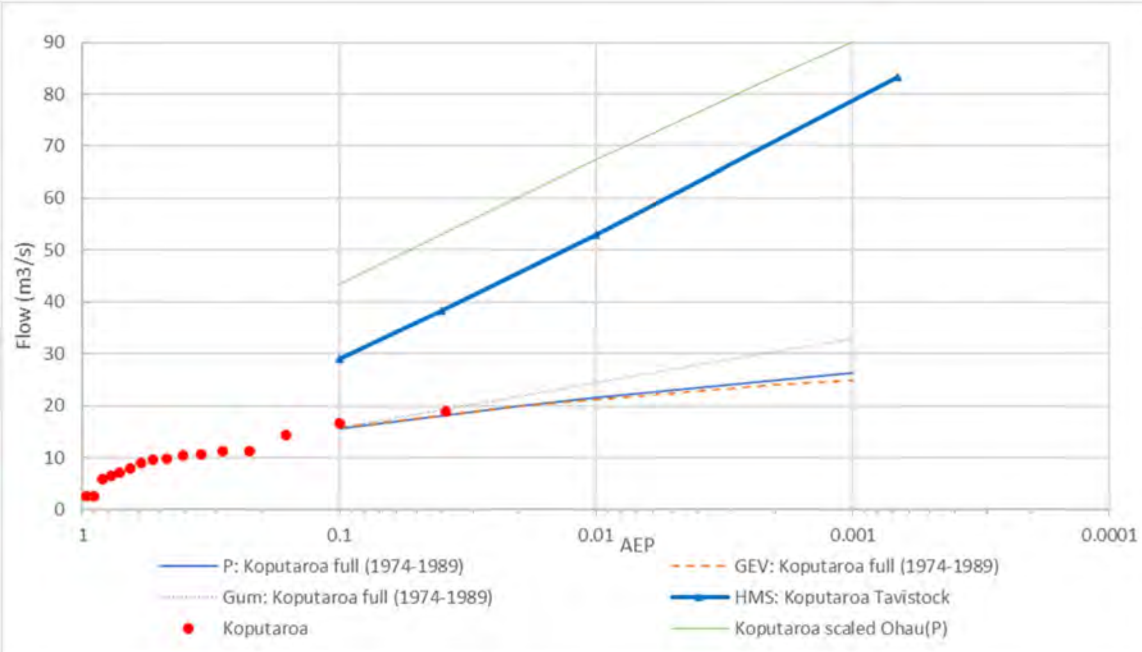


Figure 4-15: Koputaroa at Tavistock Rd flood frequency analysis

4.4 Rainfall – Runoff Model

4.4.1 Model Build

The rainfall – runoff modelling software Hydrologic Modelling System (HEC-HMS), version 4.6.1, was used to simulate precipitation-runoff. The software is developed by the Hydrologic Engineering Center within the U.S. Army Corps of Engineers and is used widely within New Zealand and internationally. It was applied for the Manakau Stream to SH1 (including sub-catchments for Waiauti / South 1 and Manakau / South 2), the Waikawa Stream (South 3), Kuku Stream (South 4), and Koputaroa Stream (incorporating sub-catchments North 1, and sub-catchment 39 used as a check point for the 2D model). Schematics for each model are provided in Appendix B. An example of the Waikawa (South 3) model is featured in Figure 4-16 below. The North Manakau Road gauge is at the outlet of S_7. The sub-catchment S_1 represents stream id 27.1 which enters the Waikawa downstream of the gauge and is a separate input to the hydraulic model.

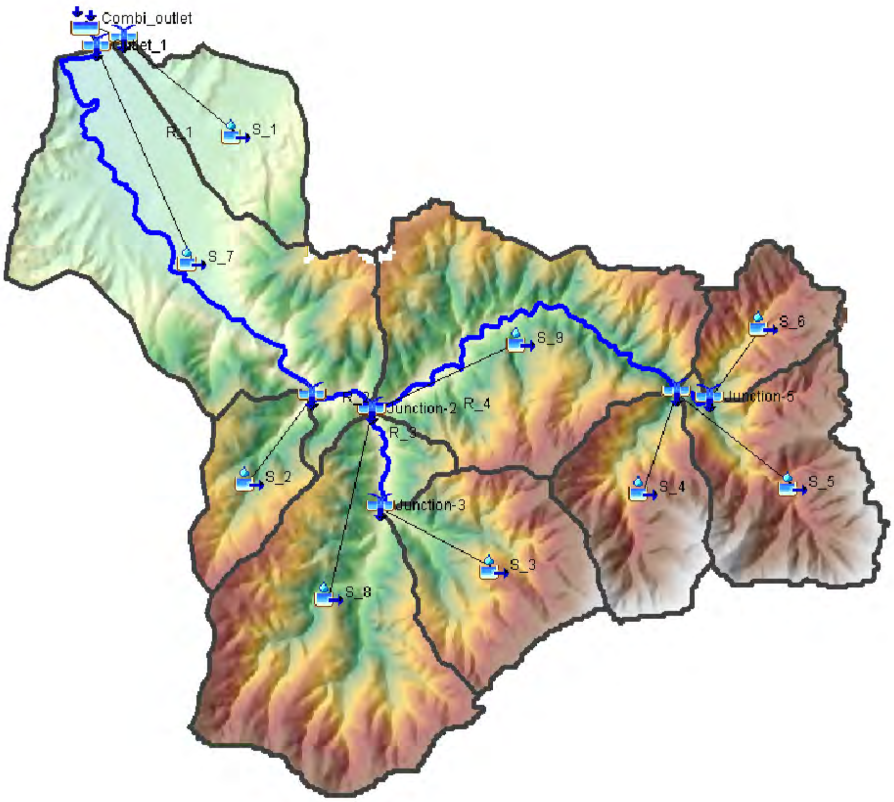


Figure 4-16: HEC-HMS model schematic example of the Waikawa (South 3) catchment

The following is a summary of final hydrological model input parameters after the calibration had been confirmed as acceptable (as discussed in 4.4.3 and Appendix C):

Loss: Initial and Constant

The initial and constant losses were informed by the Fundamental Soil Layer Drainage Class map from Land Resource Information Systems Portal (Appendix D). Whilst there is some variation in drainage class, the calibration and validation is satisfactory with a consistent set of runoff parameters across all hydrological models (refer to Section 4.4.3):

Initial Loss = 7mm

Constant Loss = 5mm/h

(Note that the application of Initial and Constant Losses over the 2D direct rainfall model zones in the HEC-RAS hydraulic model is discussed separately in Section 5.6).

Transform: Clark Unit Hydrograph

Initially, default parameters were used and then adjusted based on a comparison with other methods and calibration, to reach the following:

Time of Concentration = Bransby-Williams

Storage coefficient = Time of Concentration * 3

Routing: Muskingham-Cunge

For the HEC-HMS models used as input boundaries for the hydraulic model, the Muskingham Cunge method was used to calculate the routing up to the point required for the hydraulic model. For the separate models that were solely used for model validation (Koputaroa & Manakau Gauge catchment), a simpler Lag routing method was used for the downstream reaches. This was tested and compared with the Muskingham Cunge method and was not found to make a significant difference. The net effect was also confirmed by comparison with the observed validation event hydrographs.

Temporal rainfall profiles

The temporal profiles mentioned in Section 4.2.5 were used to provide the rainfall hyetograph for the design events, with an override total rainfall depth applied to each sub-catchment as per Table 4-3 (Section 4.2.8).

4.4.2 Critical Storm Duration

Each HEC-HMS modelled catchment was initially run with 1:100 AEP current climate rainfall with various storm event durations from HIRDS data and the critical duration selected that resulted in the greatest peak modelled flow. See Figure 4-17 below for an example of a hydrograph for Waikawa (South 3) showing the critical duration as 4 hours. Once each catchment's critical duration was found, each model was then tested with 1:10 AEP current climate with various durations to confirm that the chosen duration was still applicable. Refer to Table 4-7 in Section 4.5 for final critical durations used in the model.

Figure 4-17 below shows the varying durations used to find the critical storm duration for Waikawa (South 3) catchment.

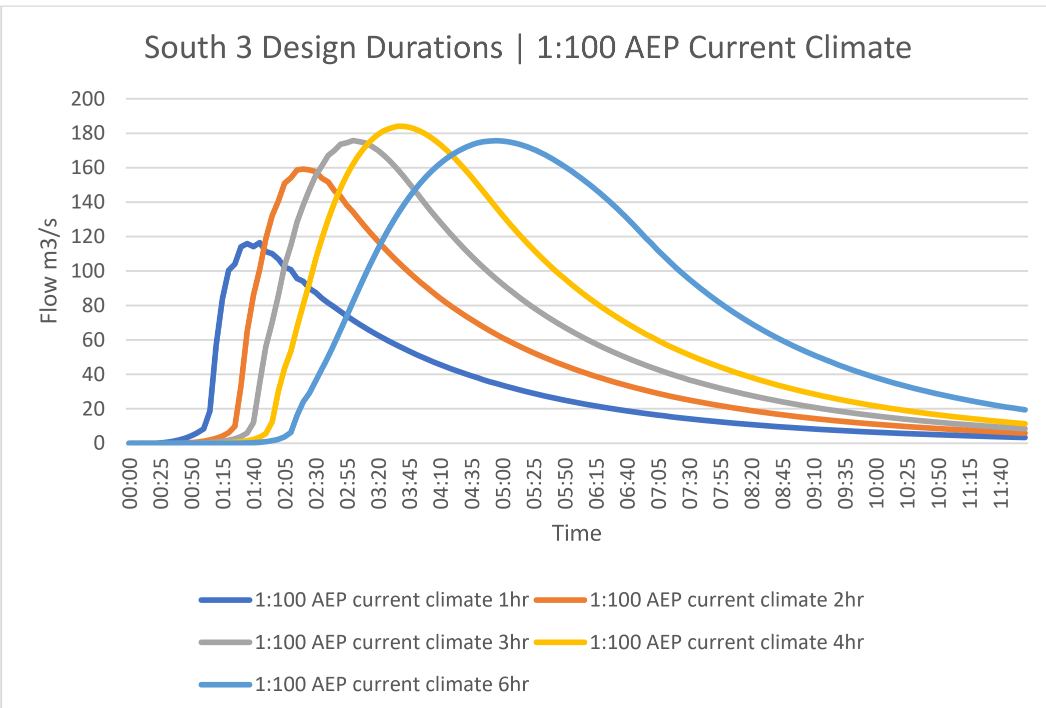


Figure 4-17: Design durations for the Waikawa catchment for 1:100 AEP current climate

Note that it was not required to calculate critical duration for the Ōhau River catchment, since peak flows were based on Flood Frequency Analysis (which uses peak values only). Historical observed flood hydrographs were analysed for the Ōhau and Waikawa gauges. Both were found to have a similar rapid response to short duration rainfall. A comparison of the timing of the Waikawa and Ōhau is shown below for the December 2009 event. A 4h storm hydrograph shape was therefore used for the Ōhau catchment, based on the hydrograph shape generated from the Waikawa HEC-HMS model for a 4h storm and scaled to the Ōhau target peaks derived from statistical flood frequency analysis. The relatively short length of the Ōhau being modelled relative to the large peak flows means that selection of hydrograph shape will have minimal impact on transmission of the hydrograph shape through the modelled reach.

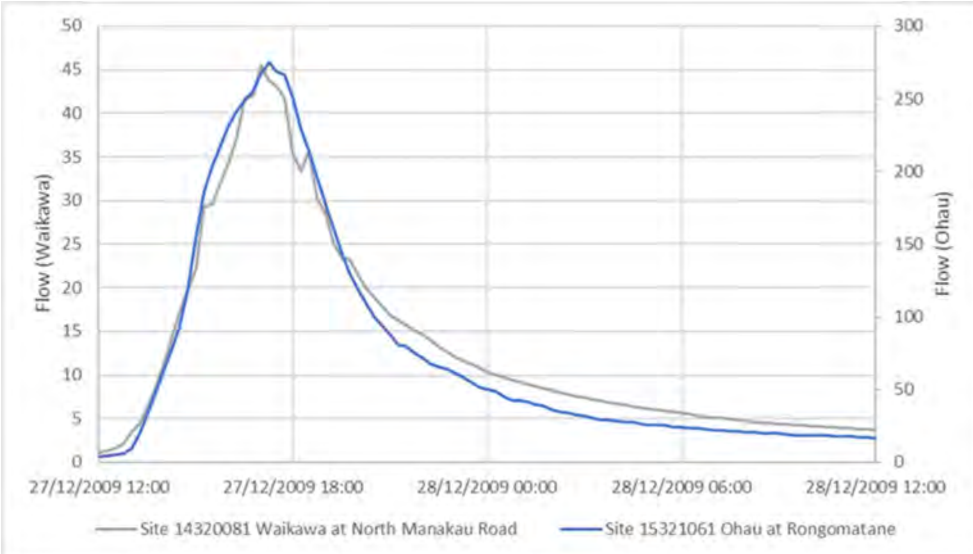


Figure 4-18: Observed hydrograph timing comparison, December 2009 event

For the direct rainfall zones applied in the 2D hydraulic model, the selection of storm duration and their application in the hydraulic model is discussed in Section 5.6).

4.4.3 Model Calibration

As described in Section 4.3 and Figure 4-2, three catchments & their respective flow gauges were used to aid in the validation of the hydrological models. The catchments used for validation are:

- The Koputaroa Stream at Tavistock Rd flow gauge
- Manakau at SH1 flow gauge
- The Waikawa at N Manakau Rd flow gauge

The fact that a consistent set of parameters was found to fit well at three flow gauges added more confidence to the validation process than only regarding a single model in isolation. A full set of results is presented in Appendix C.

An example of recorded and modelled flow for the Waikawa gauge is shown in Figure 4-19 below.

At least four events were selected for each gauge, due to some uncertainties in spatial rainfall coverage. Rainfall gauges were used where data was available for the selected events. Thiessen polygons were created in ArcGIS and were weighted when applied to sub-catchments in the model, as illustrated in Figure 4-6. It may be technically feasible to improve the catchment average rainfall by using rainfall radar and/or by using a terrain-sensitive rainfall surface fitting approach that accounts explicitly for elevation. However, as shown in Appendix C, a reasonable fit between the modelled and observed flows has been achieved, with reasonable explanations for those events with poor fit based on spatial distribution of rainfall. Additional checks were also made on the HEC-HMS peak flows, using the gauged flood frequency analyses (refer section 4.3) and additional independent methods (refer section 4.4.4). The calibration of the HEC-HMS models shown in Appendix C is considered reasonable for historic events but required adjustment for the design events as outlined in section 4.3 mainly on account of shortcomings in the HIRDS v4 design rainfall.

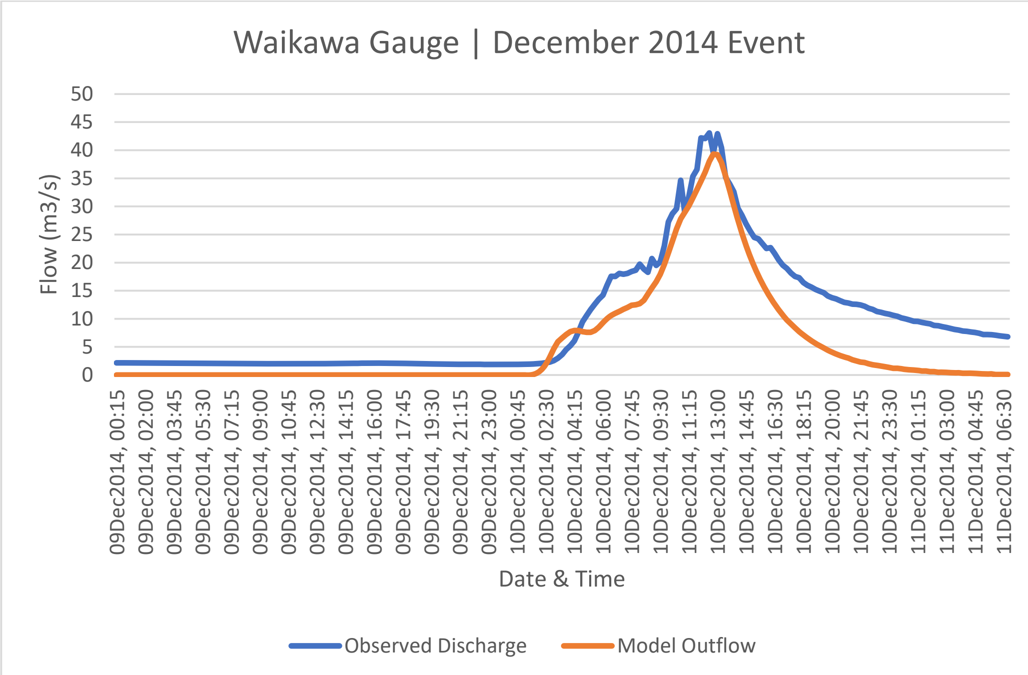


Figure 4-19: Simulated and observed flows at Waikawa gauge

4.4.4 Comparison of Results versus Rational and Regional Estimates

A 1:100 AEP event rational and regional method peak flow was calculated for selected catchments as additional independent checks on the outputs of the FFA and HEC-HMS models. Table 4-6 lists the results for each catchment. The Rational Method calculation is usually the preferred method for catchments less than 10km². The Regional Method (McKerchar and Pearson 1989)[3] is usually preferred for catchments over 10km². The online NIWA Flood Frequency Tool (Griffiths et al)[4] was also used as another regional calculation approach but was found to give inconsistent results and is therefore not presented.

Rational Method estimates were based on the following formula:

Peak Flow (m³/s) = C (unitless) I (mm/h) A (m²)/(3600*1000)

Where C is the runoff coefficient, I is the rainfall intensity for a storm duration corresponding to the catchment time of concentration (ToC) and a given AEP, and A is the catchment area. The runoff coefficient of 0.3 was estimated based on the NZ Building Code guidance – E1 Surface Water.

Table 4-6: Comparison of with rational and regional method (1:100 AEP current climate)

Catchment Name	Area (km²)	Rational (m³/s)	Regional (m³/s)	FFA (m³/s)	Raw HEC-HMS (m³/s)	Adj HEC-HMS (m³/s)
Sth 1 (Waiauti)	7.2	27	19	-	28	39
Sth 2 (Manakau)	7.1	35	20	-	30	42
Sth 3 (Waikawa)	**29**	106	68	126	184	147
Sth 4 (Kuku)	7.5	32	16	-	26	31
North1	7.5	36	20	-	23	23

Regional Method results are significantly lower than the Rational Method which is not unexpected. The significant difference in results from these methods for the Waikawa catchment is thought to be due to the steep rainfall gradient within this catchment, some of which may result from the extremely high (possibly unrealistic) rain gauge records at Oriwa. Regional Method contour resolution in this area is too low to account for the steep change in rainfall across the catchment.

[3] McKerchar, A.I., Pearson, C.P. (1989) Flood Frequency in New Zealand. Publication of the Hydrology Centre, No. 20: 87.
[4] https://niwa.maps.arcgis.com/apps/webappviewer/index.html?id=933e8f24fe9140f99dfb57173087f27d

A summary of the adopted peak flows is provided in Section 4.5 below.

4.5 Summary of Adopted Peak Model Flows

The HEC-HMS hydrographs scaled where applicable, provided the runoff hydrographs for input to the hydraulic model, with application is discussed in Section 5. The peak flows provided for the hydraulic model are shown in Table 4-7 below. The second part of the table is the specific yield (peak / area^0.9) for each inflow. A map of modelled inflow catchments is presented in Figure 5-1.

Table 4-7: Summary of peak flow results for each catchment

Inflow	Catch Area km²	Critical Duration	1:10 AEP current climate	1:100 AEP RCP 6.0 2130	1:1500 AEP RCP 8.5 2130
			Peak flows (m^3/s)		
Waiauti 14	7.2	3h	21	54	90
Manakau 15	7.1	4h	24	57	92
Waikawa 27	29	4h	91	191	302
Waikawa trib 27.1	1.8	4h	5	11	17
Kuku 32	7.5	4h	18	43	71
Makorokio 33e	11.5	4h	35	74	113
Ōhau 33	120	4h	411	861	1315
North1	7.5	4h	13	32	54
	Method summary		Specific yields (peak/area^0.9)		
Waiauti 14	HEC-HMS(HIRDS)*1.4		3.6	9.2	15.2
Manakau 15	HEC-HMS(HIRDS)*1.4		4.2	9.7	15.8
Waikawa 27	HEC-HMS(HIRDS)*0.8		4.3	9.1	14.4
Waikawa trib 27.1	Above scaled to cumulative catch increase		2.9	6.2	9.8
Kuku 32	HEC-HMS(HIRDS)*1.2		2.9	7.1	11.6
Makorokio 33e	Ohau FFA scaled to cumulative catch increase		3.9	8.2	12.5
Ōhau 33	Ohau FFA scaled to cumulative catch		5.5	11.5	17.6
North1	HEC-HMS(HIRDS)		2.1	5.3	8.8

Several different methods have been compared to inform and derive the above inflows, with a strong weighting applied to gauged flow data. Given the usual residual uncertainties in hydrological probability, it would be prudent to consider the tolerance of design decisions to the accuracy of the hydrology. Testing the possible impacts of exceedance events will be done using the ULS 1:1500 AEP RCP8.5 2130 scenario, which will help to show if/where the proposed design may be more sensitive to hydrological inputs.

5. Hydraulic Modelling and Assessment

5.1 2D Hydraulic Model Approach

HEC-RAS version 5.0.7 is used to model the flood inundation modelling and mapping in this project. The model is a widely used flood modelling tool for hydrodynamic simulation, and it is designed to perform both 1D steady and unsteady flow simulations in addition to 2D unsteady flow simulations for river and floodplain flow analysis. The program was developed by the US Army Corps of Engineers Hydrologic Engineering Centre (HEC). The model is commonly used and widely accepted by industry and has the following advantages that are applicable to this project:

- Can perform 1D, 2D, and combined 1D and 2D modelling, including rain on grid.
- Saint-Venant or Diffusion Wave Equations in 2D.

- Implicit Finite Volume Solution Algorithm.
- Structured or Unstructured Computational Meshes.
- Detailed Hydraulic Table Properties for 2D Computational Cells and Cell Faces derived from fine underlying DEM.
- Detailed Flood Mapping and Flood Animations.

Two-dimensional component of the model allows water to move in both longitudinal and lateral directions, while velocity is assumed to be negligible in the z-direction. However, unlike 1D models, 2D model represents the terrain as a continuous surface through a mesh or grid. To improve the computational time, HEC-RAS uses a sub-grid approach, which uses a relatively coarse computational grid based on finer scale information from the underlying the topography. The fine topographic grid informs the flow calculations between computational mesh cells. Mesh refinements are applied in the areas of the watercourses or other features of interest, to constrain the computational mesh to an appropriate size. Mesh size can be reduced or enlarged by the modeller to suit the terrain and features that influence the hydraulic calculations.

The HEC-RAS suite has geospatial editing tools that allow for full development of geometric data for hydraulics models, including analysis of terrain data, developing geometric data, refining model layout, and visualizing results directly within HEC-RAS. This makes the process of river hydraulic modelling efficient without reliance a standalone GIS pre- and post-processor. Within the software, hydraulic model development process begins with terrain model and continues with an interpretation of the land surface and elevations, establishing hydraulic model elements, and enters a cycle of iterating between model simulation, analysis of results, and model refinements. Tools assist in the process of creating a terrain model and modifying incorrect elevation data. Visualization of the elevation data along with aerial imagery in HEC-RAS then allows for laying out 1D modelling objects (such as culverts or weirs where available) and creating 2D Flow Areas. Mapping of hydraulic results allows quick identification of model deficiencies for improvement. 2D Flow Area mesh refinement tools and terrain modification capabilities let the hydraulic modeler efficiently improve model geometry and simulation results. The software allows for a detailed assessment of the results both globally and locally to points of interest.

Three different models were created as follows.

- South Model. A single 2D model for all watercourse crossings south of the Ōhau River to Taylors Road near Ōtaki (i.e. the northern extent of PP2Ō). Point inflow hydrographs include the Waiauti Stream (South_1), Manakau Stream (South_2), Waikawa Stream (South_3) and Kuku Stream (South_4).
- Ōhau Model. A 2D model of the Ōhau River and adjacent floodplain. A single point inflow hydrograph is applied for the Ōhau River (Ōhau_1).
- North Model. A single 2D model for all watercourses north of the Ōhau River to the intersection of SH1 and Kaputahoa Road, north of Levin. One of the sub-catchments north east of Levin, that drains past near the corridor towards Koputaroa, is represented as a point inflow hydrograph (North_1) to reduce the size of the computational mesh.

Figure 5-1 shows the extents for each model. The models extend approximately 2 km downstream and 1 km upstream of the proposed Project, so that results in the vicinity of the proposed project are not sensitive to boundary effects. For the larger upstream catchments approaching from the east, the model uses point inflows as derived in chapter 4 (Hydrology) of this report. For the 2D domain, additional rainfall is applied directly over the model domain.

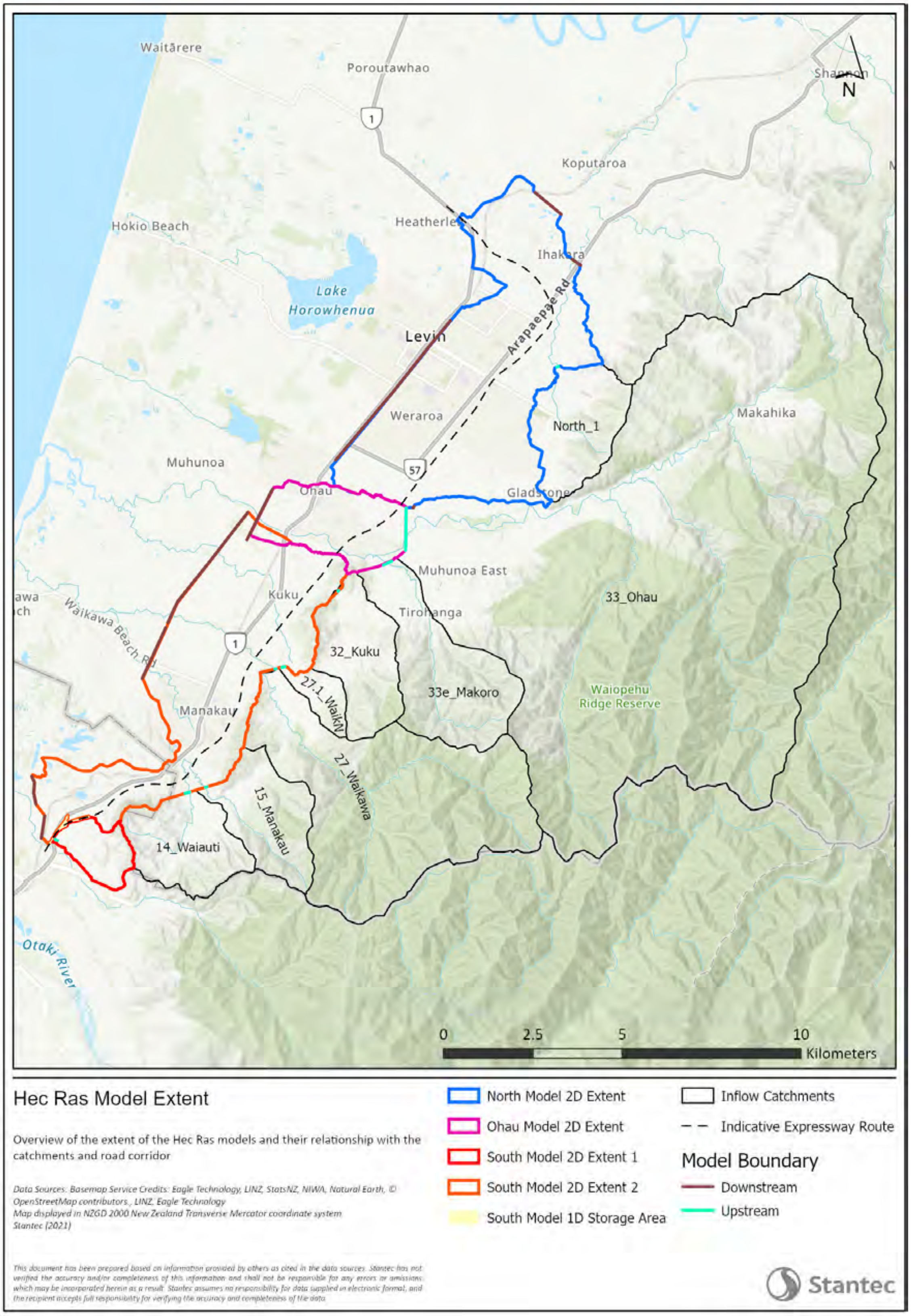


Figure 5-1: HEC-RAS 2D model extent for the three 2D hydraulic models

5.2 Digital Elevation Model Preparation

5.2.1 Reference Digital Elevation Models

Different DEM datasets were evaluated to prepare the terrain model within HEC-RAS. A key requirement was for the DEM domain to provide full coverage of the required 2D zones / catchments where direct rainfall modelling would be applied. As shown in Figure 2-1, the required model coverage extends well beyond the 2020 drone DEM, meaning that the 2020 drone DEM on its own would not be sufficient, and the step changes between datasets would require careful management.

In the areas of overlap, the 2020 drone DEM showed significant differences from the Regional Councils LiDAR DEM, both in terms of moving average or trends (apparent vertical shifts, although not continuous in nature) and in terms of large, localised errors. This is illustrated graphically in Appendix F. The 2020 drone DEM was more recent and higher resolution but being based on photogrammetric techniques appears poorer than the filtered Regional Council LiDAR DEM in vegetated watercourses and floodplains which are key flow paths. This may be because the LiDAR had managed to achieve better penetration of the vegetation. Away from the watercourses, the 2020 drone LiDAR higher resolution data also included many high points, possibly due to trees, fence poles, etc. Even when downsampled to a 1m x1m grid using the minimum elevation in each grid cell, there was still considerable noise or scatter in the terrain which could cause anomalous results in the direct rainfall modelling. It would have required considerable additional editing of the 2020 drone DEM and the step changes on its boundaries before achieving an overall DEM suitable for hydraulic modelling. On balance of these factors, it was therefore decided to use the Regional Council LiDAR DEM for the hydraulic modelling, with the following additional actions taken to mitigate the possible effects on the Ō2NL geometric design process which was being based on the 2020 drone DEM:

- Minor improvements to the 2020 drone DEM where particularly large errors were found relative to the Regional Council LiDAR DEM in watercourses, such as the example shown in Appendix F, to provide a more realistic ground surface to support the design process.
- Extensive minor improvements to the Regional Council LiDAR DEM to reduce or remove obstructions to flow paths, as discussed in Section 5.2.3 below, to reach an overall DEM more suitable for representing the passage of water. Some residual DEM issues remain as outlined in Section 5.2.4 below, although these were determined to be of minimal impact to the results within or near the Ō2NL corridor for calculating water levels and Assessment of Effects.
- Modelled depths (rather than water surface elevations) within the Ō2NL corridor are provided to the geometric design team, to apply above their improved ground surface where water levels are required.
- Proposed culverts created within the geometric design model will be modified in the hydraulic model to meet the HEC-RAS channel inverts when performing the calculation of post-scheme water depths and assessment of effects.

Once the decision had been made to retain primary use of the Regional Council LiDAR datasets, the model DEM preparation for each model extent involved clipping and/or mosaic (merging) of the Regional Council datasets into a single seamless raster file. Each of the model DEM files is 1mx1m horizontal resolution, meaning that there is an elevation value for every 1 metre travelled along the ground surface. The DEM files are all projected to the "NZGD 2000 Transverse Mercator" coordinate reference system. All DEM's and model rasters are in Wellington 1953 datum to provide a consistent set of outputs for the project. Further processing steps are outlined in the following sections.

5.2.2 Model Meshing

For all models, a terrain file was created slightly larger than the proposed model extents. For each model different computational mesh cell spacing criteria were used to create the initial 2D modelling surface. Mesh refinements are applied to limit the cell size around certain features, such as roads, watercourses or structures. The maximum cell size is lower near the project corridor, with additional mesh refinements being applied for spatially sensitive features in the with-scheme model being re-applied within the final baseline model. This ensures that the final baseline and with-scheme models are run with the same computational mesh, to minimise the slight differences that can otherwise be introduced when comparing results from different computational meshes. The values shown on Table 5-1 are the results of iterations to reduce the mesh generation errors whilst achieving a reasonable accuracy and computational time.

Table 5-1: Hydraulic model maximum 2D cell spacing

	Model South	Model Ōhau	Model North
2D Flow Area (maximum spacing in m^2)	20	20	20
Refinement Zone nearer to corridor (m^2)	10	-	10
Break lines Structures (m^2)	5	5	5
Break lines Road (m^2)	8	8	8
Break lines on ditch (in refinement zone) (m^2)	5	-	5
Break lines on ditch (outside refinement zone) (m^2)	15	15	15

The smaller cell spacing in the refinement zone and near hydraulic features allows for more detailed results through critical flows areas. Information from the 1m grid is used to inform the computational mesh calculations, providing a high degree of accuracy. An extract of the model mesh is provided in Figure 5-2 below to illustrate the concept of computational grid refinements on regular mesh and around hydraulic features.

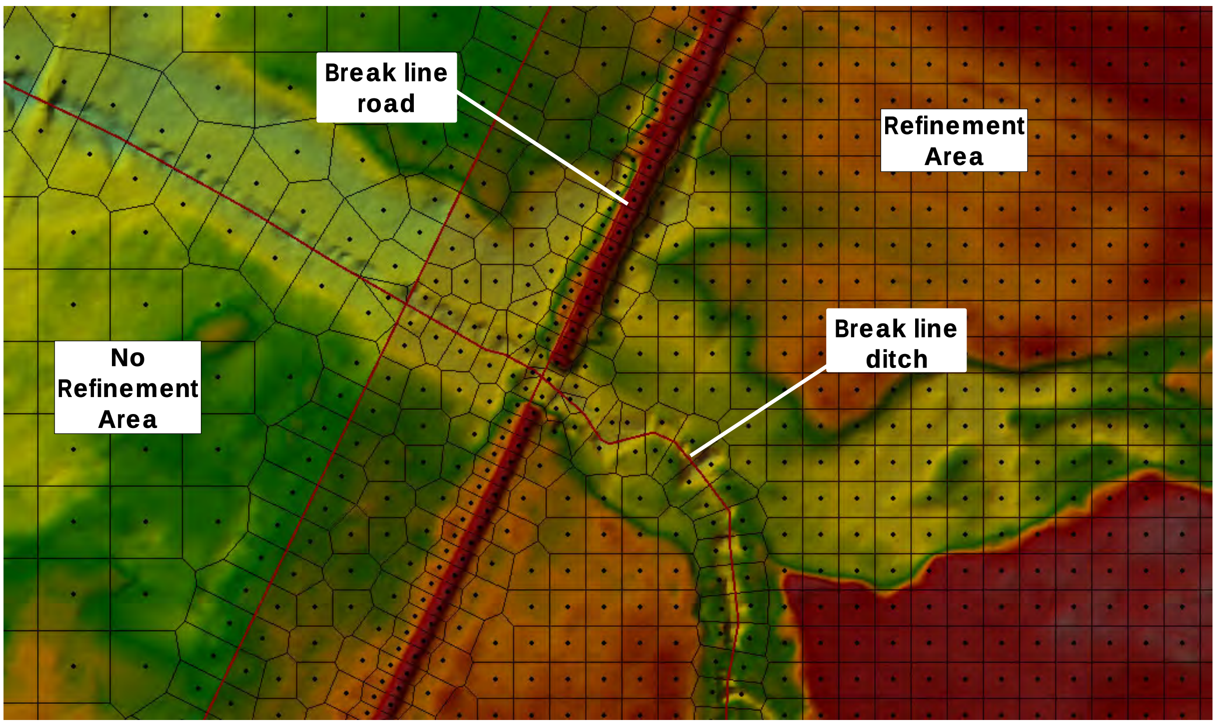
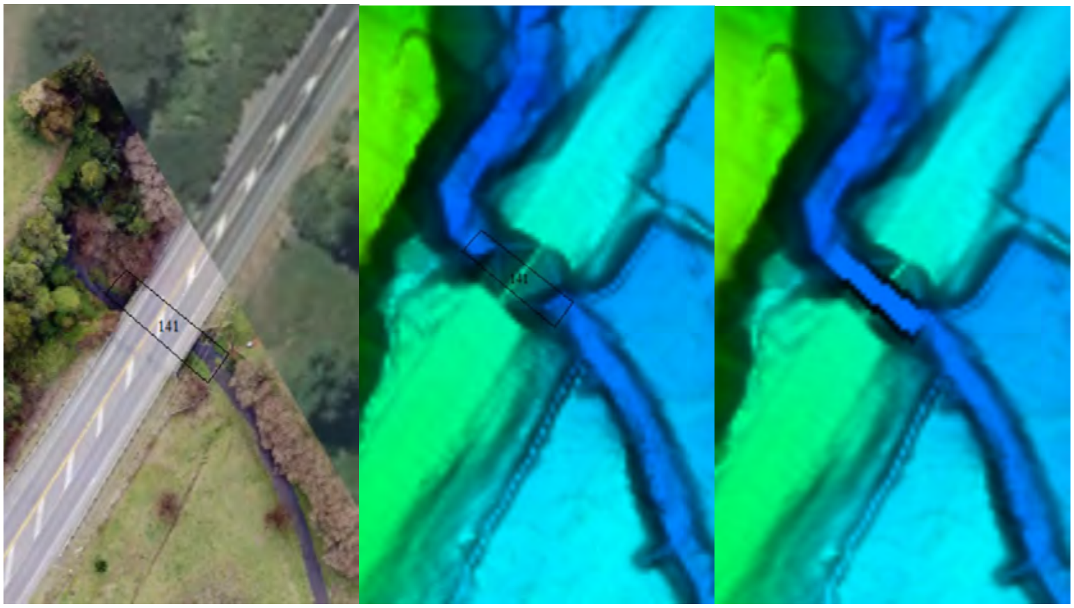


Figure 5-2: Example of model mesh refinements

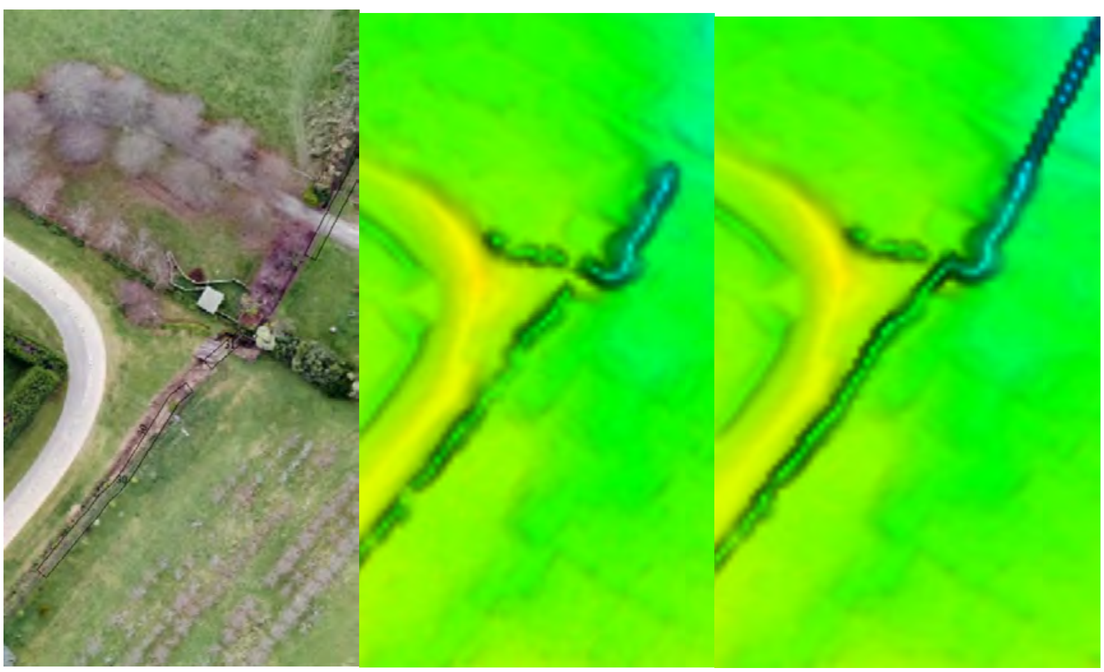
5.2.3 DEM modifications

To provide stream continuity on the flow surface, the terrain model was burnt down mainly at small bridges outside the area of interest or farm access tracks where there is not information available. The burning process was applied using GlobalMapper, by creating a polygon on the intersection between the stream and the road, then the polygon vertices were edited to match with the lowest nearby level in the DEM. The polygons were turned into a raster and stamped over the model DEM. Figure 5-3 below shows an example burning process on Koputaroa bridge (Model North) and Figure 5-4 shows the burning process in a ditch close to Waihou road. Approximately 200 DEM modification polygons were created to improve the three model DEM's.



Aerial Before burning After Burning

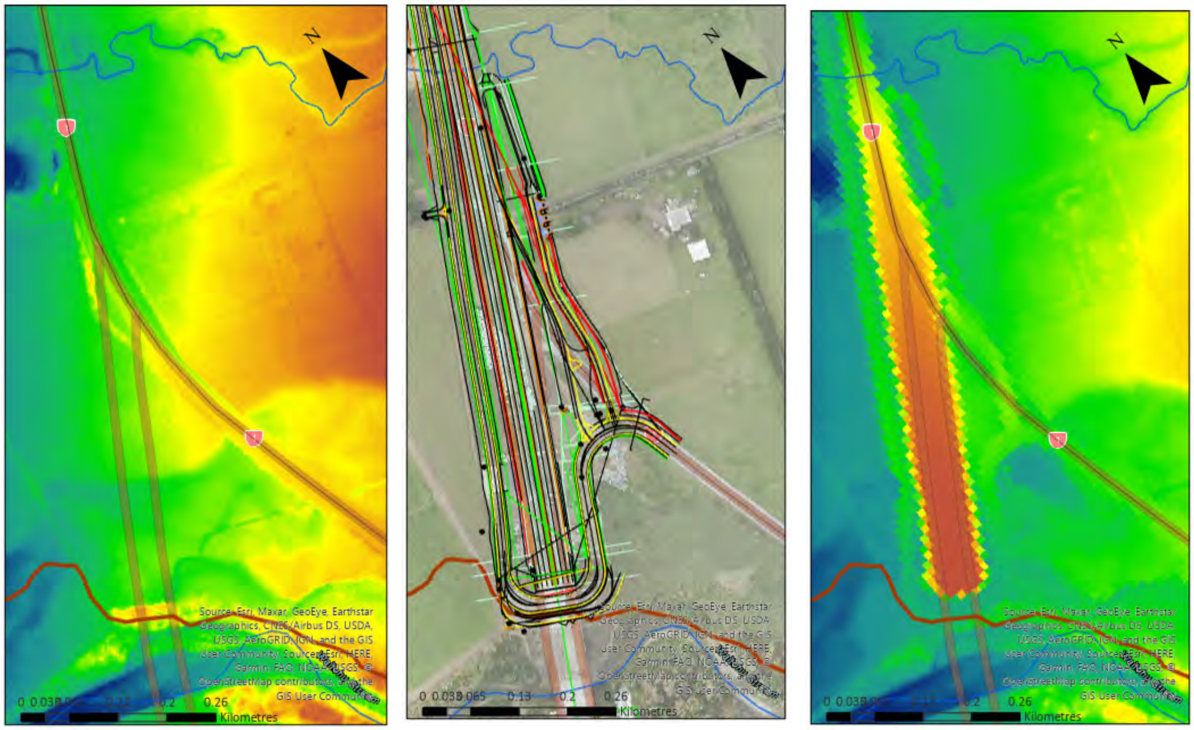
Figure 5-3: Example DEM burning process for a bridge on Koputaroa Stream.



Aerial Before burning After Burning

Figure 5-4: Example burning process for a ditch near Waihou Road

At the southern end of the South model, the northern end of PP2Ō Project is currently under construction. The baseline for the Ō2NL assessment should therefore include the PP2Ō as-built information. Since the PP2Ō was not completed at the time of the regional council LiDAR, it was necessary to stamp the PP2Ō as-built and proposed design information onto the South model domain as shown in Figure 5-5 below. The PP2Ō embankment terrain model was extracted from the PP2Ō hydraulic model files. The unmodified terrain (in flat areas) was consistent with the current DEM, therefore no datum adjustments were made.



Original LiDAR | PP2Ō As Built drawing | Terrain after stamped PP2Ō embankment

Figure 5-5: Adding PP2Ō embankment onto updated baseline terrain

5.2.4 DEM issues

Analysis of the LiDAR DEM provided by the regional councils (and used for the model DEM preparation) showed some additional residual inaccuracies. The HRC DEM contained some significant step discontinuities in data along features of significant length, most likely as a result of HRC merging different datasets. This also affected the SH1 road upgrade near near Gleeson Road Manakau, which was not fully evident at the time of the LiDAR capture but is visible as completed in more recent aerial photographs. Unfortunately, this could not be patched using the 2020 drone DEM as it was beyond the 2020 drone DEM extent. The discontinuities have been left in the hydraulic model build, as they tend to be mostly in parallel with the flow direction or downstream of the Ō2NL area of hydraulic interest or influence. The Ō2NL merged dataset also contains a step discontinuity (varying size) along the boundary between the HRC and GWRC terrain datasets. These are discussed in more detail in Appendix F.

5.3 Surface Friction

For all models, Manning's roughness coefficient values ('n') were applied based on the following key sources:

- Land Cover Database version 5.0 (LCDBv5) polygons, as at January 2020.
- LINZ – NZ -primary-road-parcels, as at November 2020.

The Land Cover Database contained 21 different classification within the model extent. Land cover names were grouped into four roughness categories, over which the road parcels was then applied. The final cover categories and the applied Manning's n coefficient are summarized in Table 5-2 below.

Table 5-2: Manning's n roughness coefficients

CATEGORY	Manning's 'n' Value	FINAL SOURCE
HIGH VEGETATION	0.08	Wellington Specification, Table 4. - Vegetation: forest
URBAN	0.1	Wellington Specification, Table 4. Residential Properties: small fenced backyard.
OPEN SPACE	0.048	Guidelines for Stormwater using MIKE FLOOD, Tauranga City Council, 2017. Table 4.1. - Open vegetation
WATER	0.045	Wellington Specification, Table 4. - River
ROAD	0.014	Guidelines for Stormwater using MIKE FLOOD, Tauranga City Council, 2017. Table 4.1. - Road

A manual refinement process was made to update these classifications in some locations using the DEM and recent aerial photography. This included edits to avoid high vegetation obstructions along streams alignment (particularly on larger streams). Also, some new high vegetation patches and urban area corrections were applied. Appendix E shows an overview of the final roughness used in the modelling.

5.4 Existing Bridges and Culverts

Existing bridges and culverts were analysed using the datasets referenced in Section 2.2, notably:

- KiwiRail bridges and culverts.
- State Highway bridges and drainage assets (culverts included).
- Horowhenua District Council (HDC) bridges and culverts.
- Kapiti Coast District Council (KCDC) stormwater points and pipes (culverts included).
- Inspection of aerial photography, LiDAR DEMs and Street View (by Google).
- The locations of bridges and relative to the proposed Project corridor and watercourses is illustrated in Figure 5-7, Figure 5-8 and Figure 5-9.

Most of the existing significant bridges are relatively far (in terms of hydraulic influence) from the Project area, and many of the datasets did not include bridge soffit heights. Therefore, most bridges were modelled in the 2D domain, without their bridge deck present. This could underestimate local water levels when they are above soffit height, but these bridges are far enough from the Ō2NL Project to not have a significant impact on water levels within the Project corridor.

A small 1D/2D bridge structure was added for the existing South Manakau Rd Bridge as it sits close to the proposed project and thus required more accurate modelling of the bridge deck. This bridge was built by drawing a cross section in Hec RAS, used to set the locations of the linked upstream and downstream cross sections. The bridge cross sections were given elevations based on the underlying terrain. A bridge deck width of 10m and thickness of 0.2m was assumed based on design drawings and confirmed by site photographs. A weir coefficient for overtopping was assumed to be the default 0.92.

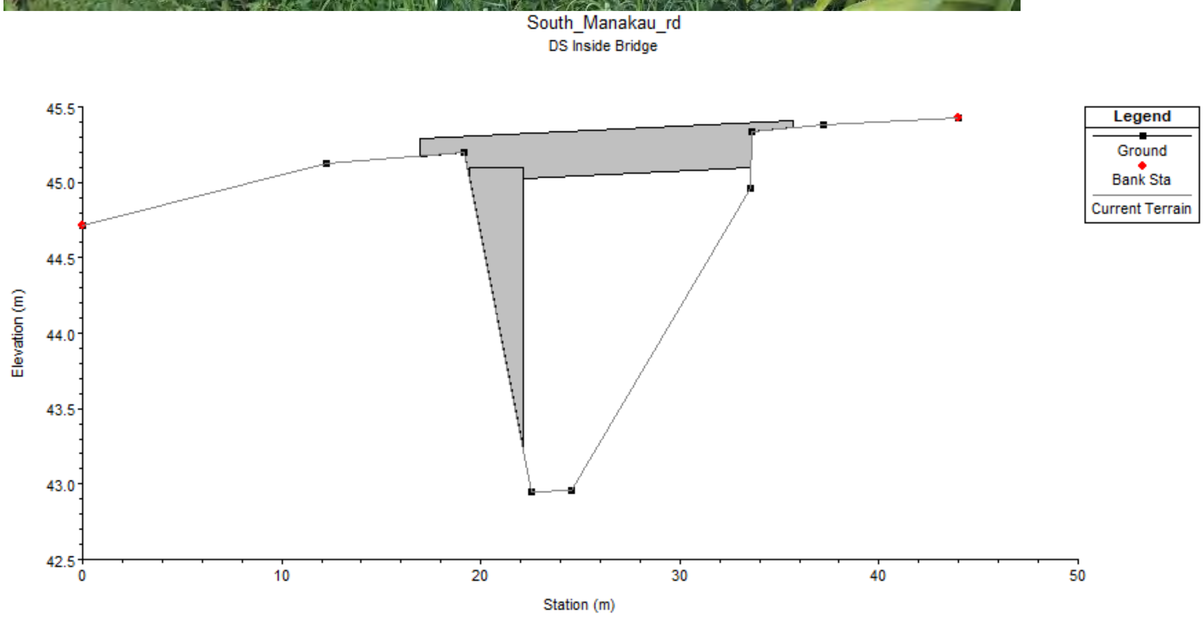


Figure 5-6: Cross section Manakau Stream bridge at South Manakau Road

Culverts that are considered hydraulically significant near (upstream or downstream) to the corridor were built into the model. To include these in the model, internal boundary connections were created in HEC-RAS to allow water to pass from the 2D domain into the 1D feature and back into 2D. The structure embankment is enforced as a 2D break-line to prevent premature overtopping of the structure, as discussed in section 5.2.2. Internal boundary connections were used in the model as needed along existing roads to prevent unrealistic flow through or over the road. Each connection can contain one or more culverts and these were named according the indicative highway crossing number (as an internal reference system), location downstream (DS) or upstream (US) relative to the Ō2NL corridor, and the asset type (railway, highway, or road).

Since the urban built-up area of Levin is significant distance and elevation downstream of the proposed Ō2NL corridor, it was not required to model the urban drainage pipes with Levin.

Culverts with pipe diameter larger or equal to 450 mm and those that affect the flooding near the indicative corridor were applied in the model. A summary of the existing structures represented in the models is included in Table 5-3. This table highlights in red any assumed values that were inferred or estimated (e.g. from LiDAR and assumed cover). It is relevant to mention that there was no level information from all the structure's information sources. Therefore, invert levels on culverts were set considering the lowest nearby 2D surface elevation plus 0.01m. The assumptions used to estimate missing data are summarized in Table 5-4.

Table 5-3: Existing culvert connections

Culvert Name	No Culverts	Total Barrels	Source	Diam (m)	Shape	Length (m)	Invert levels (us/ds, m)
00_DS_HWY	1	1	Highway	2.429	Pipe Arch	17.8	23.9/23.31
00_DS_RD1	2	2	KCDC	1.5	Circular	15.0	17.63/17.63
00_DS_RLW	1	1	KiwiRail	1.2	Circular	14.8	17.28/17.28
00_US_RD1	2	2	ASSUMED	0.6	Circular	15.0	29.29/29.29
00_US_RD2	1	1	KCDC	0.6	Circular	19.5	41.43/41.1
00_US_RD3	1	1	KCDC	0.75	Circular	10.1	51.48/51.3
01_US_RD1	1	1	ASSUMED	0.6	Circular	15.0	22.45/22.41
01-04_DS_HWY	1	1	Highway	1.2	Circular	30.0	15.22/15.95
01-04_DS_RD1	1	1	ASSUMED	1.2	Circular	15.0	15/14.26
01-04_DS_RD2	2	2	ASSUMED	0.6	Circular	15.0	12.76/12.63
01-04_DS_RLW	1	1	KCDC	0.6	Circular	14.0	12.54/10.32
04_DS_RD1	1	1	ASSUMED	0.6	Circular	15.0	23.8/22.26
05-06_DS_HWY	2	2	Highway	1.2	Circular	30.0	16.34/15.7
05-11_DS_RD1	1	1	KCDC	1.2	Circular	21.1	10.14/9.7
05-11_DS_RLW	1	1	KiwiRail	0.91x1.2	Box	43.4	11.98/11.98
11_DS_HWY	1	1	Highway	1.2	Circular	72.3	35.3/33.66
11_DS_RLW	3	3	KiwiRail	0.6	Circular	32.0	37.56/38.7
14-15_DS_HWY	*0*	0	Highway		BRIDGE		
14-15_DS_RD1	*0*	0	HDC		BRIDGE		
14-15_DS_RLW	*0*	0	KiwiRail		BRIDGE		
15_US_RD1	*0*	0	HDC		BRIDGE		
17_DS_HWY	1	1	Highway	0.6	Circular	69.4	34.24/34.24
17_DS_RD1	*0*	0	HDC		BRIDGE		
17_DS_RLW	1	1	KiwiRail	0.6	Circular	28.6	35.83/35.83
17_US_RD1	1	1	HDC	0.6	Circular	15.0	45.78/44.37
17_US_RD2	1	1	HDC	0.6	Circular	14.0	45.78/45.44
18_DS_HWY	1	1	Highway	0.6	Circular	22.3	25.86/25.86
18_DS_HWY2	1	1	Highway	0.45	Circular	22.5	28.9/30.11
18_DS_RLW	1	1	KiwiRail	1.2 x 1.47	Pipe Arch	42.6	34.13/34.84
18_US_HWY	2	2	HDC	1.05	Circular	17.2	42.93/42.57
18_US_RD1	1	1	ASSUMED	0.6	Circular	15.0	49.87/49.87
18_US_RD2	1	1	HDC	0.6	Circular	15.0	52.92/52.92
19_DS_RD1	1	1	HDC	0.6	Circular	15.0	43.04/42.98
19-25_DS_HWY	1	2	Highway	1.05	Circular	18.1	27.38/27.38
19-25_DS_RLW	1	1	KiwiRail	1.2	Pipe Arch	21.5	27.36/27.36
27_DS_HWY	*0*	0	Highway		BRIDGE		
27_DS_RLW	*0*	0	KiwiRail		BRIDGE		
27_US_RD1	1	1	HDC	0.6	Circular	15.0	66.2/66.93
28_DS_HWY	1	1	Highway	0.38	Circular	24.0	44.71/44.19
28_DS_RLW	1	1	KiwiRail	0.45	Circular	10.0	39.72/39.32
29-30_DS_HWY	1	1	Highway	1.05	Circular	20.9	37.9/38.9
29-30_DS_RLW	1	1	KiwiRail	1.5	Pipe Arch	10.0	24.34/23.93
31-32_DS_HWY	1	2	Highway	0.6	Box		25.71/25.71
31-32_DS_RD1	*0*	0	HDC		BRIDGE		
31-32_DS_RD2	*0*	0	HDC		BRIDGE		
31-32_DS_RLW	*0*	0	KiwiRail		BRIDGE		
32_DS_RD1	1	1	ASSUMED	0.6	Circular	15.0	33.15/33.15
33_DS_HWY	*0*	0	Highway		BRIDGE		
33_DS_RLW	*0*	0	KiwiRail		BRIDGE		
33_US_RD1	*0*	0	HDC		BRIDGE		
34_US_RD1	*1*	2	HDC	0.3	Circular	9.0	48.87/48.67
34_US_RD2	*1*	1	HDC	0.6	Circular	20.0	49.29/49.12
34_US_RD3	*1*	1	HDC	0.6	Circular	20.0	46.42/46.1
34_US_RD4	*1*	1	ASSUMED	3.42x2	Box	12.0	45.02/44.94
36_DS_RD1	*1*	1	HDC	0.75	Circular	11.9	41.5/41.29
36_DS_RD2	*1*	2	HDC	0.45	Circular	10.0	35.27/35.02
36_DS_RD3	*1*	1	HDC	0.45	Circular	10.0	33.13/33.13
36_US_RD1	*1*	*1*	HDC	0.6	Circular	10.0	63.83/63.6
37_DS_HWY	*2*	2	HDC	0.525	Circular	31.0	48.75/48.1
39_DS_HWY	*1*	2	Highway	0.375	Circular	17.1	19.27/1
39_US_HWY	*1*	1	Highway	0.85	Circular	31.0	30.81/30.81
41-42_DS_RD1	*3*	3	HDC	0.6	Circular	15.0	11.36/10.66

Culvert Name	No Culverts	Total Barrels	Source	Diam (m)	Shape	Length (m)	Invert levels (us/ds, m)
42_DS_RD1	*1*	1	ASSUMED	0.6	Circular	15.0	31.34/31.75
42_DS_RD2	*2*	2	HDC	0.6	Circular	15.0	29.57/27.4
42_DS_RD3	*1*	1	ASSUMED	0.6	Circular	15.0	30.84/30.24
42_DS_RD4	*1*	1	ASSUMED	0.6	Circular	15.0	34.46/33.26
42_DS_RD5	*1*	1	HDC	0.6	Circular	15.0	23.6/21.73
42_DS_RLW	*4*	4	KiwiRail	0.45	Circular	10.0	40.8/39.1
42_DS_RLW2	*1*	1	KiwiRail	0.45	Circular	10.0	38.83/39.41
43_DS_RD2	*0*	0	HDC		BRIDGE		
48_DS_RD1	*1*	1	HDC	0.6	Circular	15.0	62.4/62.3
48_DS_RD2	*1*	1	HDC	0.6	Circular	10.0	60.3/61.2

Table 5-4: Assumptions used to infill missing structure data

Assumptions for Weirs	
Width	Weir on railway: 4m Weir on local road (no highway): 6m Weir on highway: 10m
Coefficient (Cd)	0.92 (see HEC-RAS Hydraulic Reference for SI units also ref[4])
Weir Crest Shape	Broad crest
Overflow Computational Method	Normal 2D Equation
Assumption for Culverts	
Type	Circular
Diameter	600mm, or nearest known existing culvert diameter if larger
Invert Levels	The lowest upstream level from DEM
Chart /Type	1-3. Concrete pipe culvert. No headwall.
In / Out Loss coefficient	0.5 and 0.1 respectively
Length	If there is not information from source the following lengths were applied: On railway: 10m On highway: 20m On local road (no highway): 15m
Roughness	• Concrete 0.013 • Corrugated arch pipe 0.024

[4] https://engineerpaige.com/lateral-structure-weir-coefficients-in-hec-ras/

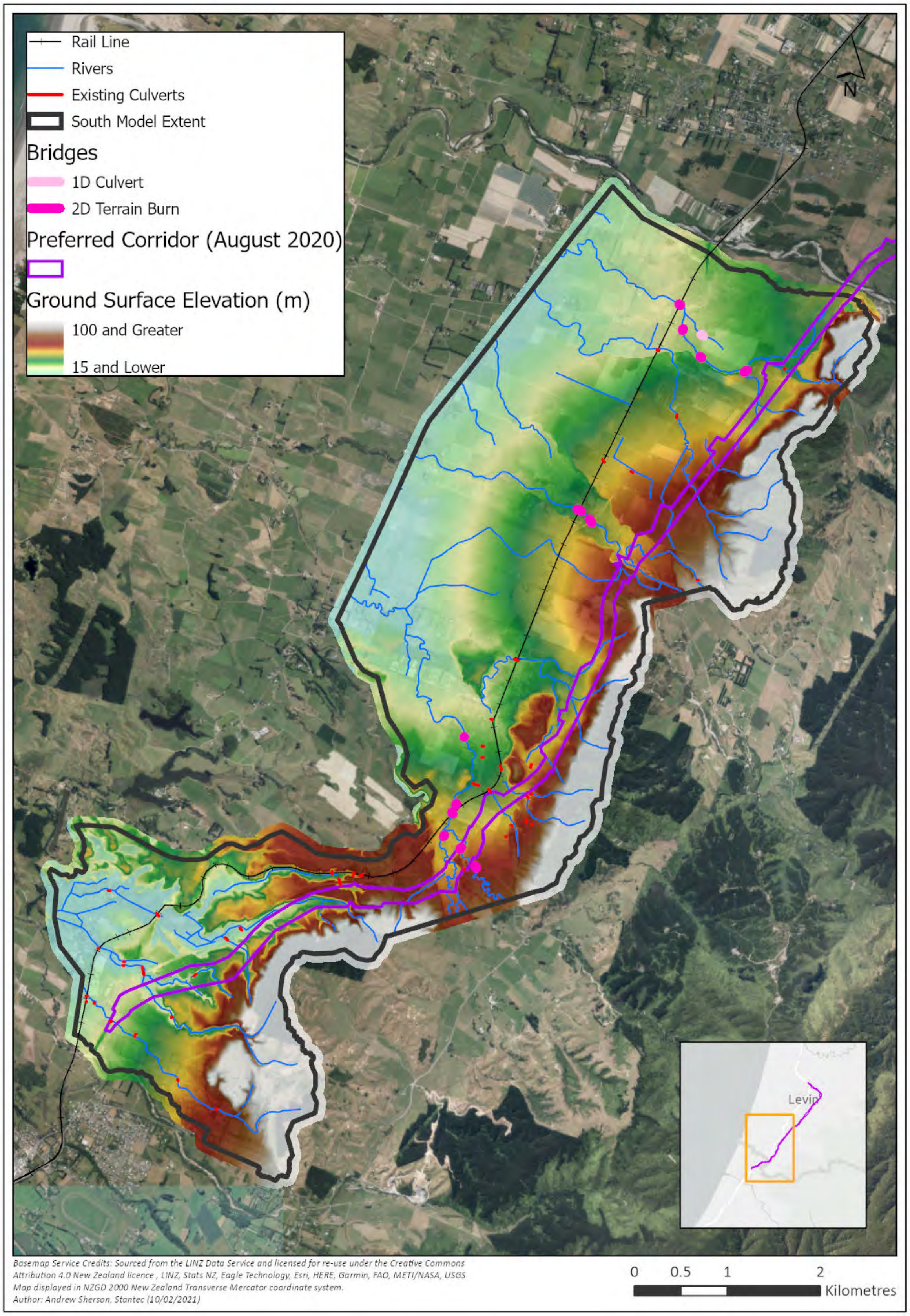


Figure 5-7: Map of modelled existing bridges and culverts in South model

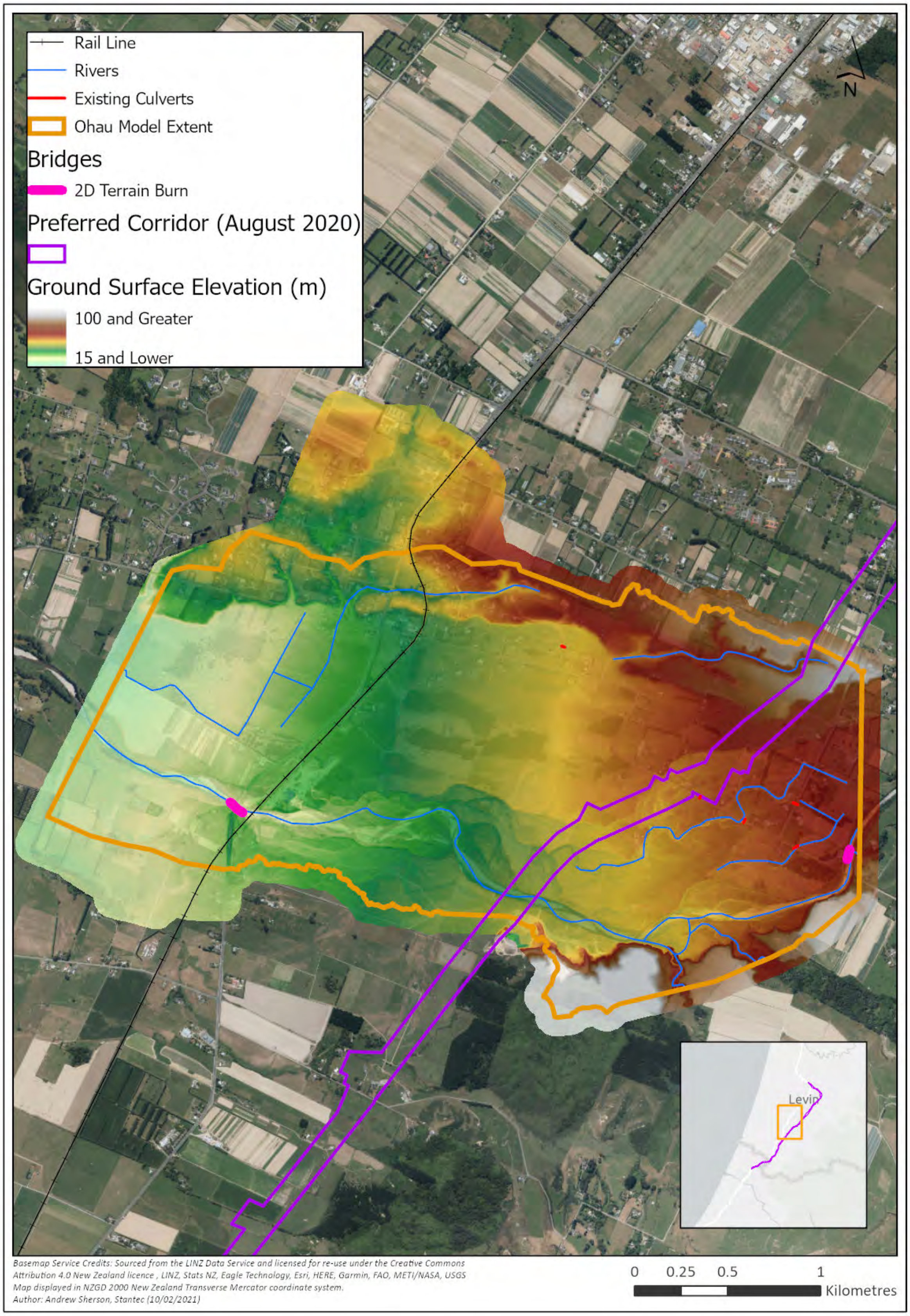


Figure 5-8: Map of modelled existing bridges and culverts in Ōhau model

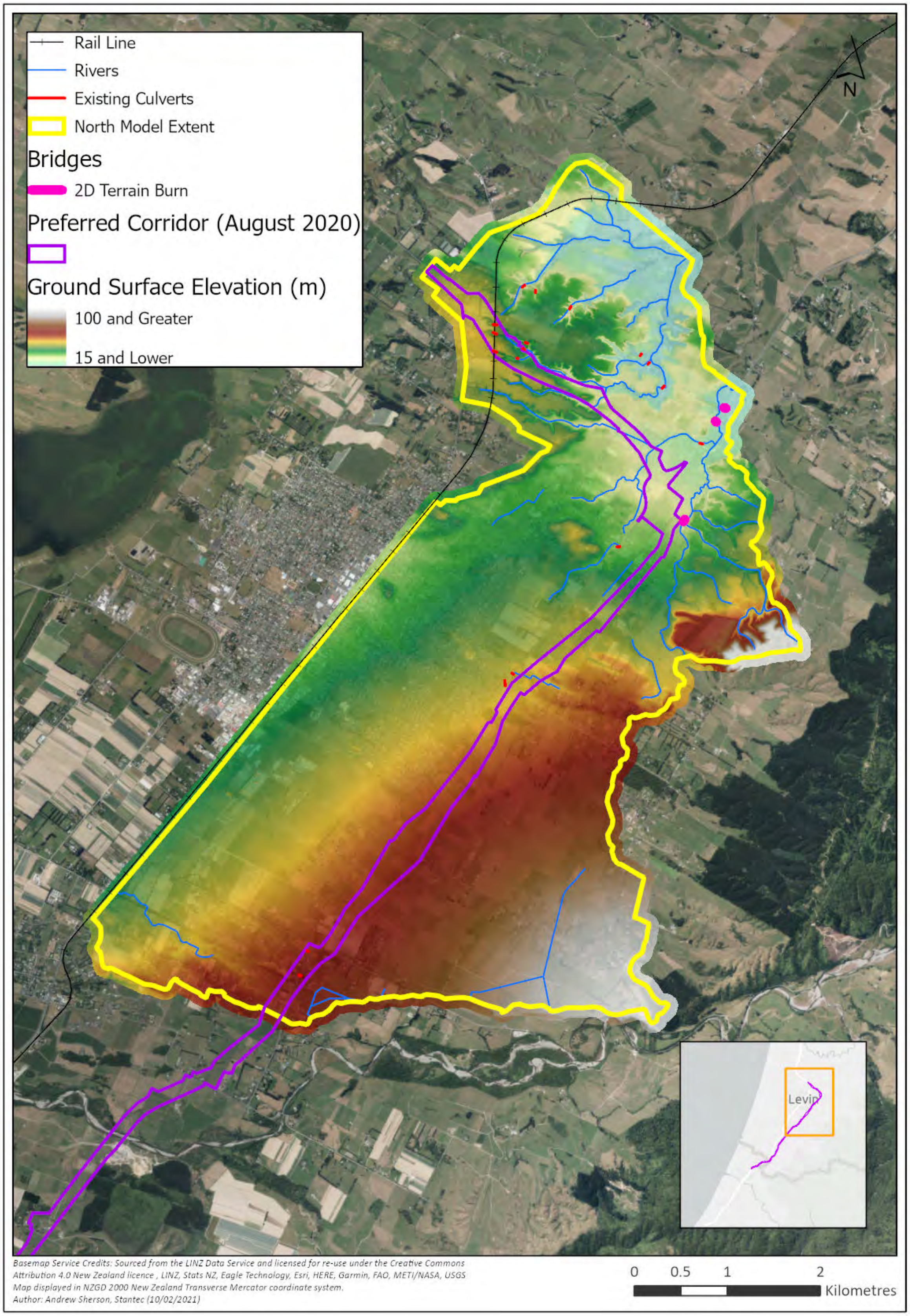


Figure 5-9: Map of modelled existing bridges and culverts in North model

5.5 Boundary Conditions and Scenarios

Downstream boundaries of the 2D models were set using the normal slope of the streams at these locations, derived from the DEM. Normal slopes were chosen for the downstream boundary conditions as no existing water levels at these points was known for different AEP events. By using the normal slope of the stream, the model calculates a water level at the boundary condition based on flow area including the floodplain, Manning's 'n' and the normal slope. A summary of the upstream and downstream boundary conditions is provided in Table 5-5.

Table 5-5: Model boundary conditions summary

Model	Boundary Condition Name	Boundary Type	Normal Depth Slope
South Model	US_14 (South_1)	Flow Hydrograph	
	US_15 (South_2)	Flow Hydrograph	
	US_27 (South_3)	Flow Hydrograph	
	US_27.1	Flow Hydrograph	
	US_32 (South_4)	Flow Hydrograph	
	South_Mesh	Precipitation	
	DS_1-11	Normal Depth	0.0034
	DS_13-22	Normal Depth	0.0023
	DS_27	Normal Depth	0.0053
	DS_32	Normal Depth	0.0026
	DS_44	Normal Depth	0.0092
	DS_31	Normal Depth	0.0035
Ōhau Model	US_33 (Ōhau_1)	Flow Hydrograph	
	US_33e	Flow Hydrograph	
	Ōhau_East_Mesh	Precipitation	
	DS_33	Normal Depth	0.0028
North Model	US_ North (North_1)	Flow Hydrograph	
	North_Mesh	Precipitation	
	DS_41-42	Normal Depth	0.0032
	DS_39-40	Normal Depth	0.0029
	DS_37-38	Normal Depth	0.0082

Upstream boundary conditions were defined using point inflow hydrographs derived in Section 4 plus distributed direct rainfall for the smaller catchments near and downstream of the Ō2NL corridor as discussed in Section 5.6 below.

5.6 Direct Rainfall

The rainfall totals (prior to applying losses) and sample temporal profiles for different storm durations were prepared for the 2D direct rainfall modelling zones, using the methods outlined in Sections 4.2.2 to 4.2.8. Direct rainfall was the preferred approach to produce a map of flooding 'everywhere' (as opposed to just within known watercourses at known crossing locations). This approach allows the effects of downstream tributary backwater effects and is independent of final road alignment.

Based on the range of indicative 2D catchment sizes draining towards and near the corridor, initial testing commenced with 20 minute and 1h storm durations with their respective total rainfall amounts and temporal profiles applied. The Initial and Constant loss values were adopted initially from the HEC-HMS models described in 4.4, namely 7mm and 5mm/hr respectively, and applied to the respective rainfall profiles. Despite the higher total rainfall intensities, the 20 minute storm was smaller than a 1h storm once run through the distributed hydraulic model with appropriate initial losses applied. Further testing therefore focussed on a 1h and 4h storm.

A subset of catchment outflows from the HEC-RAS 2D model were compared against HEC-HMS and Rational Method peak flows as the sample locations. As anticipated, the initial set of losses via the HEC-RAS produced unrealistically high peaks compared to the HEC-HMS model. This is expected to be mainly associated with the HEC-RAS model translating flow too quickly to the outlet of the catchment, even if high roughness values are applied, because the HEC-RAS model does not capture many local depressions and initial or continuing storage (in which ongoing infiltration can occur after rain has ceased falling, reducing surface flow volume at the catchment 'outlet'). By contrast, the HEC-HMS represents these processes differently via selection of unit hydrographs, time of concentration and storage coefficients. It was found that the HEC-RAS model was calculating flows (and hence associated levels) far greater than the calibrated flows from the HEC-HMS models.

The Initial and Constant loss parameters for the HEC-RAS applied rainfall were therefore increased from 7mm and 5mm/hr to 16mm and 7mm/h respectively. The resulting effective rainfall depths are presented in Table 5-6. The reduced peak sub-catchment outflows from the HEC-RAS model compared graphically to independent peak flow estimates (HEC-HMS, Rational Method, Flood Frequency Analysis) in Figure 5-10. The specific yield (peak flow divided by catchment area raised to the power of 0.9) is plotted against catchment area. The spread in the cloud of 2D data points is attributable mainly to variations in catchment steepness and storage features (ponds/depressions). It is worth reiterating that the FFA for the Koputaroa at Tavistock Rd is considered unreliable. In general, there appears to be a good correlation between the trends in the data clouds from the respective methods, which supports the newly adopted loss values (previous 2D clouds were unrealistically high). There is also a good correlation with the PP2Ō data point at Greenwood Stream (PP2Ō Culvert 1, based on a peak of 9.46m^3/s extracted from Waitohu model inflow boundary). Additional sensibility checks on the losses and percentage runoff are discussed below.

Table 5-6: Direct rainfall adopted values and losses for 1h storm

	Total rainfall	Effective rainfall with I/C 7/5		Effective rainfall with I/C 16/7 (adopted)	
Event	**Depth (mm)**	**Depth (mm)**	**%**	**Depth (mm)**	**%**
1:10 AEP current climate	26.1	16.0	61.1%	6.9	26.6%
1:100 AEP RCP6 2130	56.1	44.4	79.0%	33.7	60.0%
1:1500 AEP RCP8.5 2130	84.4	72.5	85.8%	61.5	72.9%

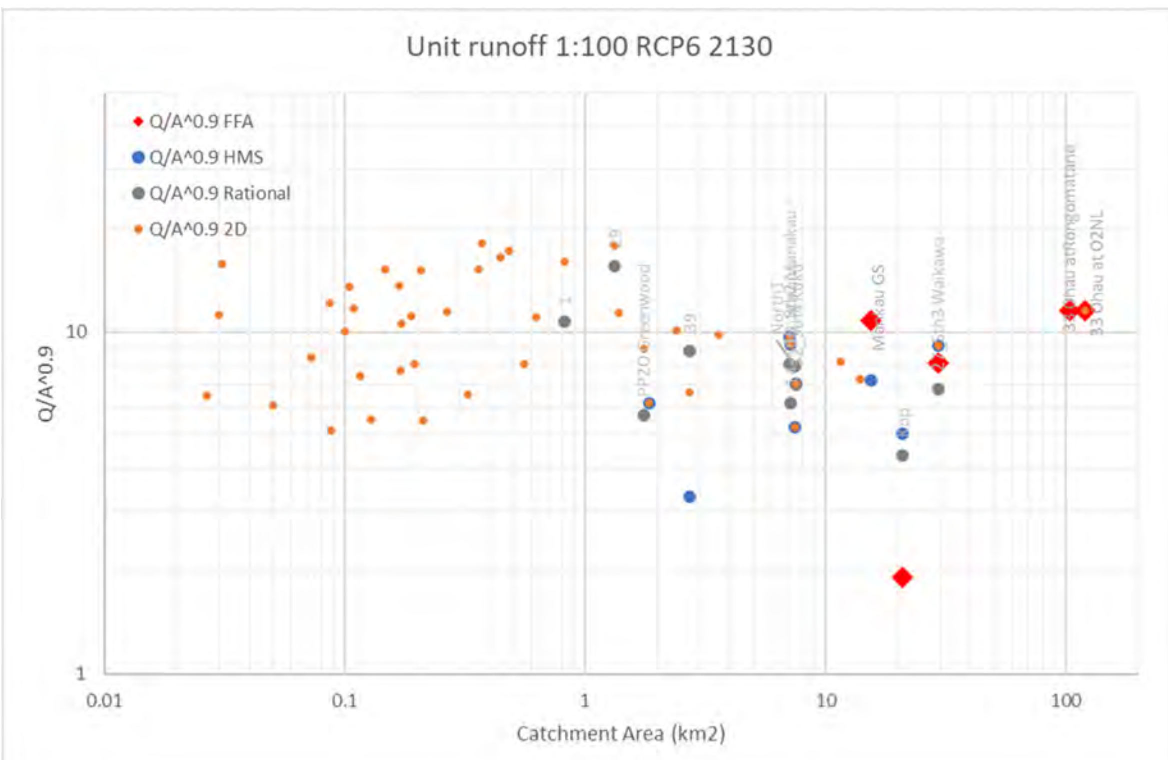


Figure 5-10: Comparison of unit runoff per km^2 (1:100 AEP RCP6 2130)

The range in % runoff presented in Table 5-6 for the updated Initial and Constant losses appears quite large (i.e. the difference between 1:10 current climate and 1:1500 with climate change). It is desirable to confirm that the resulting range of % runoff values was not being unduly influenced by the rapid response in the HEC-RAS model (compared to calibrated HEC-HMS). Therefore, the infiltration rates were compared qualitatively against design reference curves and tables for rural catchments in Figure 5-11 below (reproduced from NZ Geological Society paper “Soil Infiltration Rates” by John Hawley, June 2016). Table 5-6 is for a 1h storm, and the depths are therefore indicative of the average rainfall intensity over the 1h period. Taking the smallest total rainfall value of 26mm (1:10 AEP current climate) and the largest 84mm (1:1500 AEP RCP8.5 2130), the % runoff in the table appear to correlate reasonably to either a medium soil with forest cover or sandy soil with open crop, which provides additional confidence that the values are not unrealistic. Additional qualitative checks were also made against ultimate infiltration rates for application in the Horton method, reproduced in Figure 5-12 (from Christchurch City Council Waterways Wetlands and Drainage Guide: Part B - Chapter 21 – Updated June 2020).

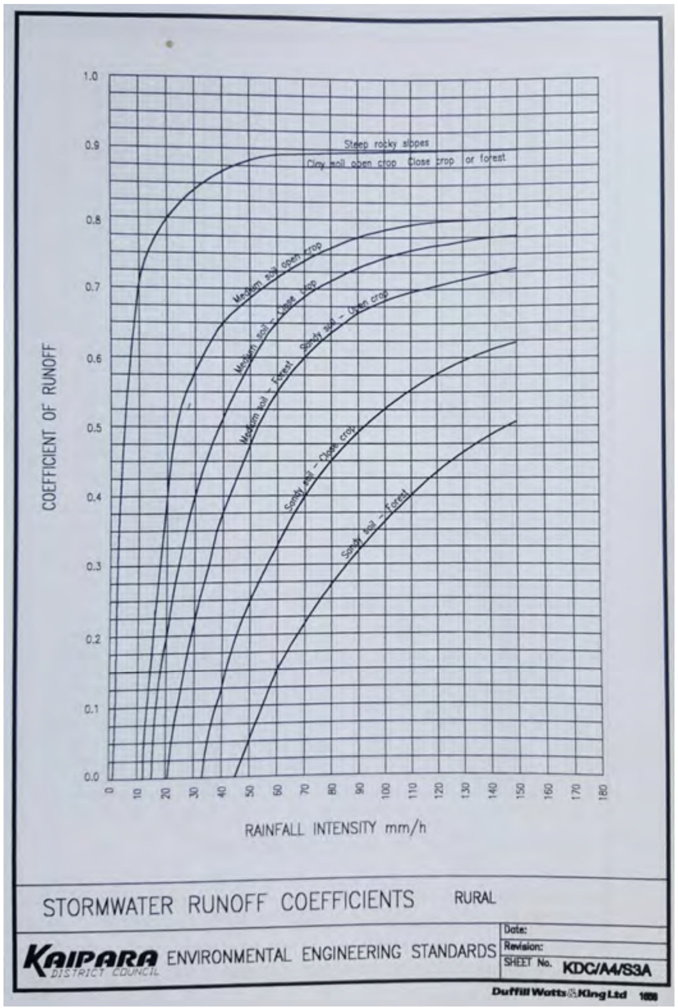


Figure 5-11: Stormwater runoff coefficients for rural catchments (Kaipara District Council)

Table 21-11: Soil type ultimate infiltration rates.

USDA Soil Texture Classification	Ultimate infiltration rate (mm/hr)	Infiltration Type
Sand	230	Free
Loamy Sand	60	
Sandy Loam	22	
Loam	13	Moderate
Silt Loam	6.8	
Sandy Clay Loam	3.0	Poor
Clay Loam	2.0	
Silty Clay Loam	2.0	
Sandy Clay	1.2	
Silty Clay	1.0	
Clay	0.6	

Figure 5-12: Ultimate soil infiltration rates (Christchurch City Council)

With the selected losses of 16mm and 7mm/h, the HEC-RAS model confirmed that the 1h storm is larger than a 20-minute storm for all sites of interest near the proposed Ō2NL corridor (due to the initial losses having a more profound effect of lowering the 20-minute total effective rainfall). Therefore, in subsequent simulations, the 20-minute storm is not used. The maximum of the 1h and 4h storm durations is used.

The timing of the 1h storm was lagged by 90 minutes so that the peak rainfall intensity for the 1h direct rainfall storm coincided with the peak rainfall intensity applied to the HEC-HMS models to derive the 4h critical duration storm point inflow hydrographs. This hybrid storm approach with coincident critical spatial intensities is more accurate and representative of reality than a nested temporal storm profile applied to the whole system.

Visual comparisons were made of the HEC-RAS model results against Regional Council flood mapping where available for the larger rivers (Waiauti, Manakau, Waikawa and Ōhau). There was generally good correlation in shape and extent, although the provenance (source, date and modelling approach or flows applied) for the Regional Council flooding layers were not known, and they appeared to include some unrealistic extents relative to ground topography. In Levin, existing HDC urban flood modelling is focussed around representing the impervious areas and pipe network capacity, with shorter duration storms and/or that drain away from the Ō2NL corridor, and therefore were of less relevance for direct comparison. The drainage network and flow paths through Levin do not have an impact on water levels within the proposed project corridor.

In-situ testing of infiltration rates as part of Detail Design could potentially be used to further refine the adopted infiltration / loss values, potentially along with additional rainfall and streamflow data collection and analysis. Consideration could also be given to applying the Horton loss model approach, depending on the outcomes or issues identified during the assessment of effects or factors shown to influence the Detailed Design.

5.7 Run Parameters and Model Stability

The model used an adaptive time step based on courant formula with a maximum value of 2 and a minimum value of 0.5. The equation set used in all models was 'Full Momentum'.

The three models were simulated successfully across all AEP events, with the model balance results summarised in Table 5-7.

Table 5-7: Model balance results (volume accounting in 1,000 m^3)

Model	Plan Name	Ending Vol (1000m^3)	Cum Inflow	Cum Outflow	Error in 1000m^3	% Error
South	P24_0.07%AEP_USv06 Gv09 Tv07	1606	8515	6911	1.768	0.02076
	P22_1%AEP_USv06 Gv09 Tv07	1096	5136	4042	1.331	0.02591
	P21_10%AEP_USv06 Gv09 Tv07	514.2	1873	1360	0.407	0.02173
Ōhau	P39 - 0.07% AEP UsV06 Gv07 Tv03	65.92	11427	11363	1.548	0.01354
	P36 - 1% AEP UsV06 Gv07 Tv03	61.82	7305	7245	1.887	0.02583
	P37 - 10% AEP UsV06 Gv07 Tv03	46.32	3141	3095	0.586	0.01866

Model	Plan Name	Ending Vol (1000m³)	Cum Inflow	Cum Outflow	Error in 1000m³	% Error
North	P32_North_1500yr_Geo v8 TerV6.0	399.4	2746	2346	0.0971	0.00354
	P29_North_100yr_Geo v8 TerV6.0	307.5	1501	1194	0.0404	0.00269
	P30_North_10yr_Geo v8 TerV6.0	182.2	369.6	189	1.53	0.4138

5.8 Baseline Model Results

The various AEP design event scenarios were applied to the models and run for 6 hours of simulation time to allow the peak value to pass the downstream boundary. Maximum results from the entire simulation are then available as flood extent overview maps in Appendix G.

5.9 Limitations and Residual Uncertainties

The modelling outputs are a reasonable representation of peak flow rates and depths, to allow representation of baseline flood risk near the proposed Ō2NL corridor, and to allow an Assessment of Environmental Effects suitable for Resource Consent. The source data and the hydrological and hydraulic modelling processes have followed industry best practice but still naturally contain some uncertainties as normally anticipated. Depending on the effects or issues that arise during assessment of effects, or if greater precision is required during detailed design (for example if seeking to reduce standard design tolerances or freeboard), consideration could be given to reducing some of the residual uncertainties. Potential limitations and uncertainties to consider include:

- Limitations in the availability of accurate spatial catchment average rainfall and flow gauge data on some streams for hydrological calibration purposes, and the hydrological methods to represent both calibration events and design (hypothetical) events at a variety of scales, including losses (evaporation, canopy/interception and infiltration losses). Some of these uncertainties could be reduced through additional data collection and analysis, for historic and new flood events, in addition to field testing of infiltration losses. Consideration could be given to improving catchment average rainfall using orographic or altitude-aware methods and applying a Horton loss-model approach for the direct-rainfall model zones.

- Limitations in the Regional Council ground model (LiDAR data) on which the modelling was primarily based. This was discussed in Section 5.2 and Appendix F, including the methods currently used to mitigate these effects. Additional improvements may be feasible depending on the level of accuracy required, for example by improving the quality of the 2020 drone DEM, and/or adding new LiDAR coverage for the model extent or performing additional ground-control survey within and beyond the 2020 drone DEM extent to allow semi-automated adjustments to be applied to the 2020 drone DEM and the Regional Council LiDAR to reduce the step changes within and between the datasets. The onus will rest on the Detailed Design project stage to confirm suitability of the ground model and any associated hydraulic modelling for their final design and construction purposes.

- Limited information on existing hydraulic assets such as bridges and culverts, which necessitated some parameters to be estimated. Most of these are relatively far from the proposed corridor so the impact on results is expected to be small. Again, the tolerance of design decisions can be used to inform what additional data could be collected. For example, the closest existing bridge to the proposed corridor is the Manakau Stream at Manakau Road, where the proposed bridge is likely to have ample vertical clearance dominated by existing road clearance requirements rather than by flood level requirements. This will be discussed further in the post-scheme modelling report and Assessment of Effects.

- The model is focussed on estimating flood depths in the vicinity of the proposed Ō2NL corridor, and therefore the model DEM and results have been more closely scrutinised where they can influence the corridor. Levels elsewhere in the model domain, as well as modelled velocities and total flow volumes for different storm durations may not be fully representative of all flood scenarios.

- The representation of climate change is based on the science from IPCC 5th assessment global climate model predictions downscaled to New Zealand by NIWA (2018). Some aspects or effects of future climate change remain uncertain. Part of the testing of climate change uncertainty is achieved through the ULS resilience case 1:1500 AEP RCP8.5 2130 which represents a conservative climate case. However, there is still minor uncertainty in other aspects of climate change and knock-on impacts such as on vegetation and sediment mobility and their effects on watercourse morphology.

- The baseline modelling presented in this report, including for future climate assessments, is based on current topography, drainage assets and estimated infiltration rates. These may change in future through natural morphological change (including earthquakes and associated debris loads, in addition to gradual erosion and aggradation processes), plus anthropological changes in land-use and its impacts on infiltration rates. Future growth in impervious areas is expected notably in the proposed Tara-Ika development east of Levin, although the HDC draft stormwater strategy for Tara-Ika indicates a design philosophy to mimic natural green-field runoff rates by storing and infiltrating roof runoff, and taking road runoff to treatment, storage and infiltration areas. The sensitivity of Ō2NL designs to the effectiveness of Tara-Ika designs and exceedance runoff rates will be tested in the post-scheme modelling and/or through the detailed design stage.

6. Conclusion

The data collection, baseline hydrology and hydraulic modelling have been completed to a standard suitable for Assessment of Environmental Effects (baseline) in preparation for Resource Consent and Notice of Requirements Application.

Example baseline floodmaps are prepared and shown in Appendix G.

This baseline report will be read alongside the with-scheme modelling report that explains the hydraulic elements of the scheme and its effects, to underpin the AEE.

Appendices

Appendix A Map Figures Data Source

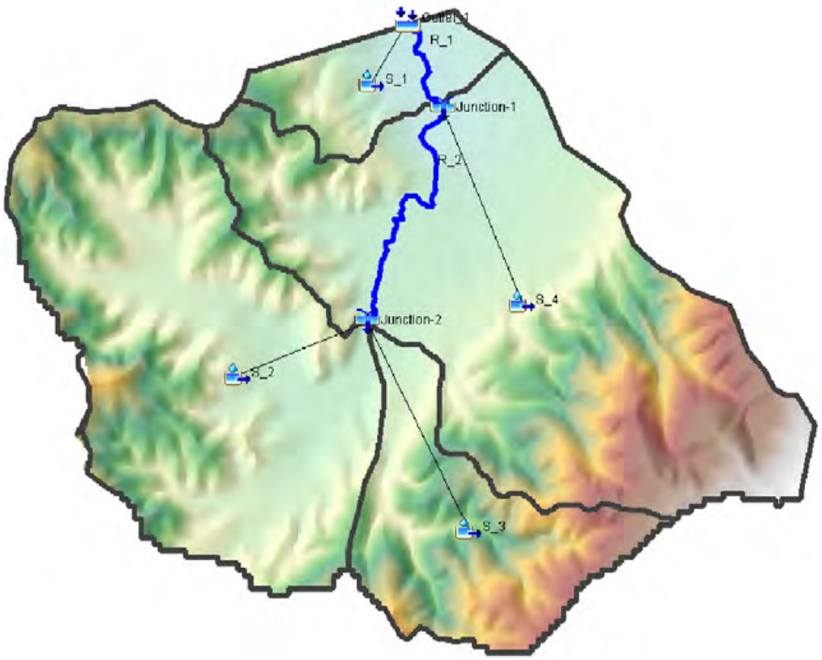
Data Sources: LINZ, GWRC, HRC, KCDC, HDC, KiwiRail and Stantec NZ.

Basemap Service Credits: Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors, LINZ, Eagle Technology, Esri, HERE, Garmin, FAO, NOAA, USGS

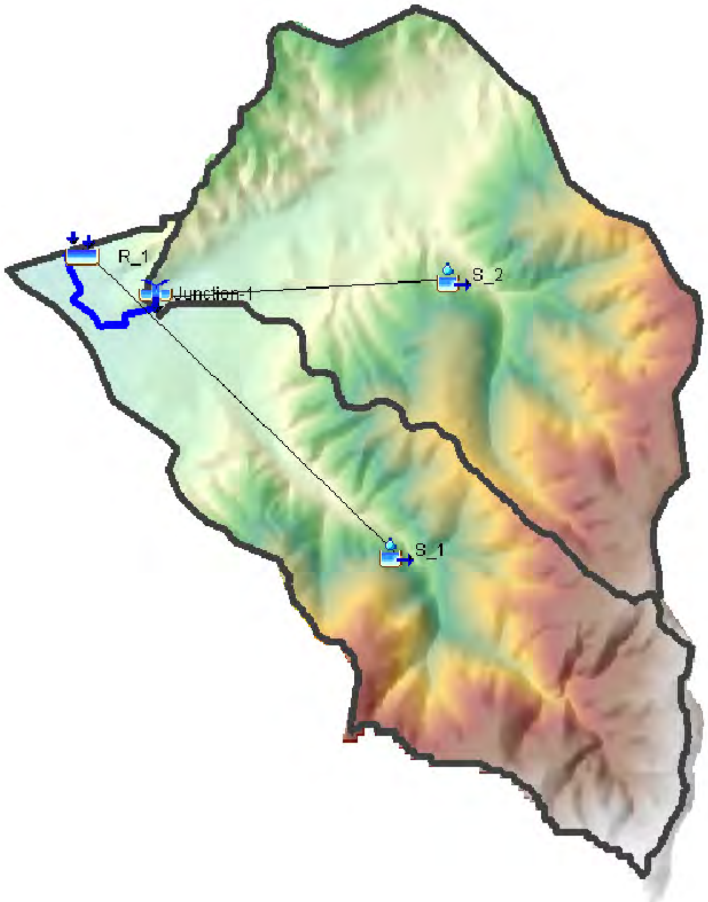
All maps displayed in NZGD 2000 New Zealand Transverse Mercator coordinate system unless otherwise specified.

Appendix B HEC-HMS Model Schematics

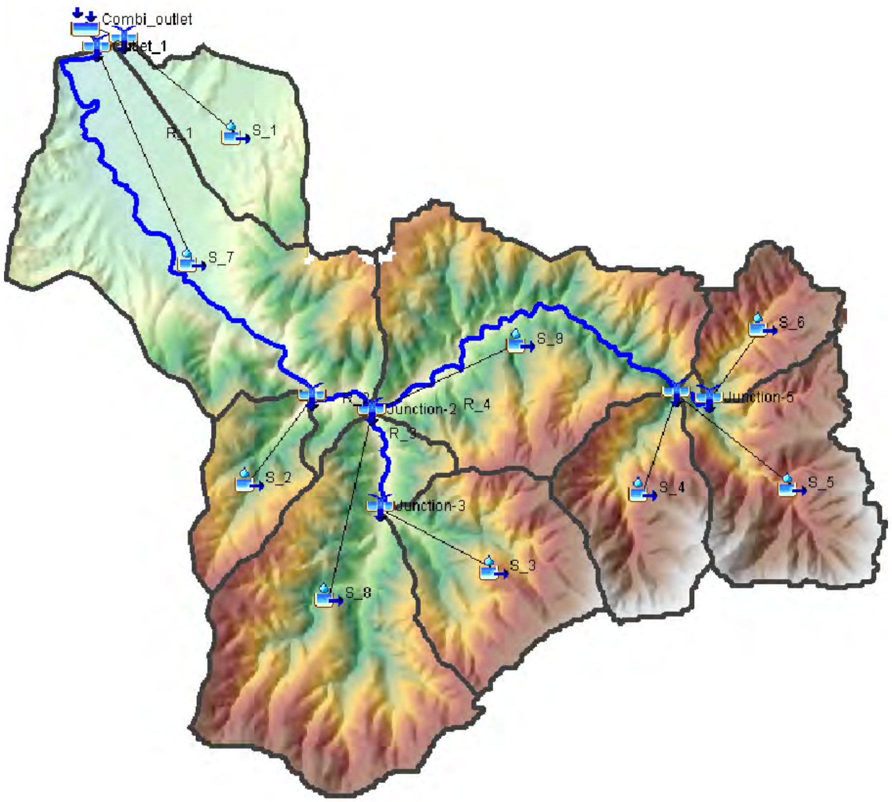
South 1 (Waiauti)



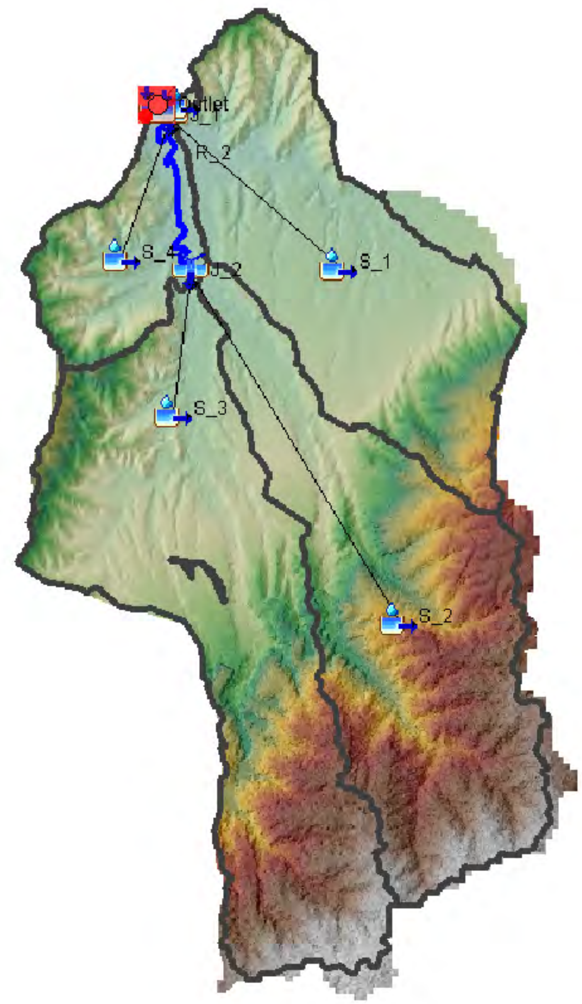
South 2 (Manakau)



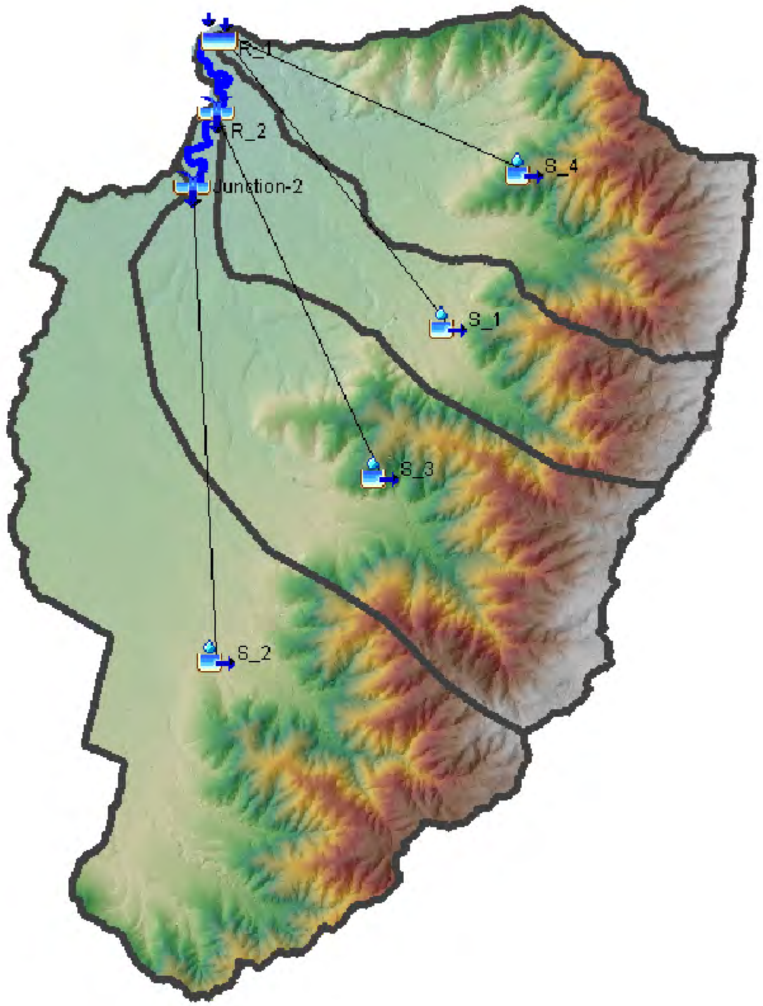
South 3 (Waikawa)



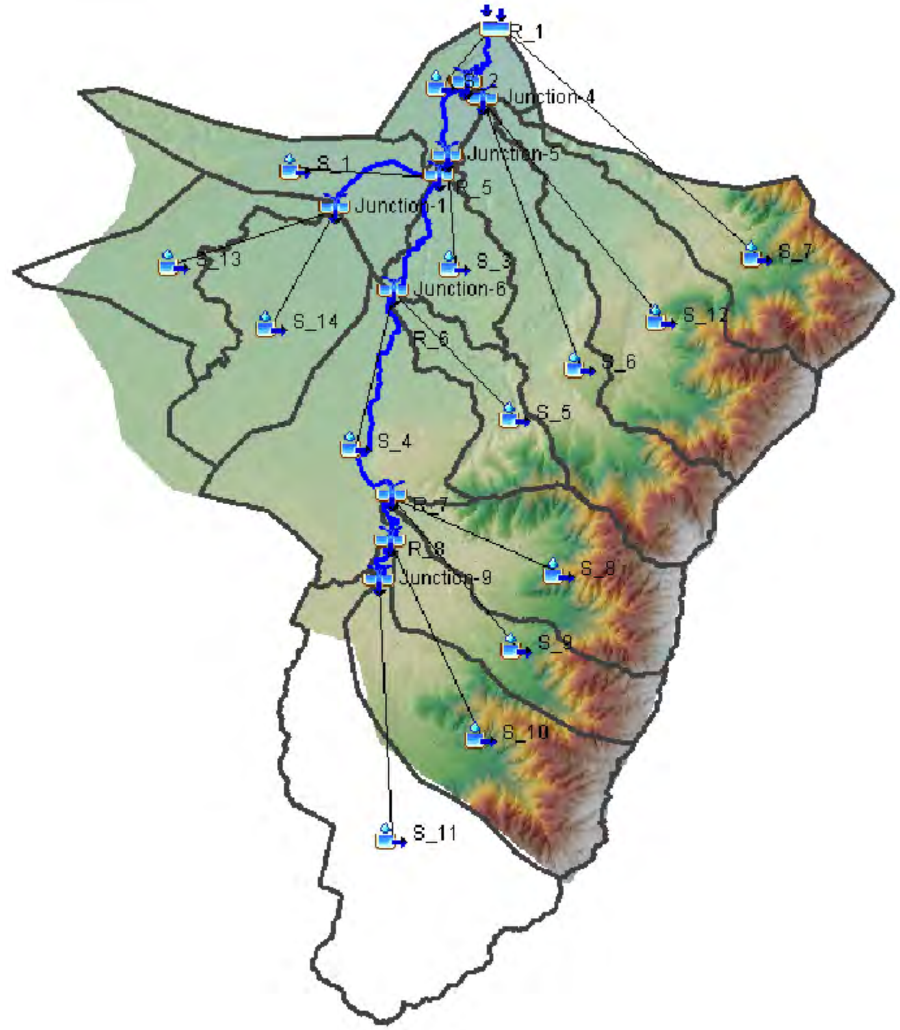
South 4 (Kuku)

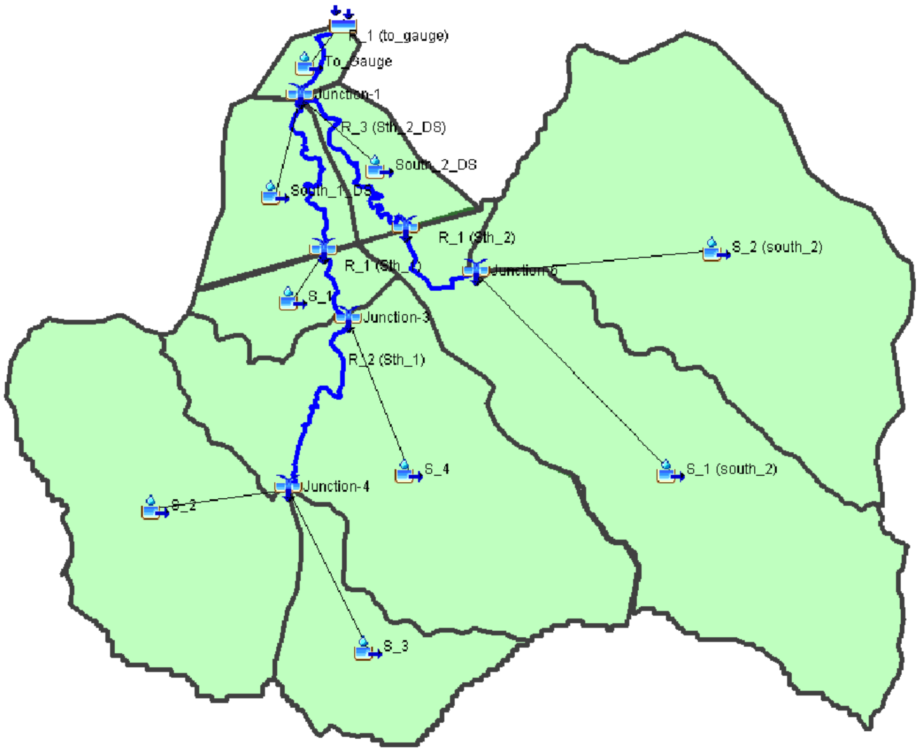


North 1



Koputaroa



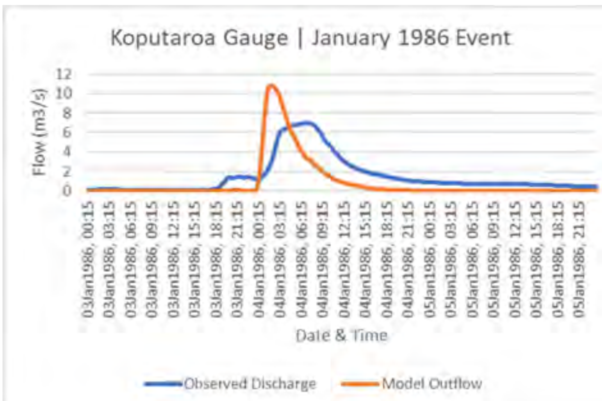
Manakau at SH1 Gauge Catchment	
	

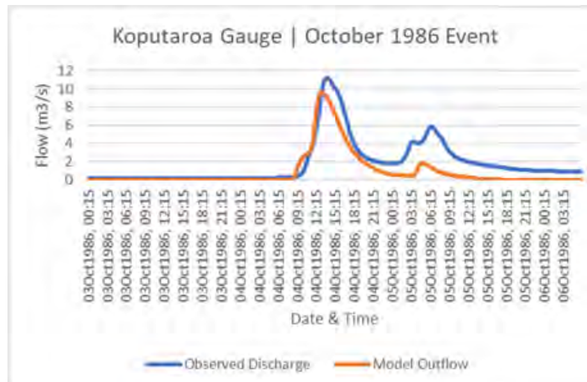
Appendix C HEC-HMS Validation Graphs

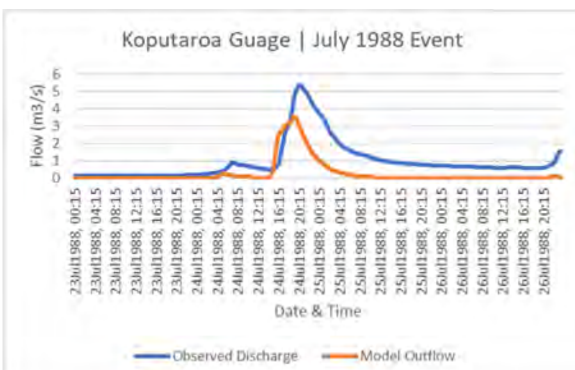
Refer to discussion in Section 4.4.3.

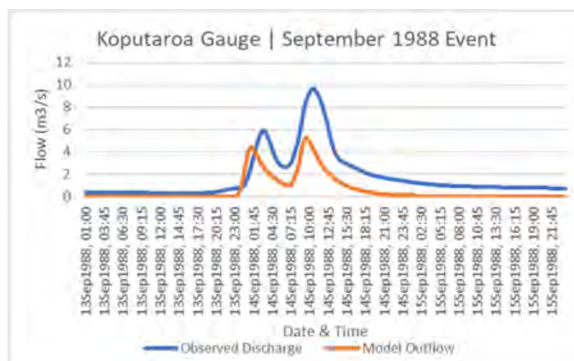
The first table below contains four separate events simulated on the Koputaroa gauge at Tavistock Rd (which had s shorter record from 1974-1989).

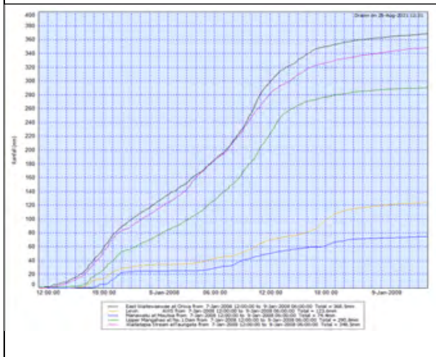
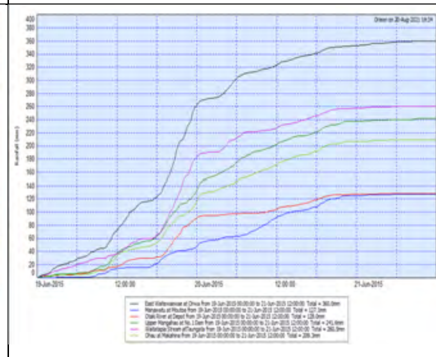
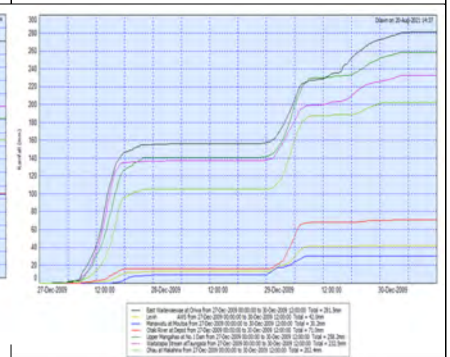
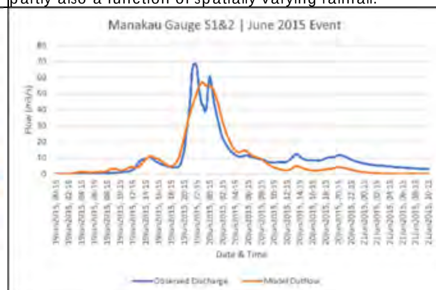
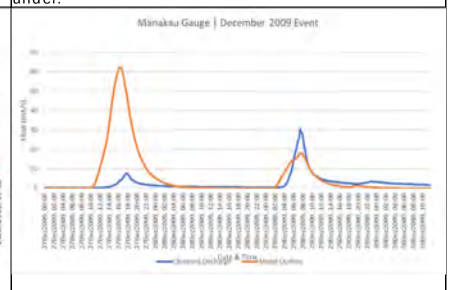
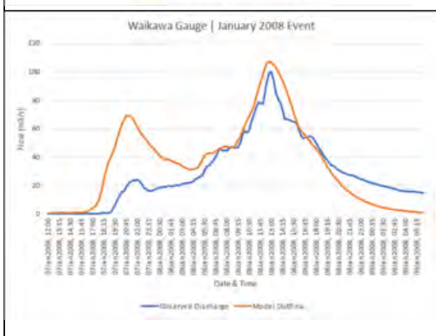
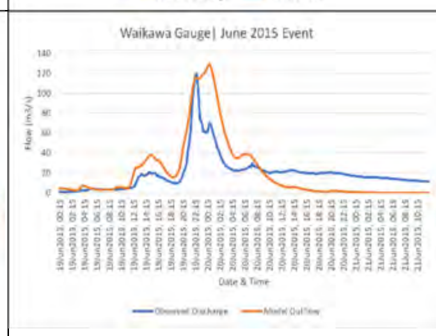
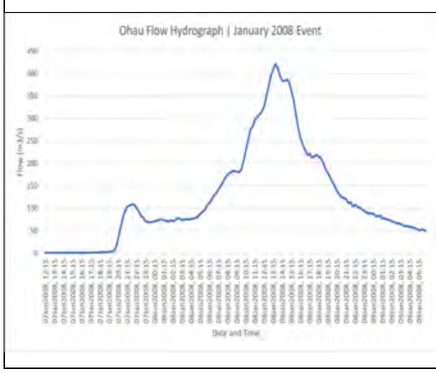
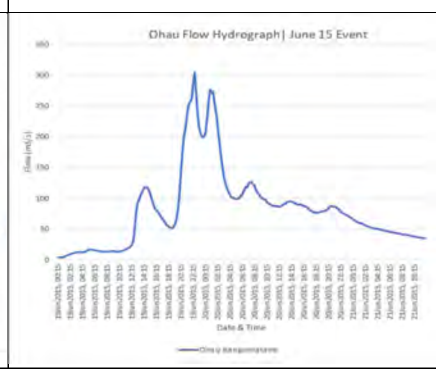
The first table is followed by a more detailed assessment for Manakau and Waikawa hydrological models. A very small layout is used deliberately to highlight patterns across rainfall plus Manakau and Waikawa on the same screen. Zooming in allows more detail to be seen. Ōhau hydrographs also shown to support discussion on hydrograph shape for some events.

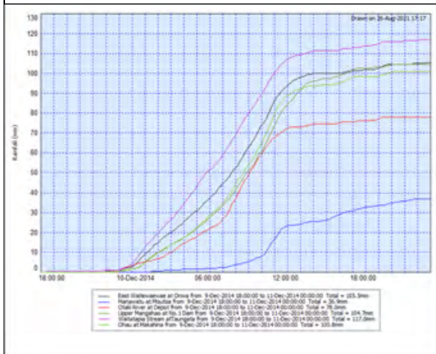
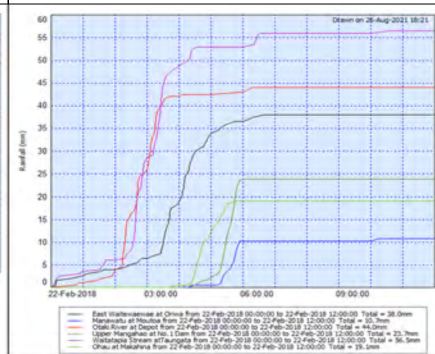
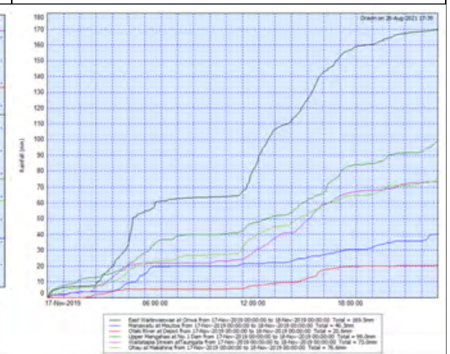
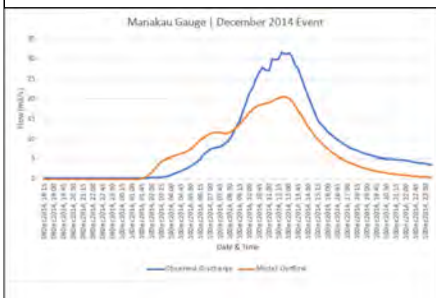
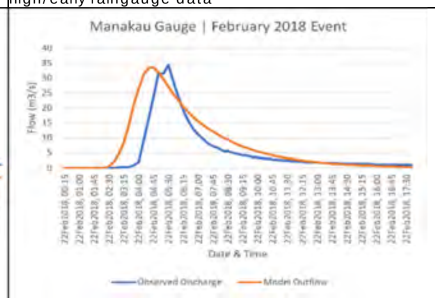
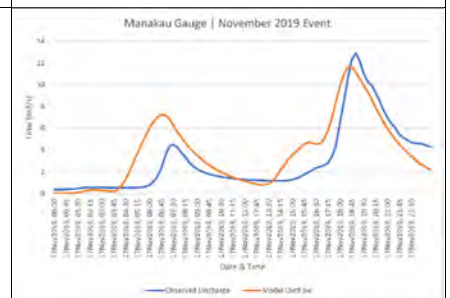
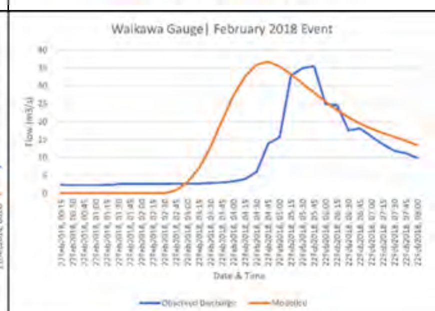
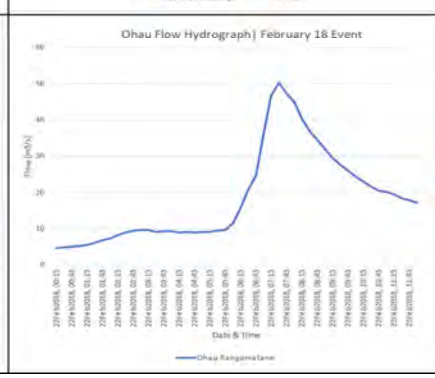
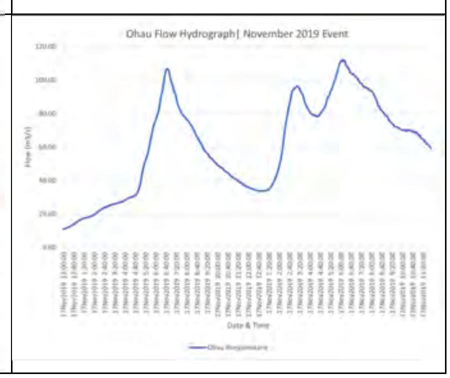




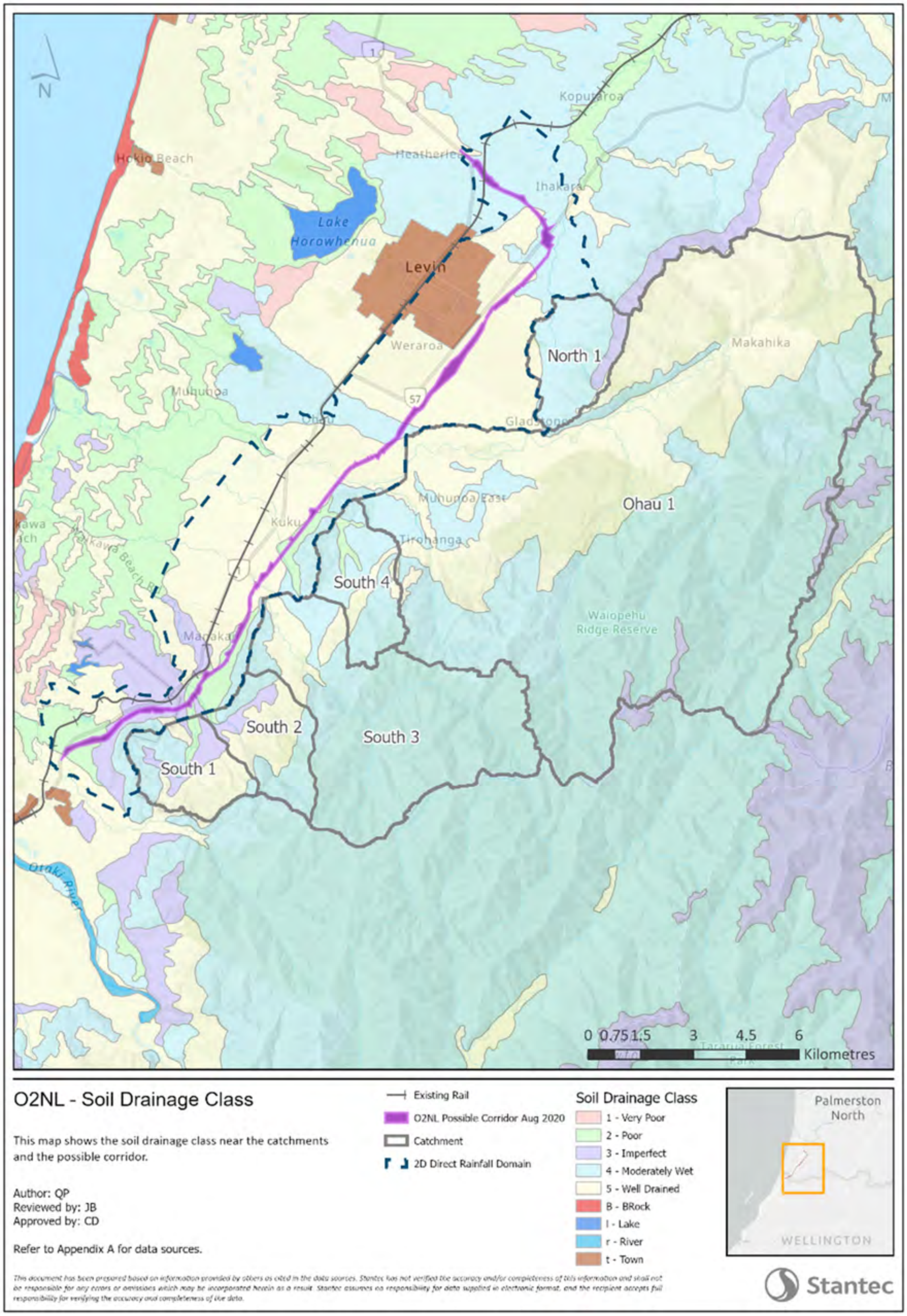




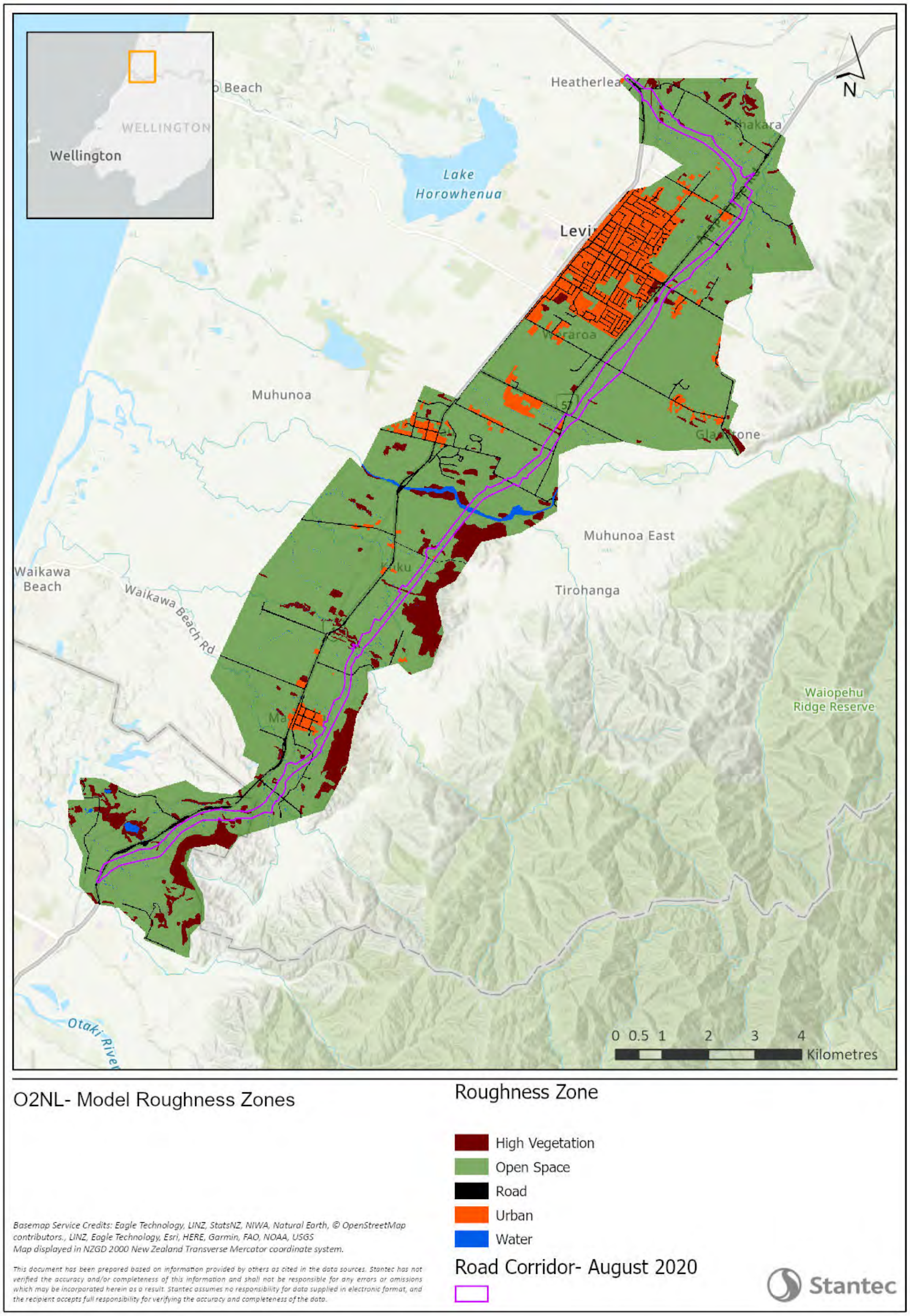
Jan 2008	Jun 2015	Dec 2009
		
Large event circa 1:10, rain spatially variable 1st peak over predicted at both sites, 2nd peak over at Manakau but **good at Waikawa**, 3rd peak observed at Mankau but not observed at Waikawa or Ōhau or most raingauges, hence would be difficult to match in model. Note Otaki rainfall no event data.	**Large event circa 1:10**, rain spatially variable and two peaks close together. 2nd peak observed at Manakau and Ōhau but hardly visible at Waikawa. Overall fit of peak magnitude at both sites is good, although model attenuation results in the two peaks mixing together, partly also a function of spatially varying rainfall.	Small double event, rain spatially variable, 1st peak way over at both Mankau & Waikawa although note the very high first hit of rainfall in the hills so would be difficult to get better model with such high rainfall. 2nd peak fits better at both sites although Manakau under.
		
		
		

Dec 2014	Feb 2018	Nov 2019
		
Small event, rain spatially variable, low at Manakau, good fit at Waikawa	Small event, **rain TIMING VERY variable**, peak values are good at both sites although early due to the use of some high/early raingauge data	Small event, rain variable, fit reasonable (mix of over and under)
		
		
		

Appendix D FSL Drainage Class

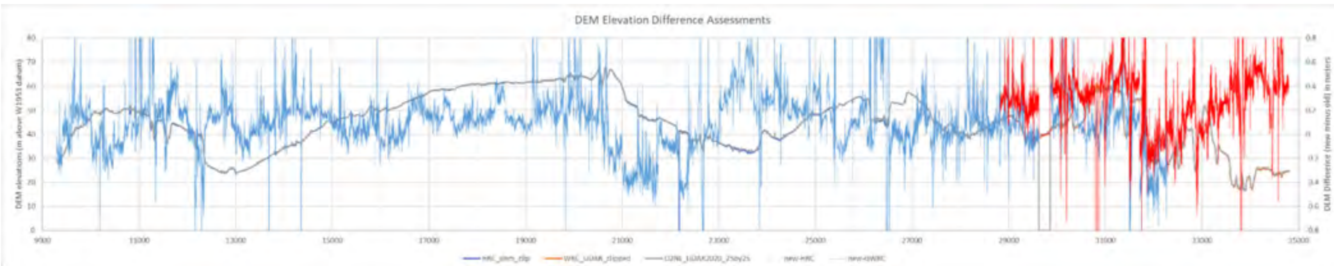


Appendix E Roughness Map

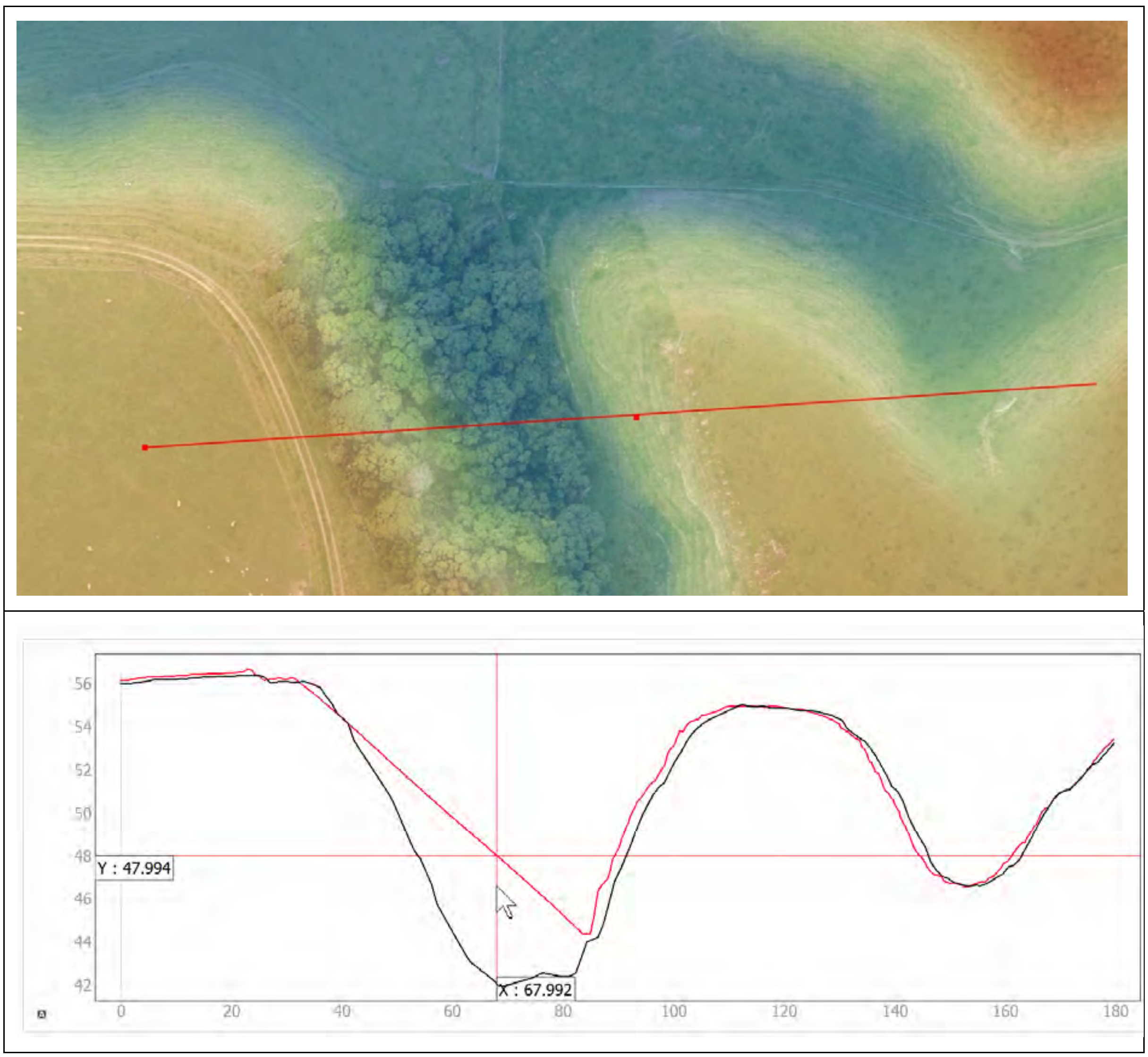


Appendix F DEM Issues

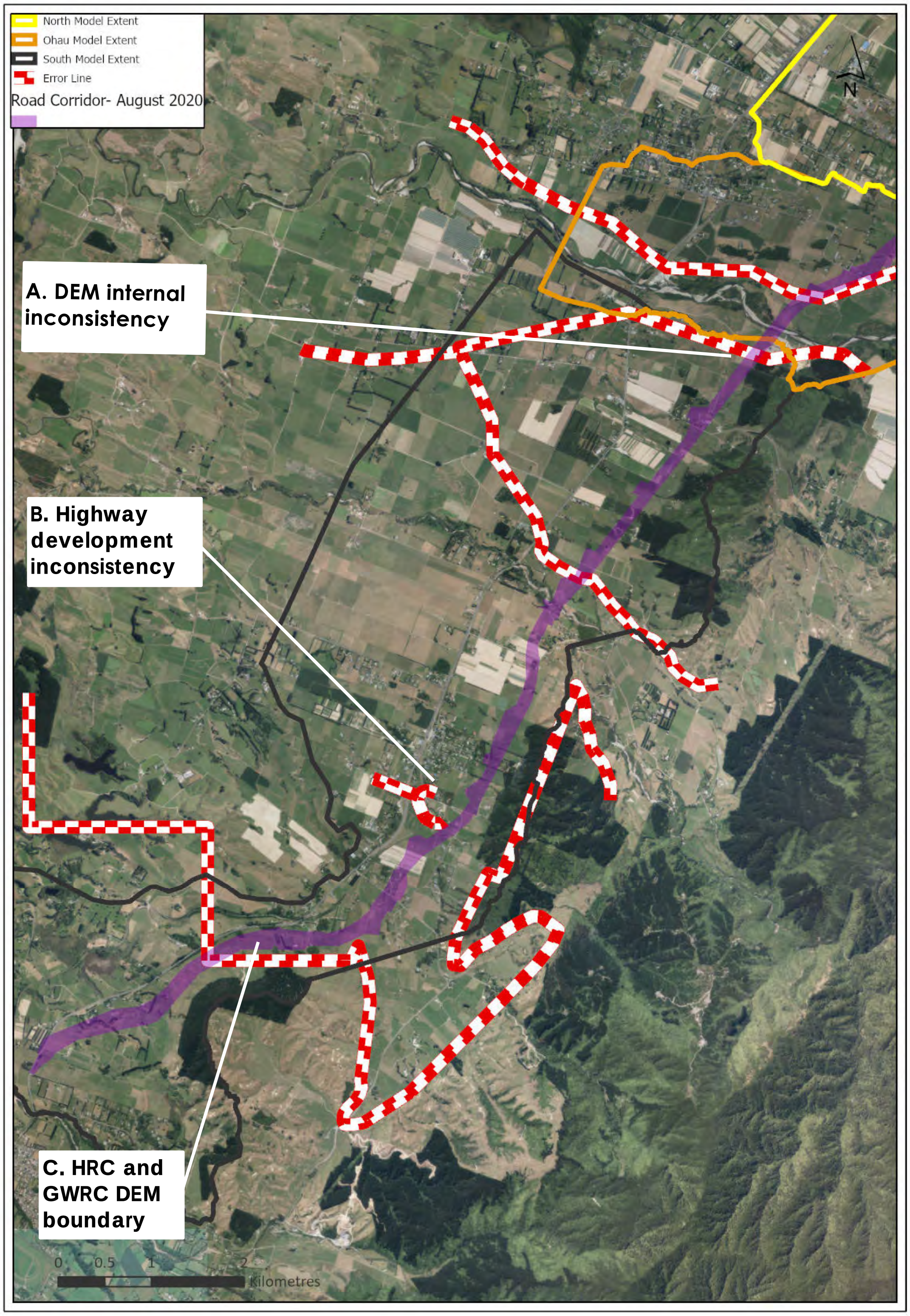
DEM differences assessment (see Section 5.2.1). The three thicker lines show the extents and elevations from the key datasets along a draft Ō2NL route, although the errors cannot be visually assessed at this scale. Therefore, the thin lines show the <u>differences</u> between the datasets, plotted on the right hand (secondary) axis. This shows moving average differences vary from +0.6 to -0.4, with localised noise and many localised deviations of much larger magnitude. The choices and mitigation measures to deal with these differences are discussed in Section 5.2.1.



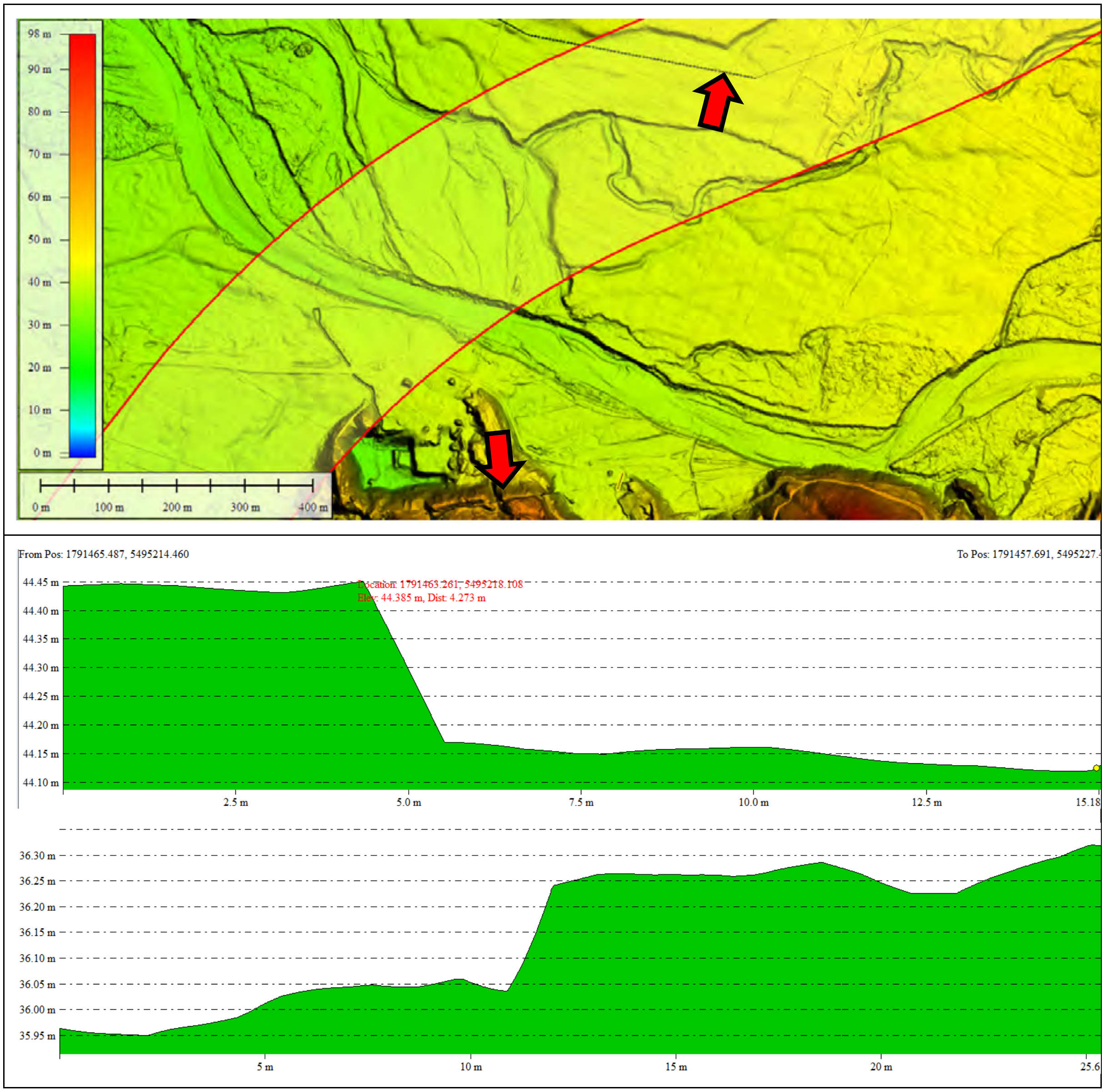
Example of missing data in 2020 drone DEM causing poor vertical interpolation in a flow path due to heavy vegetation. This area in the 2020 drone DEM (red line in cross section) will be patched using the Regional Council DEM (black line in cross section), as discussed in Section 5.2.1.



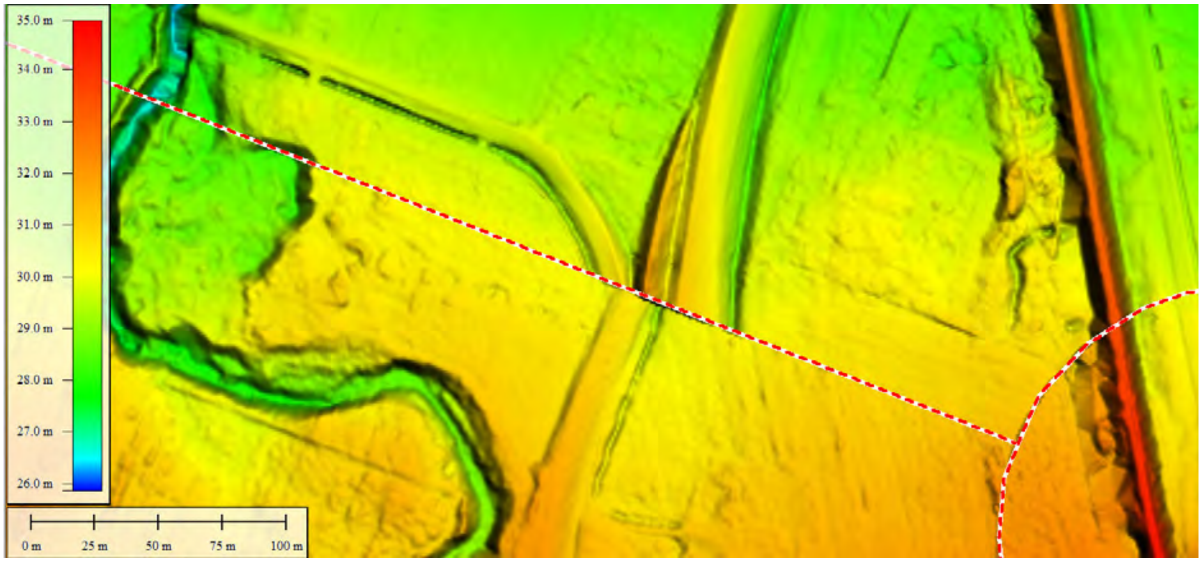
The figure below highlights some long error lines containing significant step discontinuities that are not associated with real features, as mentioned in 5.2.4. Some further description is provided below the figure.



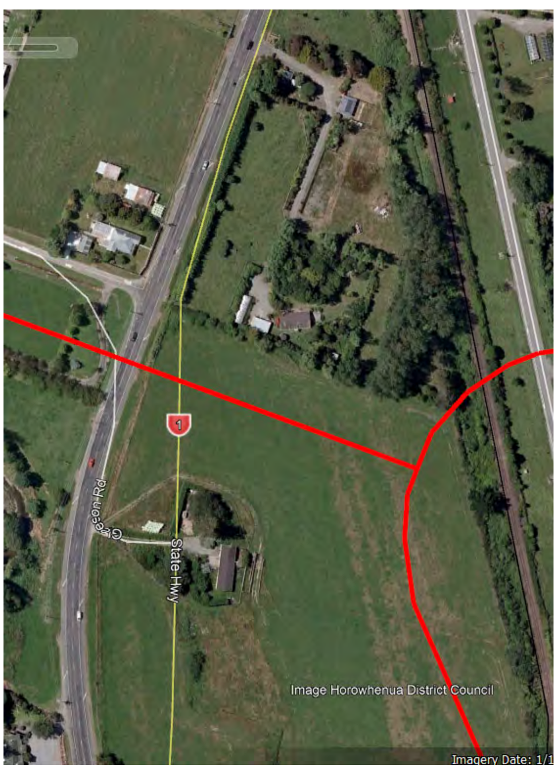
The issue A in the figure above is an internal inaccuracy in the HRC DEM. These differences in terrain appear to be due tile merging of LiDAR data datasets. A profile along this line is shown below Appendix F. The differences between tiles range between 0.2m and 0.3m.



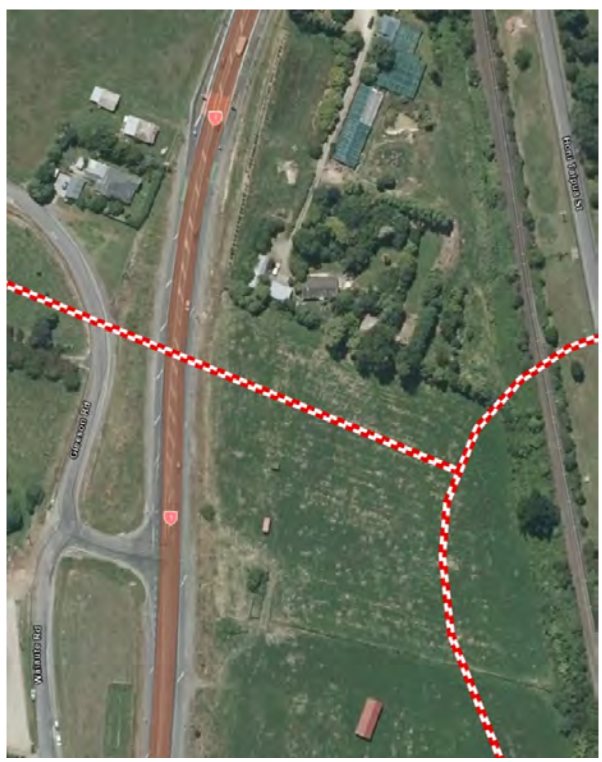
The inconsistency at location B was due to SH1 modifications since the LiDAR was captured, and the LiDAR contains some discontinuities, most notably on the new SH1 alignment although this is downstream of the Ō2NL corridor and downstream of the railway embankment (a major hydraulic impediment) therefore the results in this location will not have any influence on results in Ō2NL corridor.



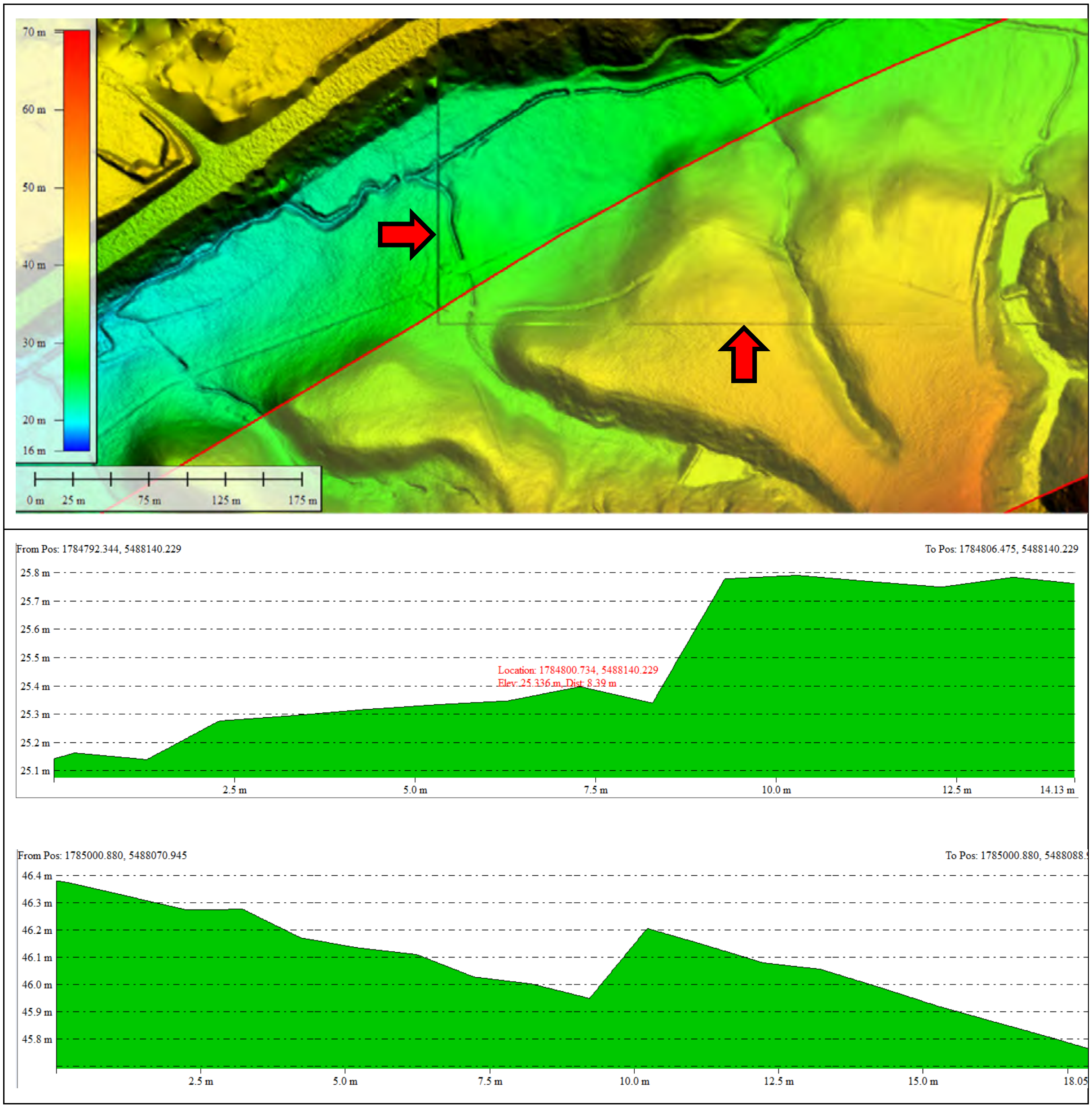
Aerial 2010



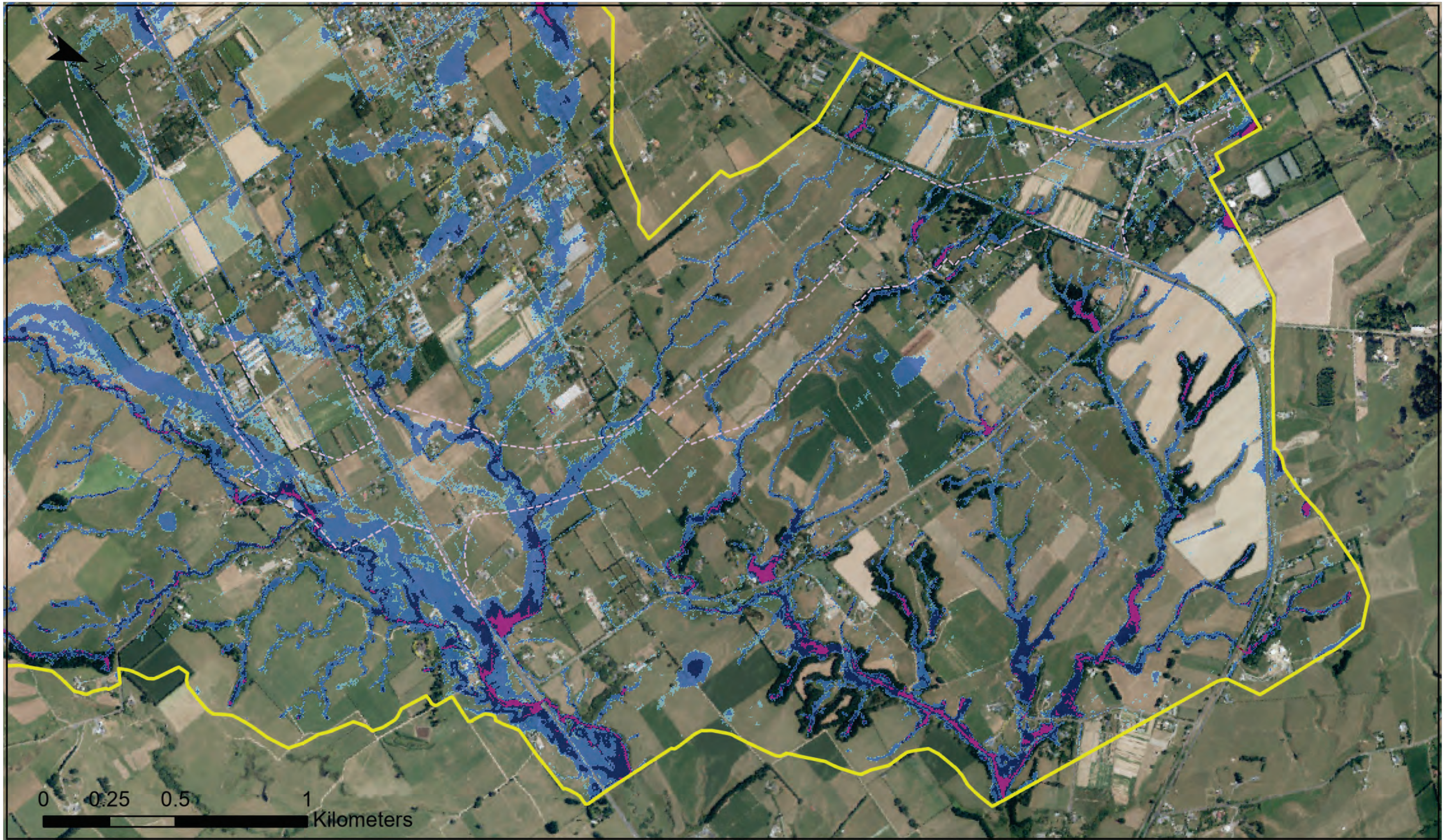
Aerial 2019



Issue C relates to the merge between the HRC and the GWRC terrain models, where the two datasets contain differences in the order of 0.2m to 0.3m.



Appendix G Baseline Results



Model Results, Maximum Water Depths 1:100 AEP With Climate Change RCP 6.0 2130

Storm duration: 4hr for the Ohau and North models and max of 4hr and 1hr for the South model

South Model Extent
Ohau Model Extent
North Model Extent
Proposed Designation July 2022

Max. Water Depth (m)

0.05 to 0.1
0.1 to 0.5
0.5 to 1
1.0 to 3.0
Greater than 3

Data Sources:Basemap Service Credits: LINZ, Stats NZ, Eagle Technology, Esri, HERE, Garmin, Foursquare, FAO, METI/NASA, USGS, Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors
Map displayed in NZGD 2000 New Zealand Transverse Mercator coordinate system.
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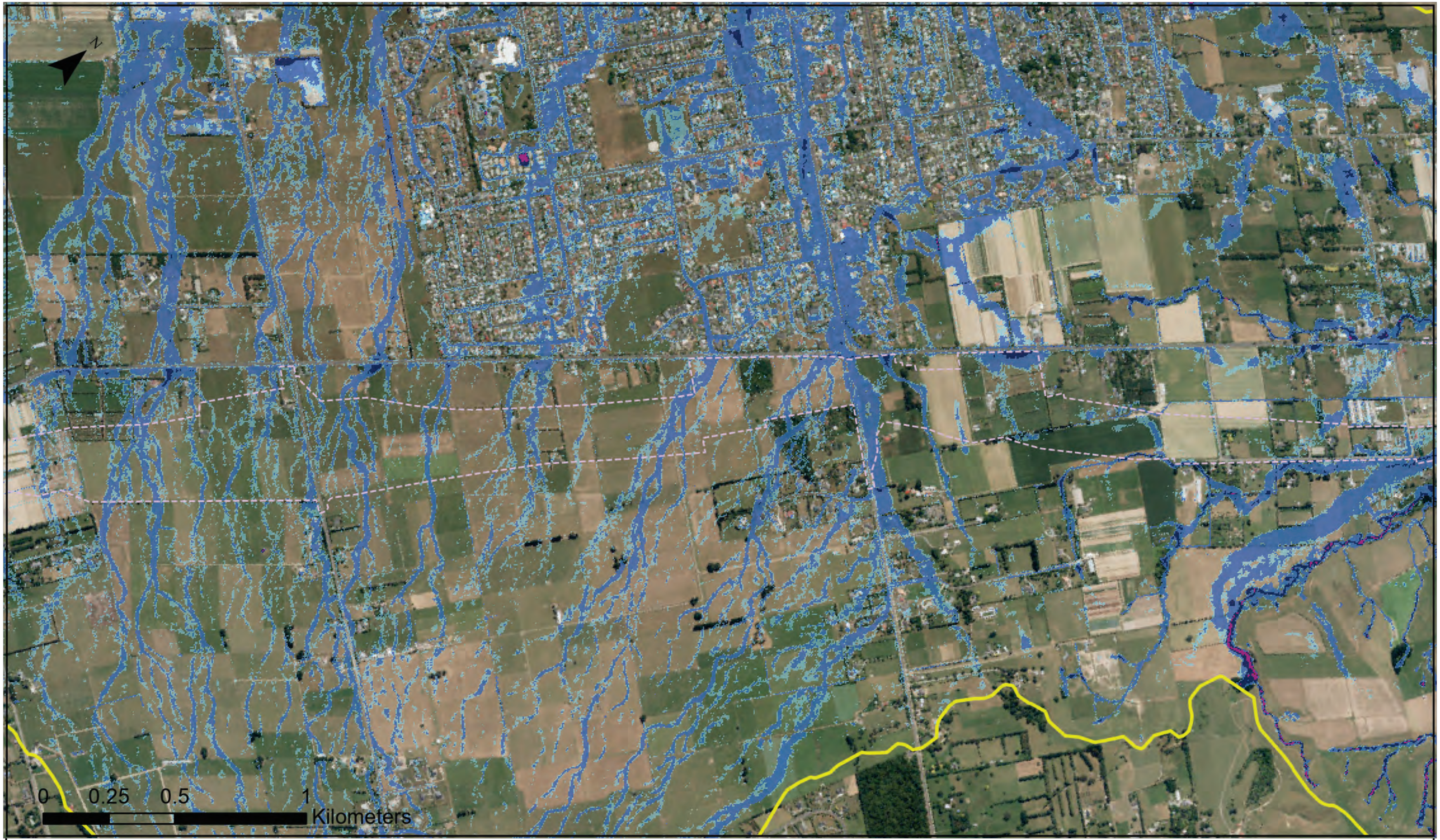

Model Results, Maximum Water Depths 1:100 AEP With Climate Change RCP 6.0 2130

Storm duration: 4hr for the Ohau and North models and max of 4hr and 1hr for the South model

South Model Extent

Ohau Model Extent

North Model Extent

Proposed Designation July 2022

Max. Water Depth (m)

- 0.05 to 0.1
- 0.1 to 0.5
- 0.5 to 1
- 1.0 to 3.0
- Greater than 3

Data Sources:Basemap Service Credits: LINZ, Stats NZ, Eagle Technology, Esri, HERE, Garmin, Foursquare, FAO, METI/NASA, USGS, Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

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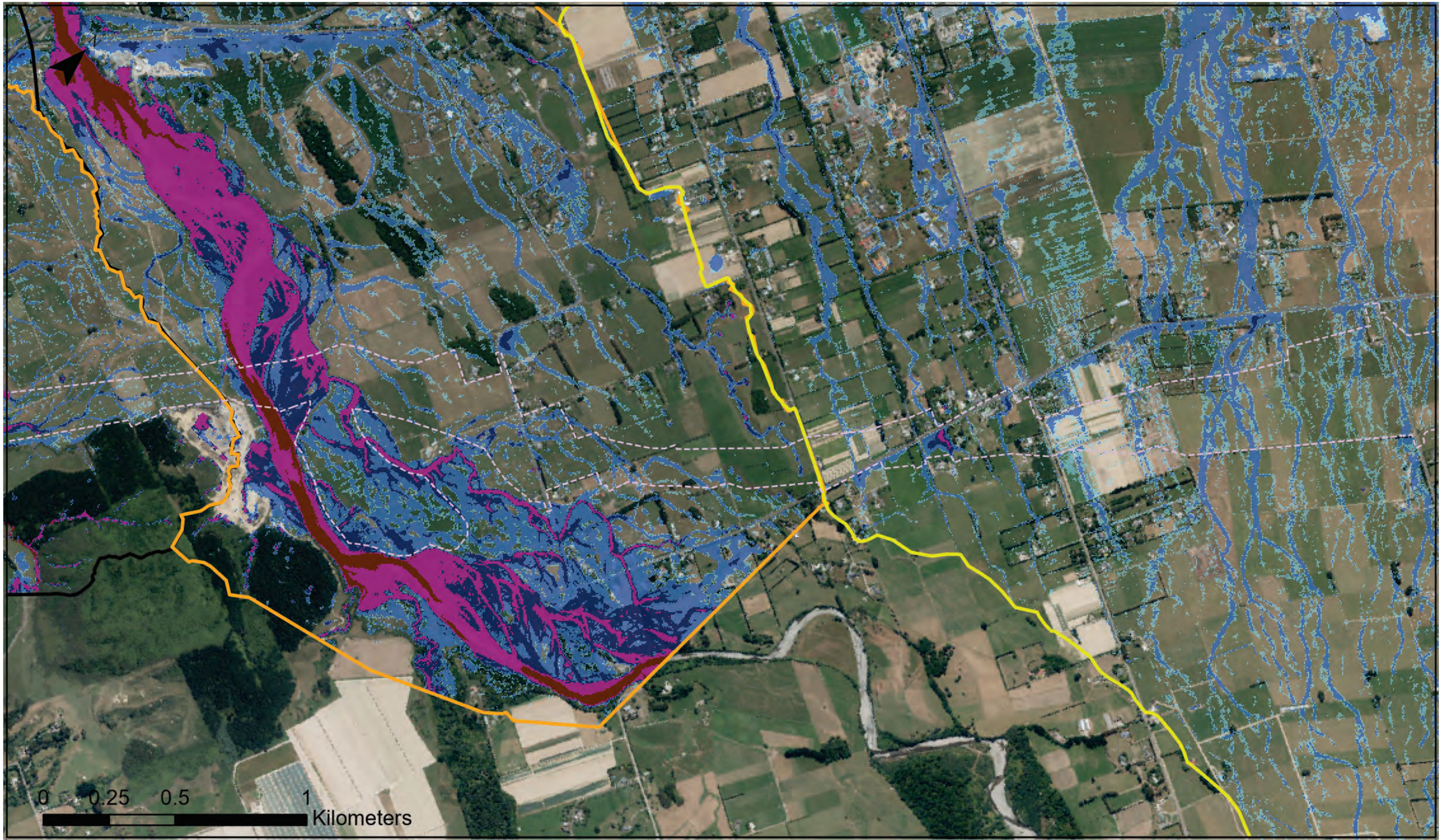

Model Results, Maximum Water Depths 1:100 AEP With Climate Change RCP 6.0 2130

Storm duration: 4hr for the Ohau and North models and max of 4hr and 1hr for the South model

South Model Extent
Ohau Model Extent
North Model Extent
Proposed Designation July 2022

Max. Water Depth (m)

0.05 to 0.1
0.1 to 0.5
0.5 to 1
1.0 to 3.0
Greater than 3

Data Sources:Basemap Service Credits: LINZ, Stats NZ, Eagle Technology, Esri, HERE, Garmin, Foursquare, FAO, METI/NASA, USGS, Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

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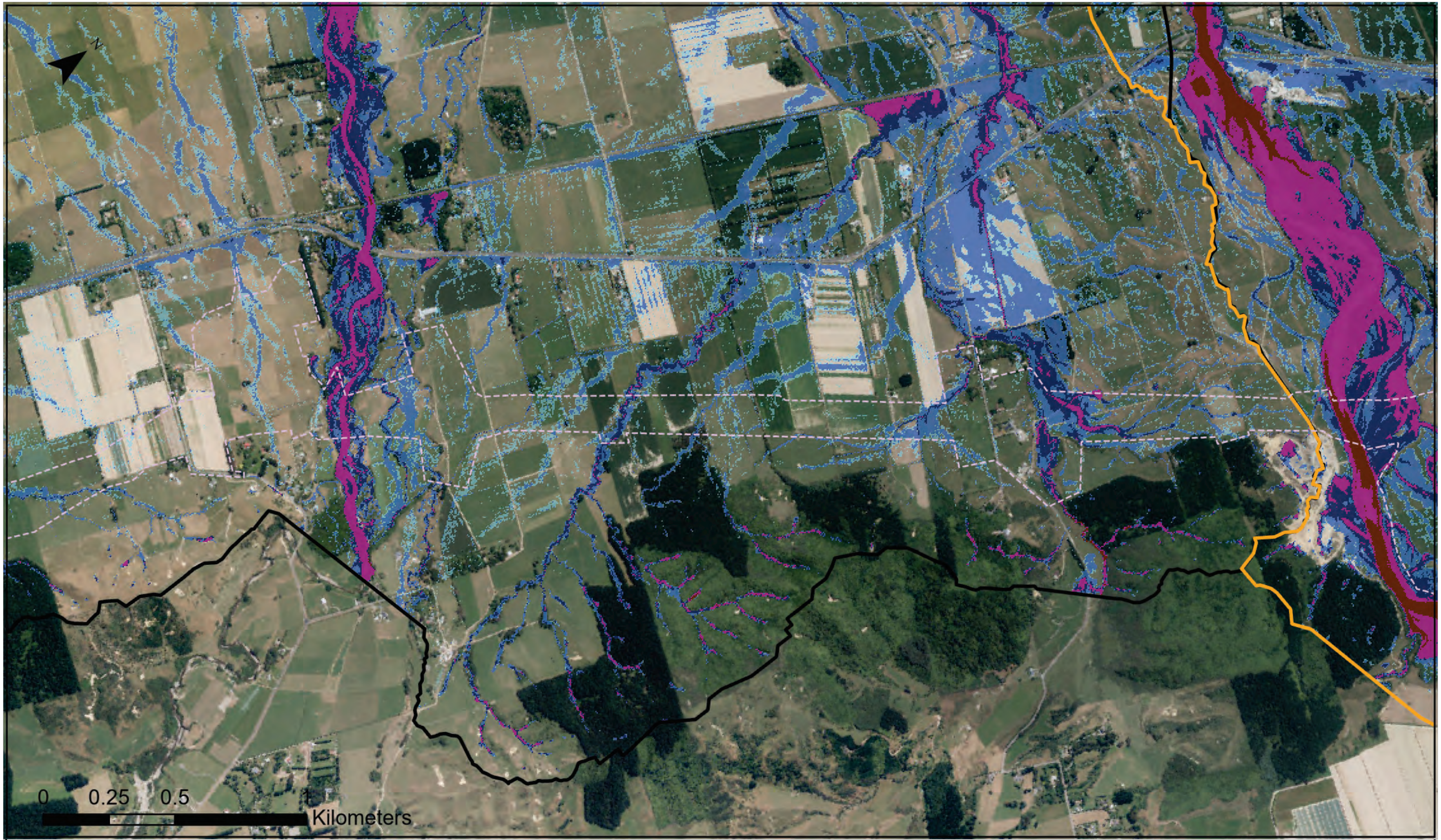

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Storm duration: 4hr for the Ohau and North models and max of 4hr and 1hr for the South model

Stantec

South Model Extent
Ohau Model Extent
North Model Extent
Proposed Designation July 2022

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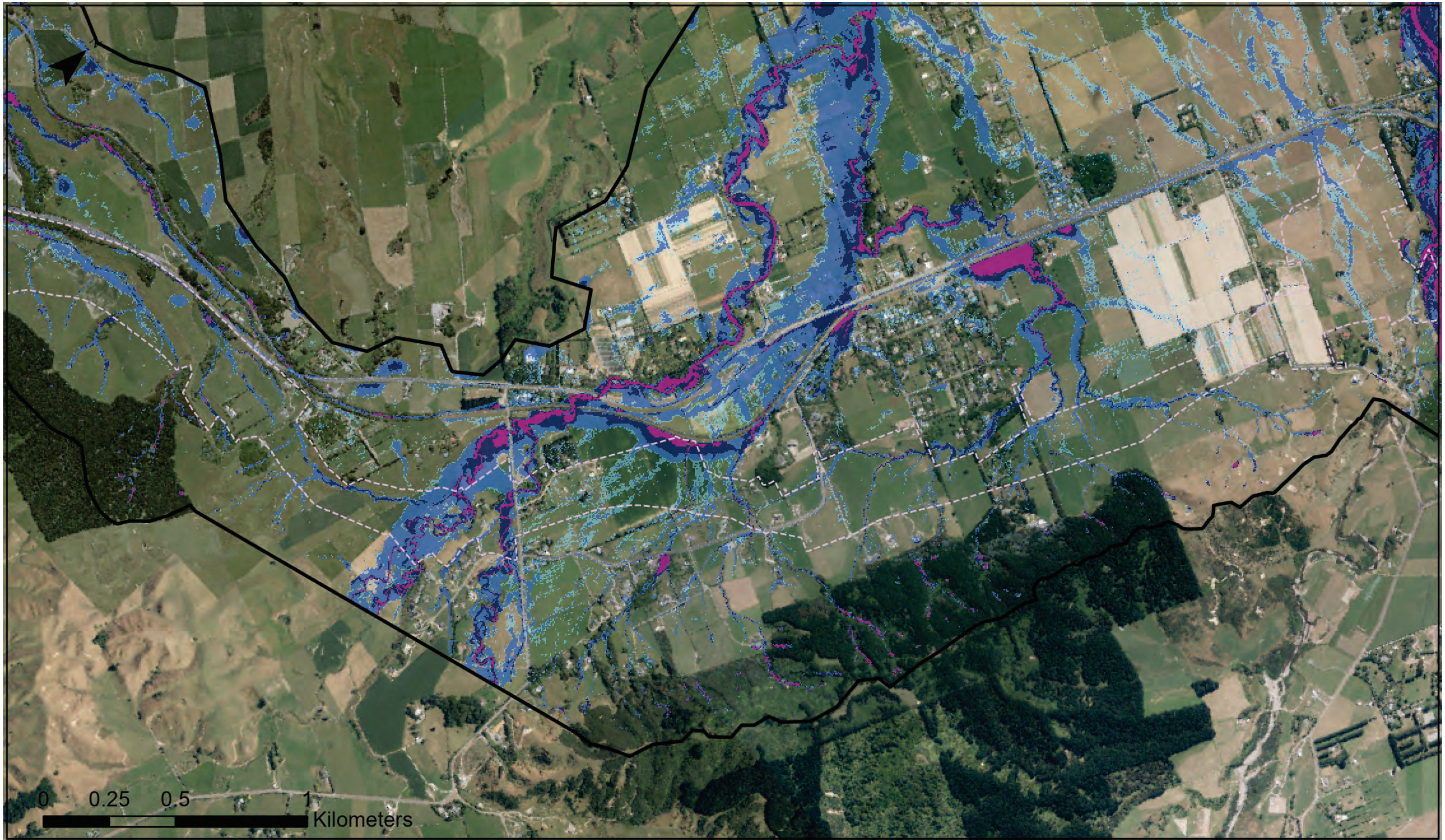

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Ohau Model Extent

North Model Extent

Proposed Designation July 2022

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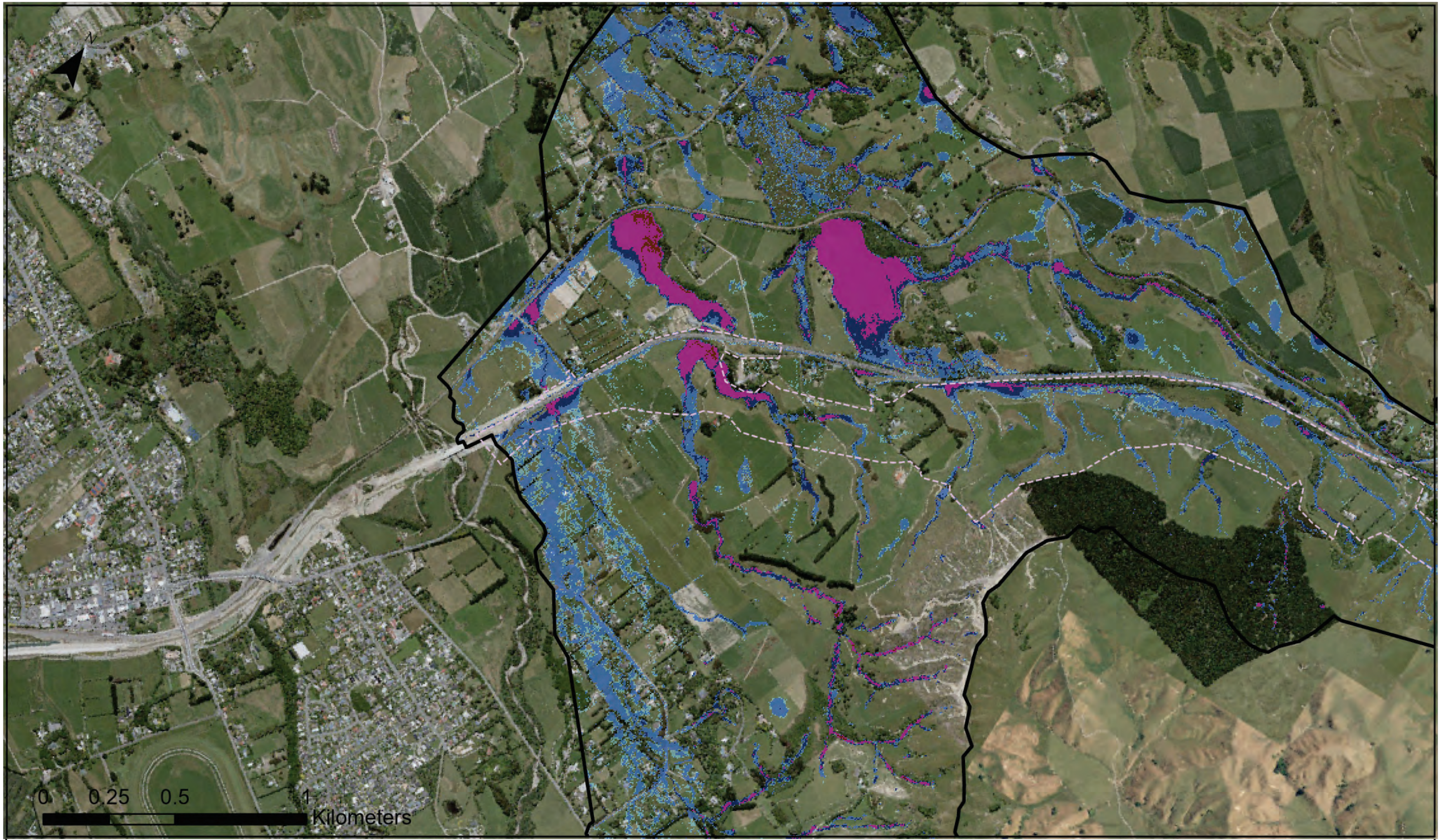

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South Model Extent
Ohau Model Extent
North Model Extent
Proposed Designation July 2022

Max. Water Depth (m)

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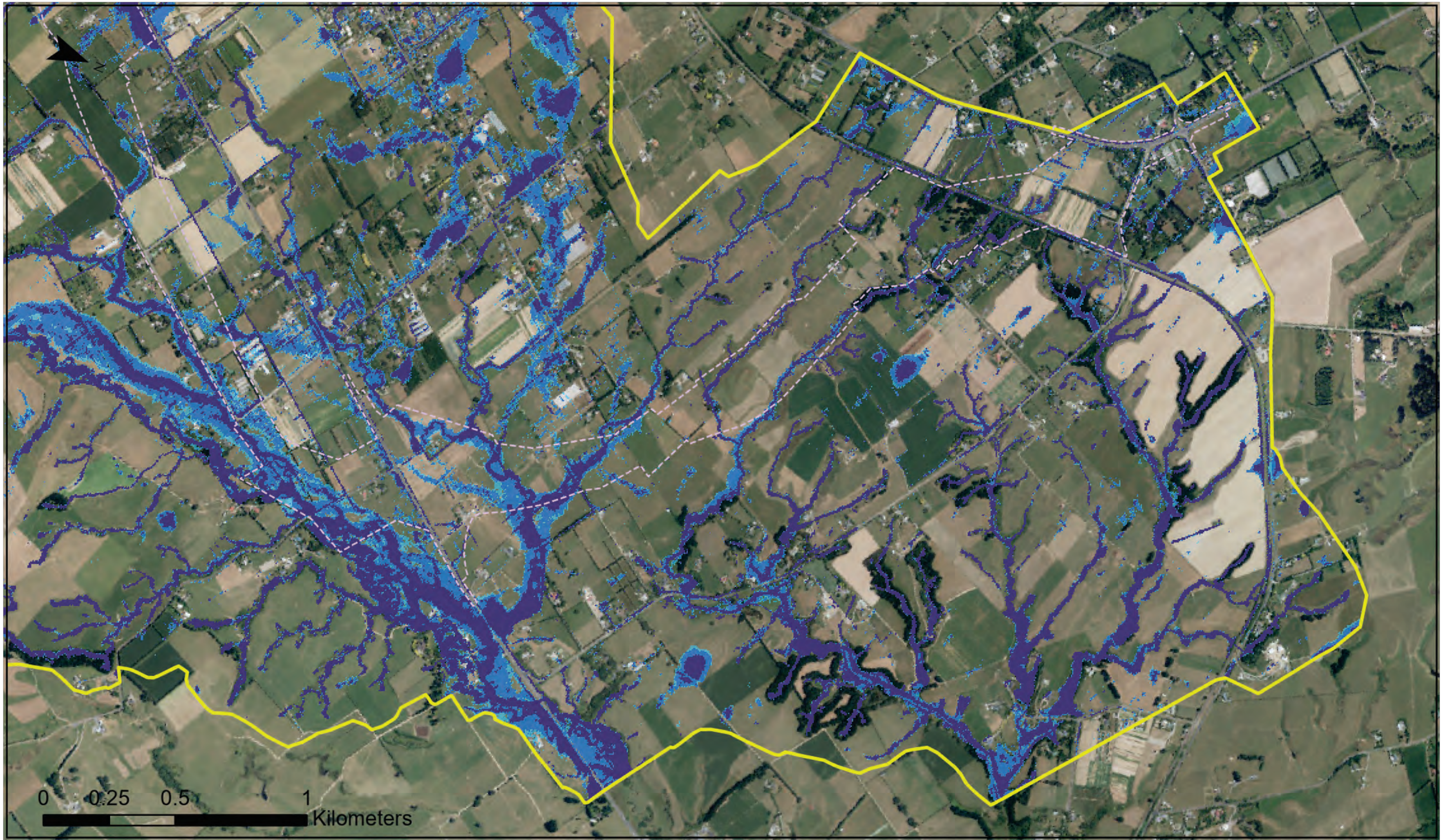

Model Results, Baseline Maximum Extents

This map shows maximum flood extents for various annual exceedance probabilities. No flood depths less than 0.05m are shown

South Model Extent
Ohau Model Extent
North Model Extent
Proposed Designation July 2022

1:10 AEP Current Climate
1:100 AEP RCP 6.0 2130
1:1500 AEP RCP 8.5 2130

Data Sources:Basemap Service Credits: LINZ, Stats NZ, Eagle Technology, Esri, HERE, Garmin, Foursquare, FAO, METI/NASA, USGS, Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors
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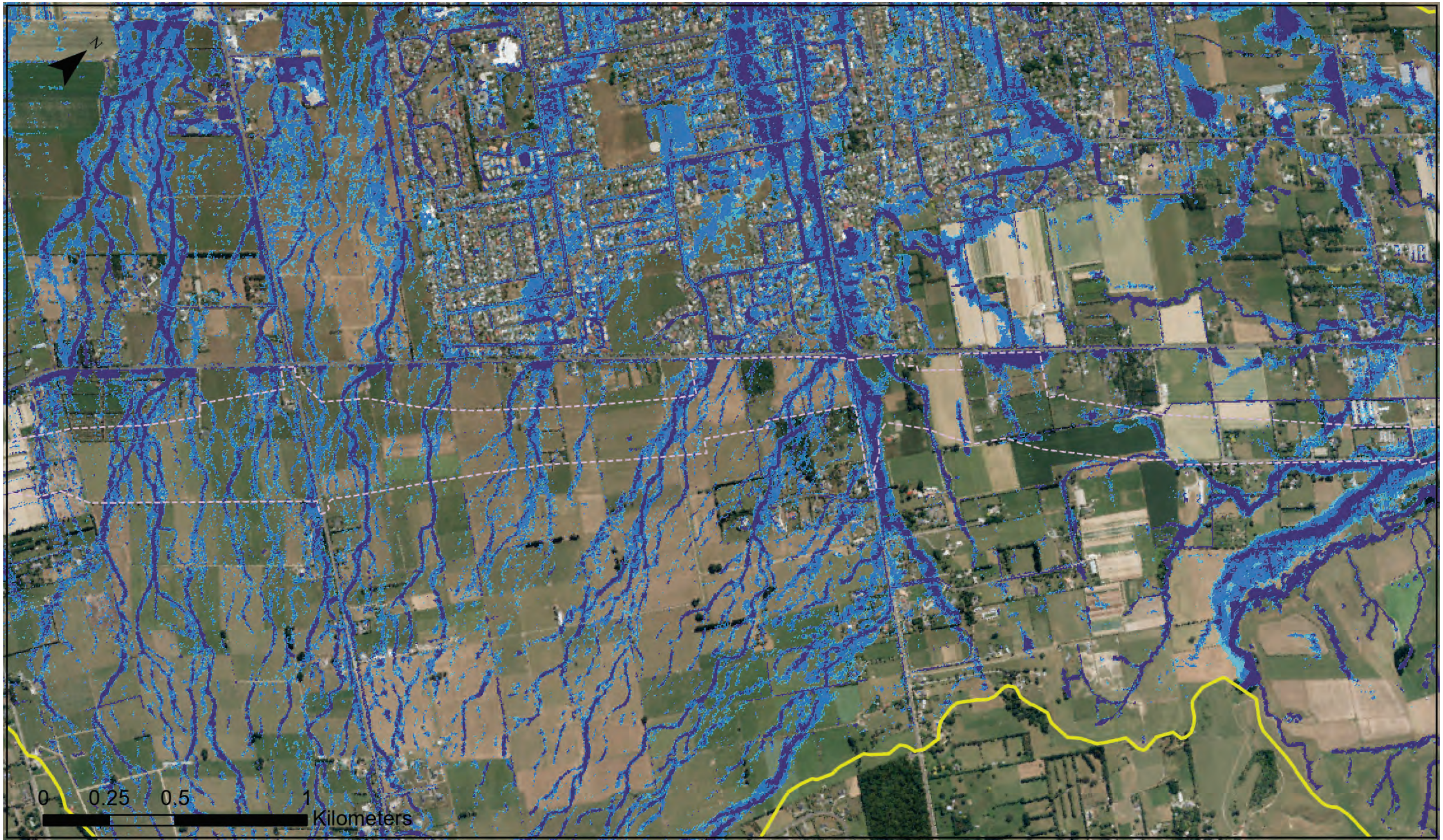

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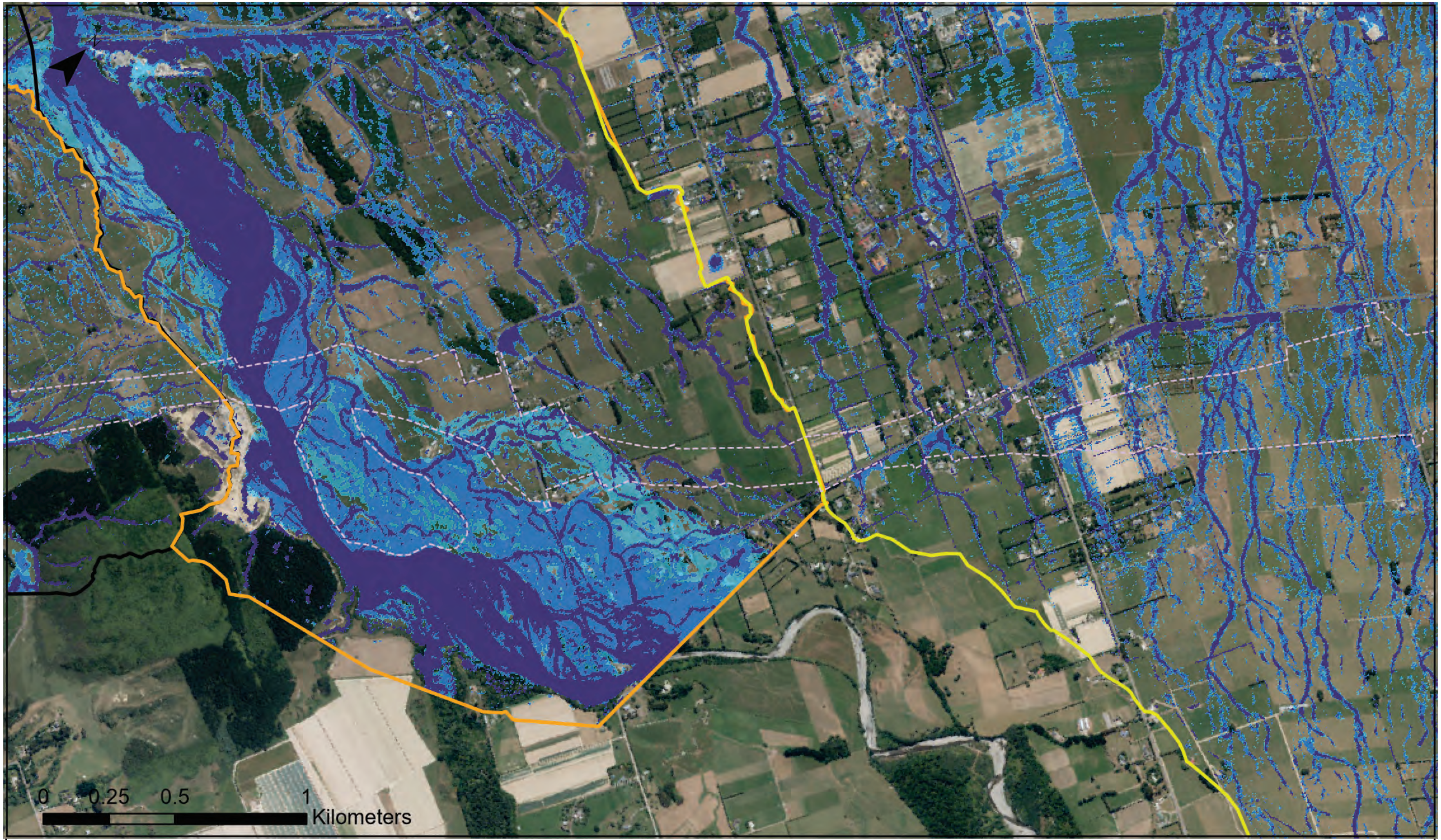

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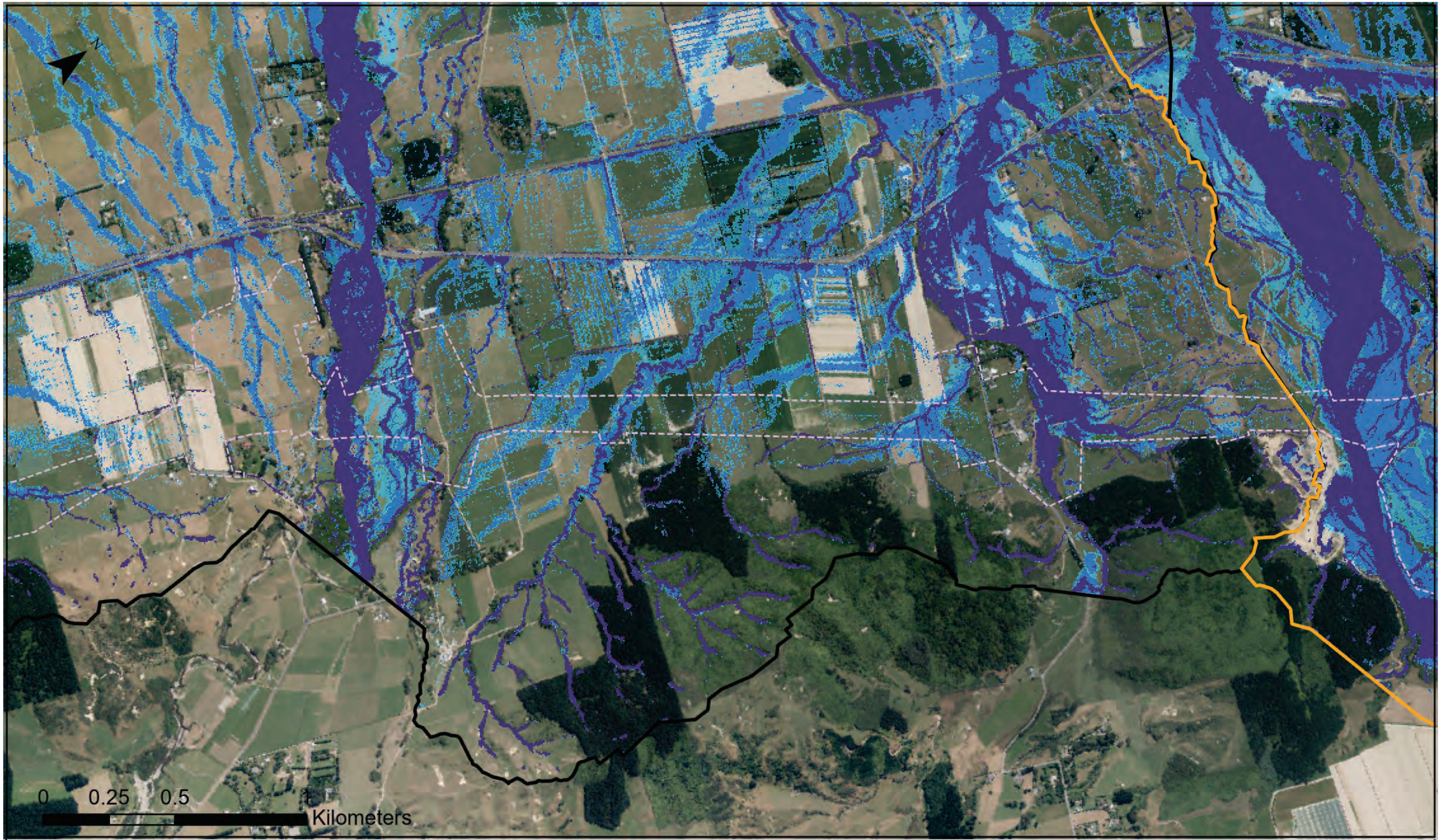

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