

TE KOWHAI EAST, TE KOWHAI,
HAMILTON

HYDRAULIC MODEL REPORT

2026

Version Control

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1	13/08/2026	James Oakley Waqas Sarwar	Mike Chapman	Final Draft
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QUALIFICATIONS, EXPERIENCE AND CODE OF CONDUCT

The qualifications and experience of the author and technical director/reviewer of this report are summarised below. We confirm that we have read the Code of Conduct for Expert Witnesses Contained in the Environment Court of New Zealand Practice Note 2023 and we agree to comply with it. We confirm that the issues addressed in this report are within our area of expertise, except where we state that we are relying on the evidence or advice of another person. We have not omitted to consider material facts known to me that might alter or detract from the opinions expressed in this report.

Report Authors

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Waqas Sarwar is a highly skilled water resources engineer with specialized expertise in GIS automation and 1D/2D hydrologic and hydraulic modelling. He has led the development and refinement of complex stormwater management models across the Waikato region, including stream restoration initiatives, urban flood hazard mitigation strategies, and integrated catchment modelling for stormwater runoff estimation. Waqas holds a Ph.D. in Water Resources Engineering from The University of Waikato, where his research focused on catchment-streamflow interactions. He is proficient in a wide range of three-waters modelling platforms, including ICM, DHI, TUFLOW, HEC-RAS (1D/2D), EPA-NET, and EPA-SWMM, among others.

Report Reviewer

Mike Chapman is a Director of Te Miro Water and Principal Water Resources Engineer with over 25 years of experience in flood risk management, stormwater design, and catchment planning in New Zealand and England. He has led and reviewed over 30 Catchment Management Plans and provides technical direction across all Te Miro Water projects including the Rotokauri Greenway project, Rotorua Airport Flood Resilience Project and Ruakura ICMP, among others. Mike plays a key role in technical governance and quality assurance, ensuring all project outputs meet regulatory and client expectations. He is recognised for his expertise in flood risk assessment and has been involved in numerous award-winning and large-scale infrastructure and planning projects. He holds a BSc in Environmental Science and an MSc (Hons) in Hydrology from The University of Waikato.

EXECUTIVE SUMMARY

A pre- and post-development TUFLOW hydraulic model was developed incorporating the proposed development earthworks, eight constructed wetlands with attenuation and retention storage, drainage infrastructure, culverts and the modified drainage network. The modelling assessed the 2-year, 10-year and 100-year ARI events, considering peak flows, water levels, velocities, runoff volumes and drain-down times. A range of sensitivity scenarios were also assessed to test the robustness of the proposed stormwater management strategy.

The modelling demonstrates that the proposed stormwater management strategy effectively controls development-related increases in peak runoff. Post-development peak flows in the Mangaheka and Te Rapa Streams are generally at or below the relevant pre-development baseline across the assessed design events.

Peak downstream water levels are also generally reduced post-development, including at key hydraulic constraints such as Horotiu Road and Crawford Road Bridge. The modelling identifies no adverse afflux or reduction in the existing flood level of service as a result of the development.

Post-development velocities are generally comparable to, or lower than existing conditions and remain generally below the adopted velocity threshold at the main Mangaheka Stream reporting locations. At Crawford Road Bridge, elevated velocities are associated with the existing hydraulic restriction and are localised to the bridge opening. The development does not worsen the existing Crawford Road constraint in terms of peak water level or velocity, although the modelling indicates a slight increase in the duration of ponding on the bridge during larger events, this is not considered to materially worsen the existing flood hazard or level of service.

The principal residual effect identified by the modelling is an increase in runoff volume and a corresponding increase in the duration of elevated water levels. In the larger storm events, this results in longer drain-down times within the downstream drains. Within the WRC-managed McBeth Drain, the modelling confirms that the drain continues to satisfy its required drainage level of service.

In the smaller storm events, the increase in runoff volume results in prolonged duration of lower velocity flows in the Mangaheka Stream channel, albeit these are lower than the velocity threshold at which erosion is expected to occur, based on the soil types.

Sensitivity testing demonstrates that additional at-source retention (via soakage or re-use) could provide further reductions in downstream flows, velocities and runoff volumes. However, full volume neutrality is not considered to be required for the proposed stormwater management strategy to achieve acceptable hydraulic performance and the adopted peak-flow and water-level targets.

Both the sensitivity assessments and an independent peer review have been undertaken. The sensitivity testing did not alter the overall conclusions of the assessment, while the peer review identified no fundamental issues with the modelling methodology or its application. Matters raised during the peer review were subsequently addressed, with no outstanding issues remaining.

Overall, the modelling demonstrates that the proposed stormwater management system is capable of managing the effects of the development without material adverse effects on downstream peak flows, flood levels or velocities. The residual effects are primarily associated with increased runoff volume and longer-duration drainage response. The assessment therefore provides a sound basis for concluding that the proposed stormwater management strategy is appropriate for the proposed development.

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1 WHAKATAKINGA | INTRODUCTION

This technical report has been prepared in support of the Te Kowhai East (TKE) development. It summarises the hydrologic assessment and hydraulic modelling undertaken to establish the existing, or baseline, stormwater runoff characteristics of the site and the drainage conditions within the surrounding receiving environments, namely the Mangaheka Stream and Te Rapa Stream.

This report has been prepared to support Te Kowhai East Limited Partnership's substantive application for approvals under the Fast-track Approvals Act 2024 (FTAA).

The assessment has been used to define the pre-development hydrologic response of the catchment, including runoff volumes, peak flows, water levels, velocities, and drainage behaviour under a range of design storm events. These baseline conditions provide the reference against which post-development effects associated with the proposed Te Kowhai East development have been assessed.

The hydraulic model has been developed to simulate pre- and post-development runoff conditions across the site and wider contributing catchments. The hydraulic model has undergone an external peer review by Colin Roberts at 89 Degrees Ltd. The peer review summary is provided in Appendix B.

For the post-development scenarios, the model incorporates the proposed earthworks, stormwater management devices, including constructed wetlands with attenuation storage, culverts, drains, and other mitigation measures proposed as part of the development. The model assesses the effectiveness of the stormwater management strategy in controlling runoff rates and volumes, managing water levels and velocities, maintaining conveyance through existing drainage systems, and limiting adverse effects on downstream receiving environments. The model is not intended to set catchment wide flood levels. Rather, its purpose is to establish a suitable baseline for determining existing runoff rates and volumes, and to assess the effects of the proposed development and mitigation measures on the downstream environment. It is also not intended to represent actual or probable flooding within existing built-up areas of the sub-catchment.

The modelling results have been used to quantify changes between the existing and proposed development scenarios at key reporting locations within the Mangaheka Stream and Te Rapa Streams. This includes assessment of changes in peak flow rates, runoff volume, flood extent, water level, velocity, and drain-down duration, as well as the potential implications of those changes for channel stability, flood risk, land drainage performance, and effects on downstream infrastructure.

This report demonstrates the proposed stormwater mitigation measures are sufficient to manage development-related changes in runoff and to confirm the nature and magnitude of any residual effects on the downstream receiving environments.

1.1 THE PROJECT

The Project is a large-scale industrial development located on the north-western edge of Hamilton, adjacent to the established Te Rapa industrial area. The application provides for the creation of an integrated industrial precinct, including approximately 210 industrial lots, land for neighbourhood centre activities, reserves for recreation, stormwater management and ecological mitigation, and associated roading and infrastructure services. The application also seeks approval for the ultimate land use outcomes anticipated across the site, being predominantly industrial activities, along with a limited range of compatible non-industrial activities suited to an industrial environment. The development will form a logical extension of Hamilton's primary industrial node within the Upper North Island economic corridor and responds to projected growth and demand.

A comprehensive description of the Project is provided in the Substantive Application Report (SAR).

2 WHAKAMĀRAMA WHĀNUI | BACKGROUND

2.1 SITE DESCRIPTION

The Project site comprises a large, contiguous landholding located on the north-western edge of Hamilton. It is generally bounded by Te Kowhai Road to the south, Koura Drive and State Highway 1C to the east, and Onion Road to the north. The site is made up of multiple land titles, legal descriptions, and associated physical addresses.

For identification purposes, the site is commonly referred to by the primary addresses of 270 Te Kowhai Road and 28 Mathers Road. Full details of the landholdings are provided in the Substantive Application Report (SAR). The site is identified in Figure 1 below.



Figure 1 Site location (Source Adapt Studio)

2.2 INTEGRATED CATCHMENT MANAGEMENT PLANS

The site forms part of two distinct stream catchments – the Mangaheka and Te Rapa Streams. Integrated catchment management plans (ICMP) have been developed by Hamilton City Council (HCC) for both stream catchments and are summarised below. The ICMP's have been used as a starting point in developing a Basis of Design (BoD) for stormwater management on the site.

2.2.1 Mangaheka ICMP

The Mangaheka Integrated Catchment Management Plan (ICMP), (Revision 4.0, January 2019) provides the framework for stormwater management to support ongoing urban expansion, as rural land is progressively converted to light industrial and commercial land uses such as the proposed Te Kowhai East development.

The ICMP applies to the wider Mangaheka catchment, which covers approximately 2,080 ha and straddles the HCC and Waikato District Council (WDC) boundaries. The upper catchment lies within Hamilton City and includes much of the Rotokauri Structure Plan area, where significant industrial development is anticipated. The lower catchment is located within Waikato District and is predominantly rural land, falling within the Waipa and Central Waikato zone management plan areas administered by Waikato Regional Council (WRC).

The ICMP provides an integrated framework for managing stormwater effects associated with urbanisation in the catchment. Its primary purpose is to identify best practicable options to avoid, where practicable, and otherwise minimise cumulative adverse effects from stormwater discharge activities as development progresses. While the ICMP primarily focuses on urbanisation within HCC's jurisdiction, some recommended management options extend into WDC land and are therefore relevant to development within the lower catchment i.e. the proposed Te Kowhai East development.

2.2.1.1 Previous Hydraulic models

As part of the Mangaheka ICMP BECA Ltd had developed a hydrologic and hydraulic model for the stream catchment. At the outset of the Te Kowhai East project use of the Beca model was discussed with Council subject matter experts and it was concluded that the model is outdated in terms of landuses within the catchment and climate change requirements, accordingly, it was considered not fit for purpose to determine baseline runoff targets for the development.

2.2.2 Te Rapa ICMP

The Te Rapa Catchment is located in the north of Hamilton, on the western side of the Waikato River. The headwaters and mid-catchment are located in the HCC jurisdictional area with the lower portion of the catchment within the WDC area. The Te Rapa Stream flows in a generally northerly direction, with the Waikato River confluence located approximately 400m downstream of the Horotiu Bridge. In addition to the main Te

Rapa Stream, two small ephemeral watercourses which discharge directly to the Waikato River are also included in the ICMP area. The total area of the catchment is approximately 1,140 ha, with approximately half of this lying within the HCC jurisdictional area.

The Te Rapa Integrated Catchment Management Plan (ICMP) (Revision 0.3 Final- December 2025) has been prepared to support an integrated approach to managing potable water, wastewater and stormwater within the Te Rapa catchment. Its purpose is to accommodate urban growth and changing land use while ensuring three waters activities comply with relevant legislation, statutory planning documents, Hamilton City Council's Comprehensive Stormwater Discharge Consent, and industry best practice.

The ICMP seeks to avoid, where practicable, and otherwise minimise the cumulative adverse effects of three waters activities on the catchment and its receiving environments. It also aims to ensure that the existing levels of service for water, wastewater and stormwater networks are not compromised, while optimising the use of existing infrastructure and reducing the need for new infrastructure where practicable. The ICMP provides a framework for ongoing decision-making, maintenance, infrastructure planning, and future connectivity with adjoining land and catchments.

The ICMP also enables future subdivision and development stormwater diversion and discharge activities to be authorised under Hamilton City Council's Comprehensive Stormwater Discharge Consent, where those activities are consistent with a Waikato Regional Council-certified catchment management plan. To support this, the ICMP identifies catchment values, objectives, flood hazards, water-sensitive design principles, implementation measures and developer requirements relevant to managing growth and stormwater effects within the Te Rapa catchment.

The ICMP sets a course towards the restoration and protection of the health and wellbeing of the Stream and its receiving water, the Waikato River. This includes restoring and protecting water quality, biodiversity, and the relationship of Waikato-Tainui and Hamilton residents with the Te Rapa Stream, protecting significant sites, and improving access to the Te Rapa Stream for recreation and cultural activities. It also includes involving mana whenua in freshwater management, including decision-making processes. There is an expectation that all who live in the catchment will contribute to restoring and protecting the health and wellbeing of the Stream. This extends beyond simply avoiding or mitigating the adverse effects of activities to contributing towards environmental enhancement or betterment. The Waikato-Tainui Environmental Plan identifies the state of the environment that should be aspired to, is that which existed in 1863.

2.3 BASIS OF DESIGN

A stormwater Basis of Design (BoD) has been developed in consultation with the relevant stakeholders, including WRC, HCC, and WDC. The BoD establishes the agreed framework for stormwater design for the Te Kowhai East development and provides the basis for demonstrating consistency with applicable HCC and WRC

requirements, including the Mangaheka and Te Rapa North ICMPs, and alignment with the Waikato Regional Land Drainage Management Plan.

Where applicable, stormwater design will be undertaken in accordance with the Waikato Regional Infrastructure Technical Standards (RITS v2), the Waikato Stormwater Management and Modelling Guidelines (TR 20-06 and TR 20-07), the Mangaheka ICMP (2019), the Te Rapa North ICMP, and the Land Drainage Management Plan (2019).

The BoD sets out the agreed technical basis for stormwater management across the Te Kowhai East development. It defines the performance expectations, design criteria, and technical standards to be applied before hydrologic modelling and detailed design are progressed.

The adopted BoD is found appended within the Stormwater Management Plan.

2.4 COORDINATE SYSTEM AND VERTICAL DATUM

The following coordinate and datum systems have been used for the modelling and mapping:

- Projection: New Zealand Transverse Mercator 2000 (NZTM2000)
- Vertical datum: New Zealand Vertical Datum 2016 (NZVD2016)

3 HYDROLOGY

3.1 CATCHMENT DELINEATION AND TOPOGRAPHY

The Te Kōwhai East (TKE) development lies primarily within the upper Mangaheka Stream catchment, a low-gradient system draining north-west toward the Waipā River. A small portion of the upper eastern end of the development area discharges to the east under the Waikato Expressway and to the Te Rapa Stream and ultimately the Waikato River.

As described above, the wider Mangaheka catchment covers approximately 2,080 ha. Of this, around 86% of the catchment (the “lower catchment”) is within the Waikato District but is part of a Land Drainage Area managed by WRC. The remaining 14% (the “upper catchment”) lies HCC jurisdiction. The TKE site is located near the upper extent of the WRC-managed lower catchment, within the Waikato District.

Existing land use within the lower catchment is predominantly rural, while the upper catchment contains mainly commercial and light industrial development.

The model and mapping extent of for the Te Kowhai East area was developed by importing LiDAR into QGIS and extracting the overland flow paths (OLFPs). The LiDAR and OLFP were inspected for major blockages and refined as required. The Waikato Expressway culverts and upstream Industrial area were scrutinised to ensure accuracy.

The adopted hydraulic model extent is shown in Figure 2 below.



Figure 2 Te Kowhai East Model Boundary

3.2 SOILS, GROUNDWATER, AND HYDROGEOLOGY

3.2.1 SOIL

Geotechnical investigations by CMW Geosciences indicate that the site is underlain predominantly by interbedded sands and silts of the Hinuera Formation across the entire extent of the low-lying plains with the rolling hill topography underlain by a sequence of Walton Subgroup soils consisting of weathered volcanoclastic and alluvial deposits comprising clays, silts and sands. The full Geotechnical Investigation Report by CMW can be found in the substantive application documents.

CMW have confirmed that the low-lying plains are generally overlain by a silt loam (typically 0.5m – 1.0m thick), underlain by sand / silty sand with interbedded silt layers (non-colloidal) is reflective of a Silt Loam (non-colloidal) material. This material present in the low-lying plains and drains within the site has a corresponding and has a corresponding maximum permissible velocity of 0.61m/s as per Table 7-1 in the Waikato Regional Council Stormwater Management Guidelines (TR2020/07). Refer to Figure 3 below for a screenshot of the table.

Table 7-1: Maximum permissible velocities⁸⁴

Material	Velocity (m/s)
Fine sand (colloidal)	0.46
Sandy loam (noncolloidal)	0.53
Silt loam (noncolloidal)	0.61
Alluvial silt (noncolloidal)	0.61
Ordinary firm loam	0.76
Volcanic ash	0.76
Fine gravel	0.76
Stiff clay	1.14
Graded loam to cobbles (noncolloidal)	1.14
Alluvial silt (colloidal)	1.14
Graded silt to cobbles (colloidal)	1.22
Coarse gravel (noncolloidal)	1.22
Cobbles and shingles	1.52
Shales and hard pans	1.83

Figure 3 Table 7-1 from TR2020/07

The soil type and drainage class is taken from SMAP. The SMAP soil drainage categories are well-drained, moderately well-drained, imperfectly drained, poorly drained, and very poorly drained. The soil siblings report from Soil Map Online (SMAP) is used to determine the soil hydrology groups (HGS) relevant to the site and catchment, and corresponding CN values are used accordingly. Figure 4 and Figure 5 shows the soil drainage types for the Te Kowhai East sub-catchments. The HGS make up of different SMAP soils in the TKE area is presented in Table 1 below.

Table 1: HGS composition of different SMAP soil groups present in the study area.

SMAP Drainage Class	HGS			
	A	B	C	D
Well drained	100%			
Moderately well drained		100%		
Imperfectly drained			100%	
Poorly drained				100%
Very Poorly drained				100%

3.2.2 HYDROGEOLOGY

Site investigations undertaken and groundwater monitoring undertaken by CMW and WGA confirm shallow groundwater conditions across the site, with water levels ranging from 0.5 m to 1.0 m below ground level (bgl) during winter and seasonally lowering to approximately 2.0 m bgl in the summer months. Each piezometer exhibits relatively similar responses and trends, which is assessed as indicative of a connected shallow groundwater system across the site.

3.2.3 SOIL CLASSIFICATION

As per the above, investigations have confirmed seasonal groundwater fluctuations of up to 2m between the winter and summer months within the site extents. Accordingly, as per TR2020/06 a dual soil classification has been adopted for the site.

- Low ground water / Summer conditions: Soil Type B/D (moderately permeable, suitable for soakage). These soil classifications are based on S-map data.
- High ground water / Winter conditions: Soil Type D (restricted infiltration due to high groundwater).

Although site investigations indicate that groundwater levels reduce during summer, S-map classifies a significant portion of the site as Soil Type D due to the influence of elevated groundwater conditions. This classification has been retained for those areas in the low groundwater / summer scenario, notwithstanding the observed seasonal reduction in groundwater levels, as a conservative representation of runoff response. This approach avoids underestimating runoff from areas that may remain partially constrained by shallow groundwater or poor drainage characteristics. Soil classifications elsewhere within the model extent have been adopted from SMAP data and adjusted only where the dual soil classification methodology is applicable. Refer to Figure 4 and Figure 5 below for adopted soil drainage types in the winter and summer conditions.

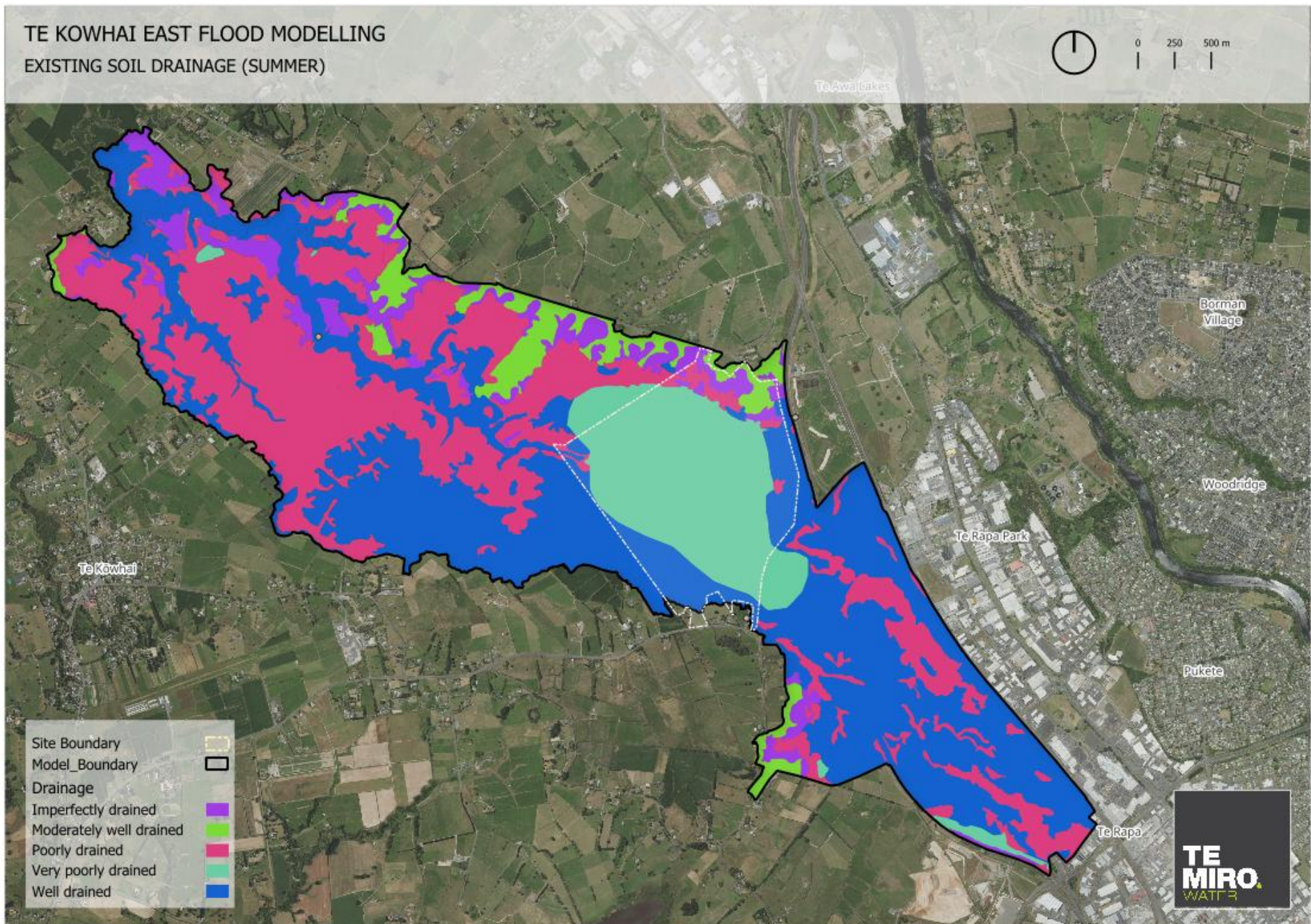


Figure 4 Soil drainage types in TKE ROG zone extracted from SMAP (Summer Ground Water conditions).

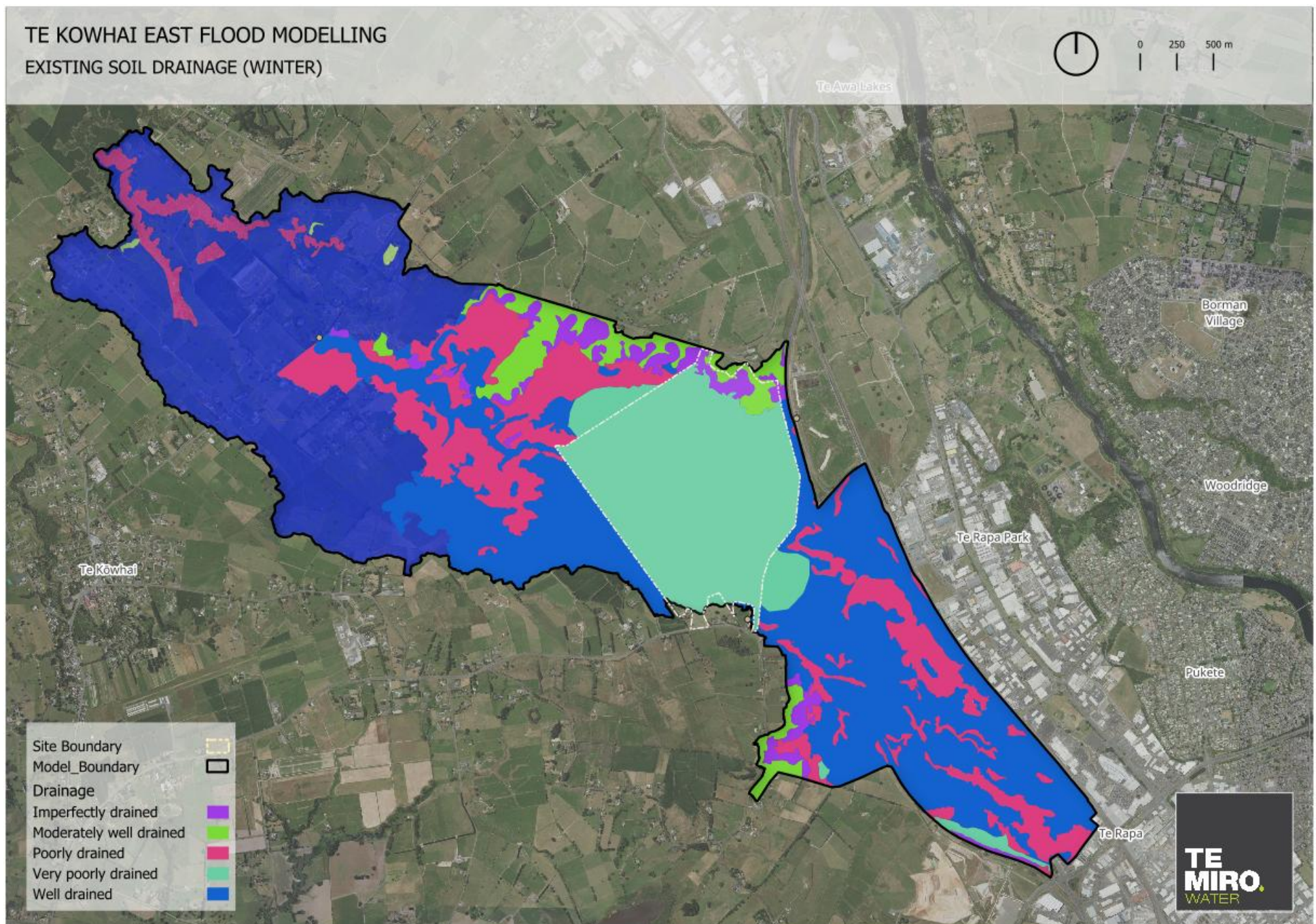


Figure 5 Soil drainage types in TKE ROG zone (Winter Ground Water conditions).

3.3 LAND USE

Existing land use cover within the catchment is extracted from the LCDB v5.0 layer, which is available at the LRIS data portal. Similar land cover categories from LCDB are merged to create simpler major land cover categories in the area. The land cover map of Te Kowhai East model area is shown in Figure 6. Analysis of the aerial image and site visits were used to verify the land cover data. The areas with dense vegetation that were not included in the landcover layer were manually added, and associated parameters were adjusted. The land cover layer was manually refined in the areas of main flow paths and water bodies to represent the roughness of the surface better i.e. the main Manguheka channel and densely vegetated floodplain were delineated and assigned different roughness values where applicable.

The majority of the undeveloped catchment comprises a mixture of farmland (grasslands), cropland and forest and is assumed to be fully pervious. The majority of the external catchments is covered by Te Rapa Industrial development, located east of SH1C and a parcel of future urban zone (FUZ) land between Koura Drive and the Waikato Expressway.

3.4 IMPERVIOUS AREAS ASSESSMENT

In the pre-development model % impervious values have been assigned based on existing landuses. Where applicable land uses have been assigned the maximum % impervious allowed in the Waikato and Hamilton City District Plans based on current and future land use as per the below.

The existing Te Rapa industrial area has been assigned a 90% impervious coverage reflective of a maximum probable development scenario. The portion of land between the expressway and Koura Drive is earmarked for FUZ and has been assigned 50% impervious coverage in both the existing and post development scenarios. It is understood this area could be developed as industrial in which case the actual % impervious could be higher than this model allows for, however the development will be required to manage any post-development increases in runoff and so the % impervious adopted in this model is considered conservative.

The fraction impervious percentages for each zone are presented in Table 2 below.

Table 2: Fraction impervious used for different growth zones.

ZONE	PERCENTAGE IMPERVIOUS
RURAL	100% impervious for buildings and hardstands and 100% pervious for grassed areas
INDUSTRIAL	90
ROADS	80
RURAL RESIDENTIAL	30
FUZ RESIDENTIAL	50

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TE KOWHAI EAST FLOOD MODELLING

LAND USE - EXISTING

Horahia



0 250 500 m

Te Awaaitakes

Borman Village

Woodridge

Te Rapa Park

Pukete

Te Rapa

TE MIRO
WATTS

- Site Boundary
- Model_Boundary
- Existing Land Use
- Cropland
- Forest
- Grass
- Highway
- Industrial
- Residential
- Water

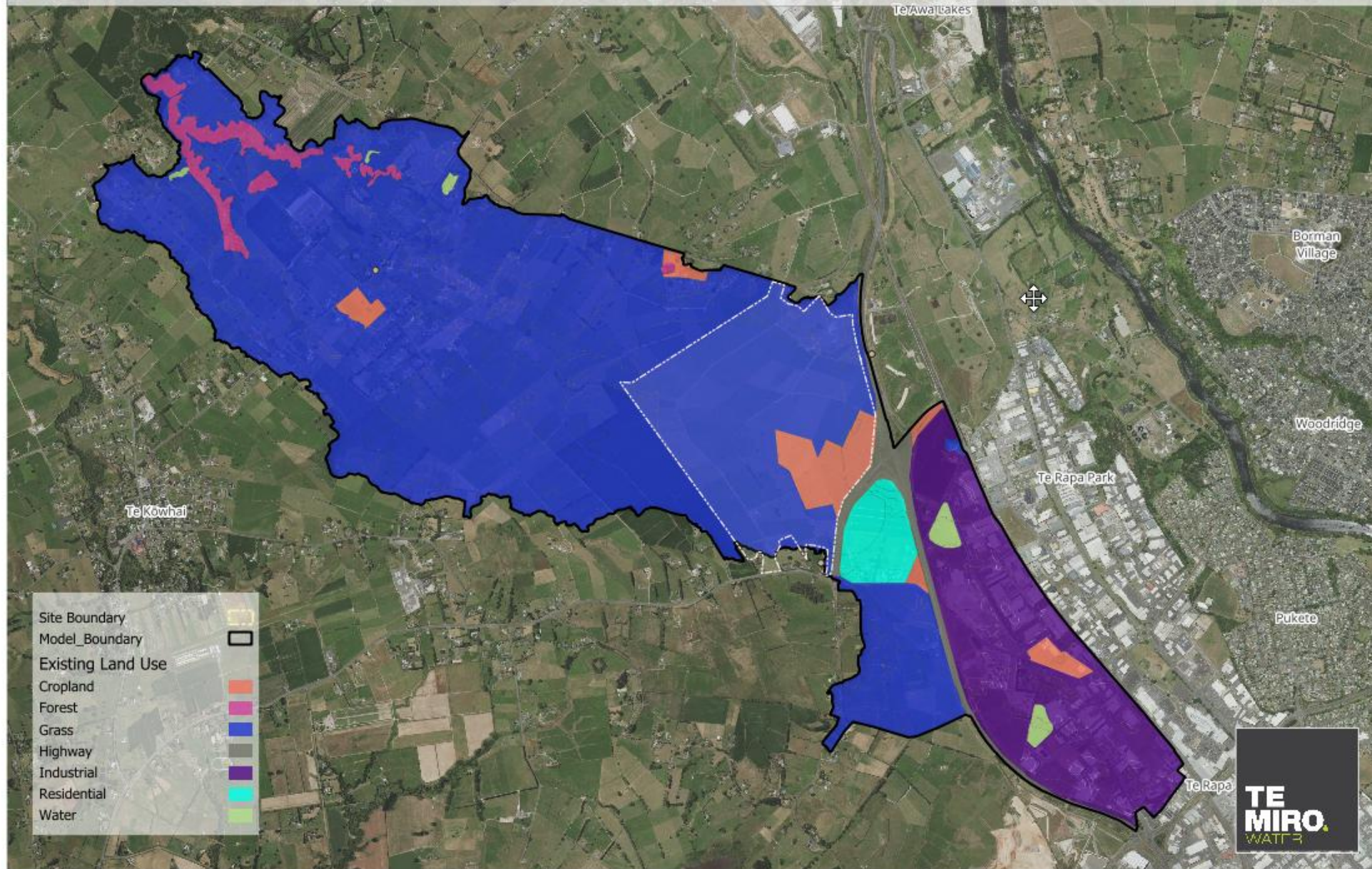


Figure 6 Existing Land use map of Te Kowhai East

3.5 CURVE NUMBERS

3.5.1 EXISTING DEVELOPMENT

Spatial variability in catchment rainfall–runoff response is represented by assigning distributed Curve Number (CN) values across the model domain based on mapped land use and hydrologic soil group data, in accordance with TR2020/06. Composite CN values are applied where necessary to account for mixed land cover within individual model cells.

For the SCS application in TUFLOW, the initial abstraction is defined using an abstraction ratio rather than as a fixed depth. The abstraction ratio represents the proportion of the potential maximum retention, S , that must be satisfied before rainfall excess and runoff are generated. For this assessment, an initial abstraction ratio of $0.05S$ has been adopted, consistent with Waikato Regional Council guidance. Refer Table 3 for adopted IA values.

The CNs for the existing scenario are determined using the HGS, land use and the percentage impervious area, as presented in Table 3.

Table 3: Existing Scenario Curve numbers and % Impervious values calculated based on soil drainage and landcover.

SOIL ID	LAND USE - DRAINAGE	% IMP	FINAL CN	IA(MM)
101	Cropland - Imperfectly drained	0%	85	2.24
102	Cropland - Moderately well drained	0%	78	3.58
103	Cropland - Poorly drained	0%	89	1.57
104	Cropland - Very poorly drained	0%	89	1.57
105	Cropland - Well drained	0%	67	6.26
201	Forest - Imperfectly drained	0%	70	5.44
202	Forest - Moderately well drained	0%	55	10.39
203	Forest - Very poorly drained	0%	77	3.79
204	Forest - Well drained	0%	30	29.63
301	Grass - Imperfectly drained	0%	74	4.46
302	Grass - Moderately well drained	0%	61	8.12
303	Grass - Poorly drained	0%	80	3.18
304	Grass - Very poorly drained	0%	80	3.18
305	Grass - Well drained	0%	39	19.86
401	Highway - Imperfectly drained	80%	93.2	0.93
402	Highway - Moderately well drained	80%	90.6	1.32
403	Highway - Poorly drained	80%	94.4	0.75

404	Highway - Very poorly drained	80%	94.4	0.75
405	Highway - Well drained	80%	86.2	2.03
501	Industrial - Imperfectly drained	90%	95.6	0.58
502	Industrial - Poorly drained	90%	96.2	0.50
503	Industrial - Very poorly drained	90%	96.2	0.50
504	Industrial - Well drained	90%	92.1	1.09
505	Industrial - Moderately well drained	90%	94.3	0.77
601	Residential - Poorly drained	50%	89	1.57
602	Residential - Very poorly drained	50%	89	1.57
701	Residential - Well drained	50%	68.5	5.84
702	Water - Poorly drained	0%	98	0.26
703	Water - Well drained	100%	98	0.26
801	Shrub - Well drained	0%	35	23.59

3.5.2 POST DEVELOPMENT CURVE NUMBER ASSESSMENT

Figure 7 shows the future development zones adopted for the post-development scenario. The scenario represents the proposed site layout, including industrial lots, road corridors, reserves, drainage channels, and associated open water areas associated with the proposed constructed wetlands. Curve numbers have been assigned to each land use category using the adopted soil drainage classifications and impervious area assumptions described above. Industrial lots and road corridors have been modelled using the relevant impervious allowances, while reserves and landscaped areas have been treated as pervious surfaces consistent with their proposed land cover. Wetlands and open water areas have been represented separately to reflect their storage and conveyance functions. Areas outside the development footprint retain the existing land use, soil classification, imperviousness, and curve number assumptions used in the pre-development model.

Where proposed industrial lots contain a drainage corridor, the corridor has been assigned reserve land use and associated imperviousness, while the net remaining lot area has been modelled as 100% impervious.

In accordance with the Mangaheka and Te Rapa ICMP's, due to the presence of erosion in the downstream receiving environments, retention of the site's initial abstraction (IA) value is required across the site post-development. As per the BoD this retention depth has been approximated at 10mm for all impervious areas of the development site. In the post-development TUFLOW model, this 10 mm retention value is assumed to be retained at-source within the post-development lots. The abstraction ratio applied to the post-development lot areas has therefore been selected such that it results in an equivalent 10 mm initial abstraction depth with the maximum ratio being 1 for 100% Impervious surfaces, while other pervious areas retain equivalent pre-development IA value.

Table 4 shows the final CNs considered for the post development scenario areas after applying the relevant % impervious. Figure 7 below contains a map of the proposed post-developed land use.

Table 4 Post-Developed CN values

SOIL ID	LAND USE - DRAINAGE	% IMP	IA RATIO	FINAL CN
51	Proposed Lot 90% Impervious(D)	100%	0.997	96.2
52	Proposed Reserve	0%	0.05	80
53	Proposed Road	80%	0.66	94.4
54	Proposed Wetland	100%	1	98
55	Proposed Lot 100% Impervious	100%	1	98
56	Proposed Lot 90% Impervious(B/C)	90%	0.75	95

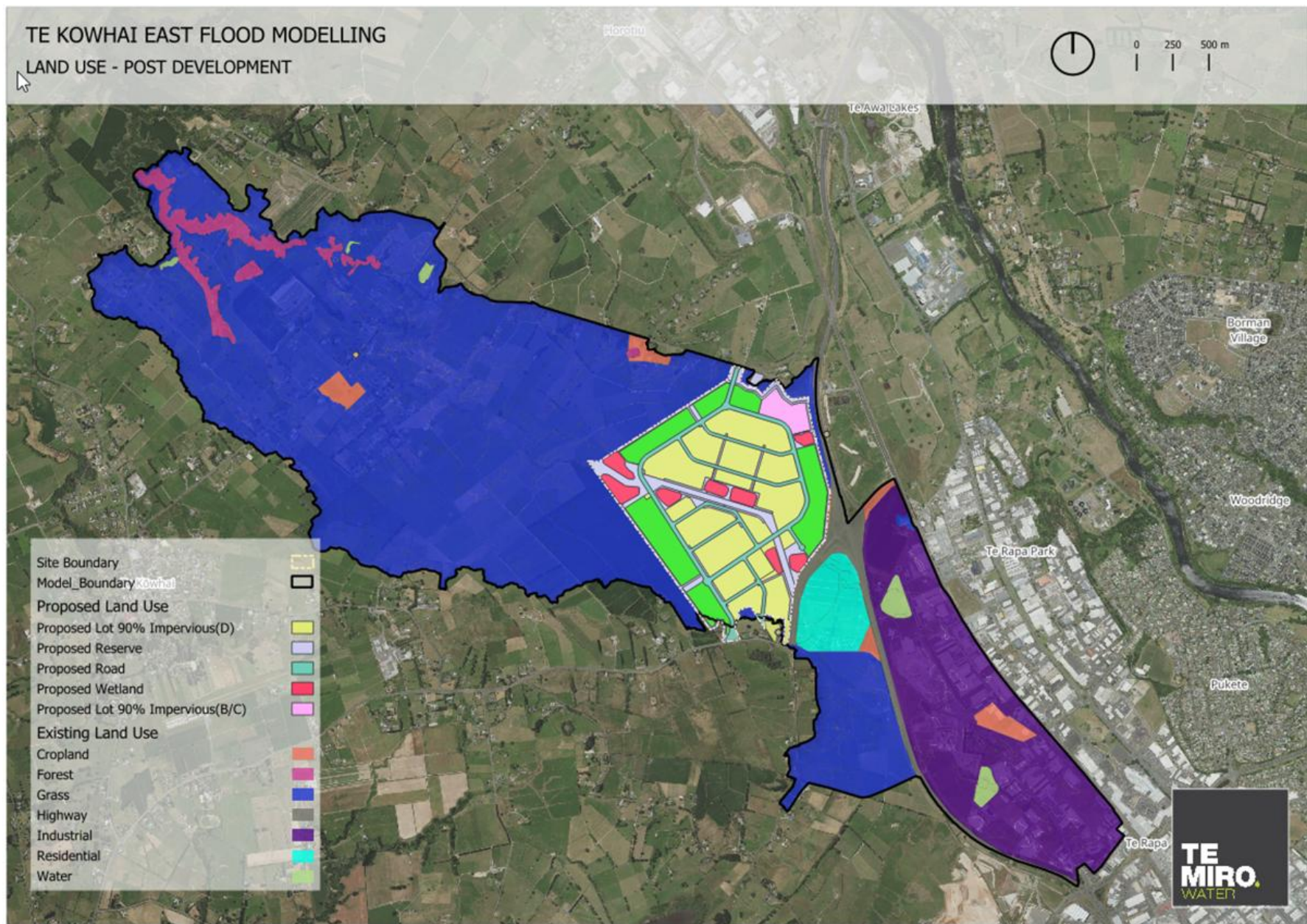


Figure 7 Post Development Land use map of Te Kowhai East

3.6 DESIGN RAINFALL

Rainfall data is sourced from the NIWA HIRDS v4 and given below in Table 5. Checking HIRDS data revealed little variability in the rainfall in the upper and lower catchments of the Mangaheka Stream. Accordingly, a single rainfall value has been used for the whole model extent.

3.6.1 RAINFALL DEPTHS

Historical rainfall depths and intensities for the site were extracted from HIRDS NIWA v4 at Latitude -37.735, Longitude 175.201. The climate change rainfall values for all events have been adjusted for the climate change scenario SSP5-8.5, which predicts a warming of 4°C by the end of the 21st century. The increased values are in accordance with the specifications outlined in TR2025/26 and RITS V2. The historical depths have been used to assess the existing baseline levels of service for the receiving environments. Post-development effects and the design of the proposed stormwater infrastructure for the development adopt the climate change adjusted rainfall depths per Table 5 below.

Table 5: Historical & CC Adjusted Rainfall Depths – 24Hr Storm

STORM EVENT	HISTORICAL DEPTH (MM)	% INCREASE PER °C PER RITS	TOTAL % INCREASE PER SSP5-8.5	CC ADJUSTED RAINFALL DEPTH (MM)
2-Year ARI	63.2	7.2%	28.8%	81.40
10-Year ARI	96.0	8.1%	32.4%	127.10
100-Year ARI	148.0	8.6%	34.4%	198.91

The storm profile was developed based on normalised design storm as per Waikato Stormwater Runoff Modelling Guidelines (TR2020/06, May 2020) using HIRDS v4 rainfall depths. Refer to Figure 8 below for the adopted temporal rainfall pattern from the WRC guidelines.

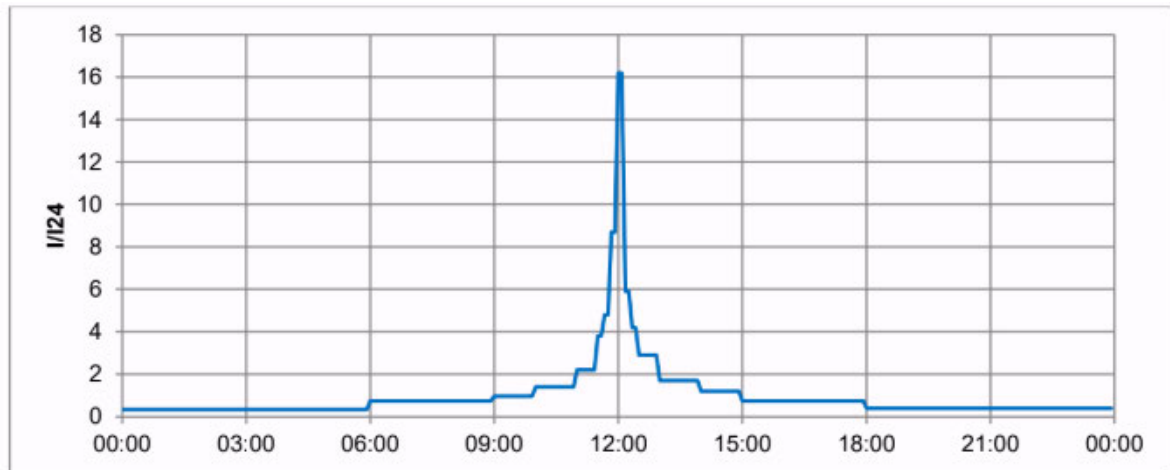


Figure 8 Normalised 24hr nested storm.

3.6.2 RAINFALL-RUNOFF

Rainfall–runoff processes in this model are represented using the Soil Conservation Service (SCS) - Curve Number (CN) methodology, consistent with the guidance provided in Waikato Regional Council Technical Report TR20-06 and the implementation described in the TUFLOW Manual (v2026.0, Section 7.3.5.1.1). The rainfall within the urban catchments was represented as Rain on Grid (ROG) to ensure that overland flow paths and storage within the terrain are captured.

The CN method is based on the concept that a portion of rainfall is abstracted prior to the onset of runoff, representing interception, infiltration, and surface detention. This initial abstraction (I_a) must be satisfied before runoff begins. Thereafter, runoff depth (P_e) is calculated from cumulative rainfall (P) and the potential maximum retention (S) using the standard SCS relationship:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where I_a is typically approximated as a fraction of the storage capacity ($I_a = 0.2S$), and S is related to the curve number, which parameterises the runoff potential of the catchment.

Within the TUFLOW modelling framework, the SCS Curve Number method is implemented as a rainfall loss model in which the entire infiltration and storage behaviour of the catchment is encapsulated within the CN parameter and abstraction ratio. In contrast to physically based infiltration models, soil properties such as porosity, hydraulic conductivity, or soil depth are not defined independently; instead, they are implicitly represented through the assigned curve number values. This reflects the empirical nature of the method and ensures a simplified but robust representation of runoff generation processes.

The TUFLOW implementation computes rainfall excess dynamically at each computational timestep by tracking cumulative rainfall and applying the SCS equations incrementally. Runoff is only generated once the initial abstraction threshold is exceeded, after which continuing losses are governed by the remaining storage capacity (S). The approach allows for a temporally varying runoff response that is directly coupled with the hydrodynamic solution, ensuring consistency between rainfall input, loss processes, and flow routing.

Overall, the use of the SCS Curve Number method within TUFLOW provides a computationally efficient and regionally consistent approach to modelling rainfall losses. By directly integrating this methodology within the hydraulic model, rainfall excess is generated in a manner that is both spatially distributed and temporally responsive, supporting reliable simulation of catchment hydrological behaviour under design event conditions. The process of applying the losses in the rain to grid zones is shown in the following diagram.

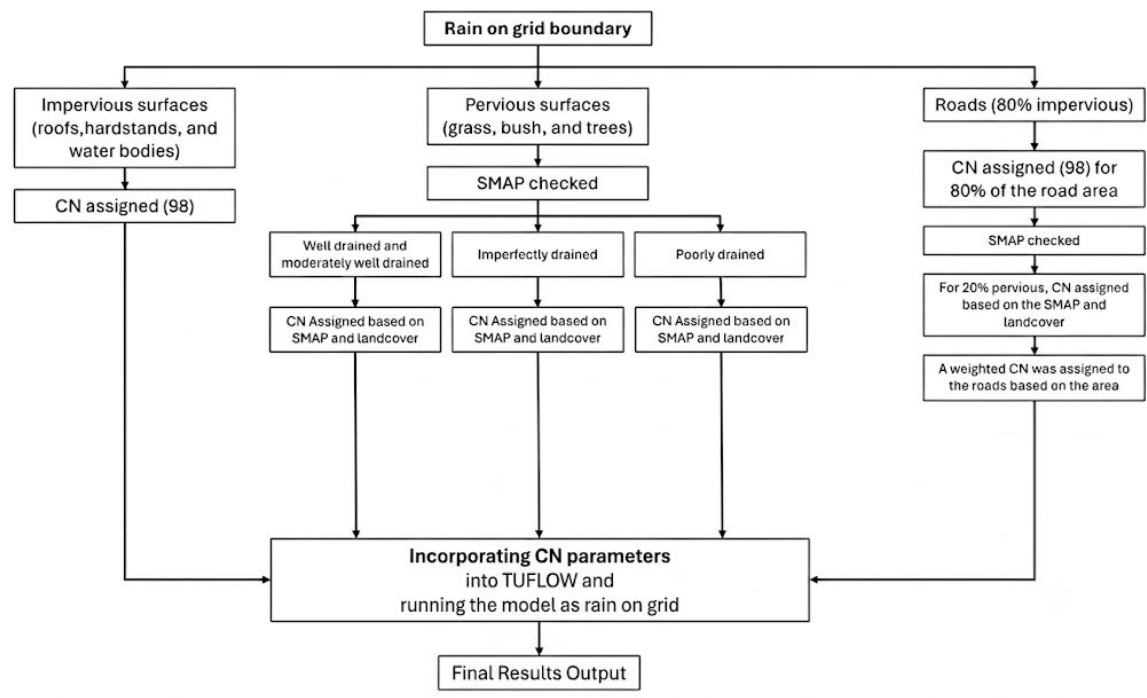


Figure 9. Flow Chart of the process of applying SCS-CN rainfall losses in the TUFLOW model.

4 HYDRAULIC MODEL BUILD

The hydraulic model was developed using the 2D TUFLOW MODEL (version 2025.0.2). The following sections outline the data and assumptions adopted for the hydraulic model.

4.1 LIDAR AND GRID

The 2021 LiDAR is sourced from LINZ and has a resolution of 1m. The 2D TUFLOW model uses a 4m x 4m grid, with the ground level applied within each grid cell as the average of the LIDAR points within that cell. This grid size was chosen because it captures the wider terrain well without creating unnecessarily long runtimes. To better capture the drains and streams within the catchment extents sub-grid sampling was used adopting a quadtree of 1mx 1m cell sizes.

Sub-grid sampling at a 1 m sample distance has been applied to improve the representation of the main central stream channel, culvert approaches, and local hydraulic controls that are smaller than the adopted 4 m model grid. This allows the model to retain the computational efficiency of the coarser grid while using the finer terrain data to define depth-varying storage and conveyance within each cell. The approach improves representation of channel capacity, flow connectivity, and overland flow routing through the drainage network, particularly where LiDAR-derived features would otherwise be smoothed by the coarser model grid. It also provides a better accuracy for reporting velocity within the receiving environment channel. Refer to Figure 10 below for the 1m quadtree areas adopted in the Te Kowhai East Model.

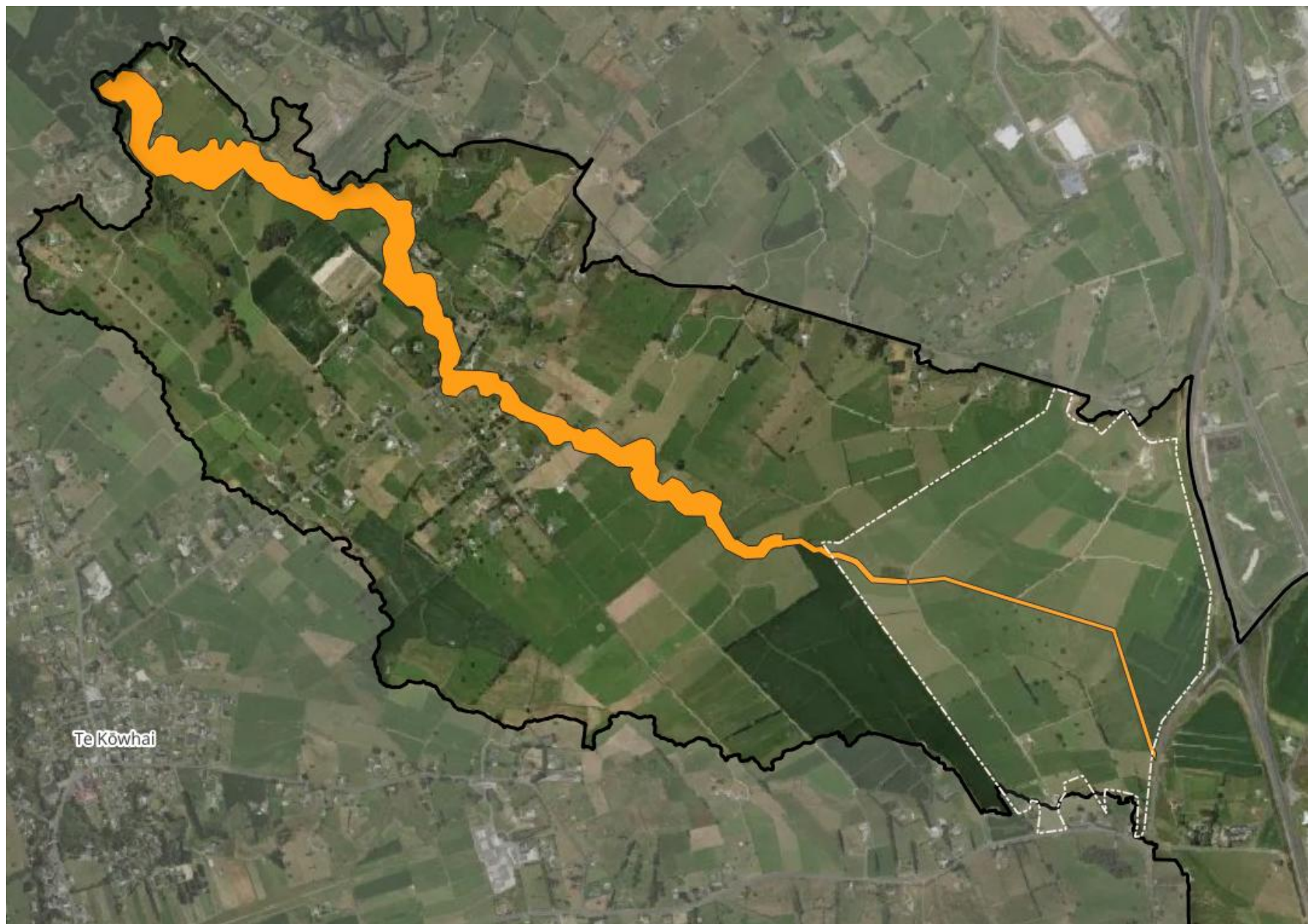


Figure 10 Quadtree Sub-grid Sampling Area (Shown in Orange)

4.2 HYDRAULIC ROUGHNESS

4.2.1 DETERMINATION OF ROUGHNESS VALUES WITHIN THE MODEL

Roughness values have been determined based on the land use coverage from LINZ data as outlined above. The impervious and hardstands layer, buildings, roads, and water bodies were added to the land cover layer and aerial images were used to manually delineate key individual vegetated areas. In some cases, broad assumptions have been made about the land use cover and its associated Manning's number i.e. the existing Te Rapa industrial area was not delineated into buildings, roads, etc as this would add unnecessary complexity to the model and was not considered necessary for the model's purpose.

In addition to the land use coverage, the Manning's number of the main Mangaheka channel and floodplain was refined in places to reflect the dense vegetation, particularly downstream of Horotiu Road. The Manning's numbers used in the model are outlined in Table 6 and were compiled from relevant stormwater and open-channel hydraulic guidance, including TR2020/07 and Chow (1959).

The material mapping has been applied with respect to the comparative purpose of the model, which is to assess changes between the pre- and post-development scenarios rather than to develop a detailed catchment-wide land cover model. Accordingly, greater emphasis has been placed on the representation of Manning's roughness within the site, the main flow paths, and the receiving channel/floodplain areas where changes in hydraulic response are most relevant to the assessment. For comparable vegetation outside these areas, a simplified material mapping approach has been retained and applied consistently between the pre- and post-development scenarios.

Table 6 Where appropriate, depth-varying Manning's numbers have been applied, consistent with the TUFLOW's direct rainfall workflow. Selected material IDs were assigned paired depth and Manning's numbers so that higher roughness is applied during shallow sheet flow, lower roughness is applied at greater depths, and intermediate values are interpolated between the defined thresholds. This avoids applying a single constant roughness value across all flow depths.

The material mapping has been applied with respect to the comparative purpose of the model, which is to assess changes between the pre- and post-development scenarios rather than to develop a detailed catchment-wide land cover model. Accordingly, greater emphasis has been placed on the representation of Manning's roughness within the site, the main flow paths, and the receiving channel/floodplain areas where changes in hydraulic response are most relevant to the assessment. For comparable vegetation outside these areas, a simplified material mapping approach has been retained and applied consistently between the pre- and post-development scenarios.

Table 6. Manning's numbers used in the ROG model.

SURFACE TYPE	MANNINGS NUMBER
Grass - Maintained / Short Pasture (Depth Varying)	0.1 - 0.045
Road Reserve /Paved - Concrete	0.015
Brush / Medium Vegetation (Depth Varying)	0.15 - 0.07
Vegetated Creek / Riparian Zone (Depth Varying)	0.2 - 0.08
Water Bodies / Channels	0.025
Dense Vegetation / Shrubs / Forest Edge (Depth Varying)	0.25 - 0.10
Footpaths / Driveways	0.02
Rock Lining	0.035
Grassed Reserve	0.175 - 0.035
Wetland	0.05

The final Manning's numbers used in the ROG model is presented in Figure 11 below.

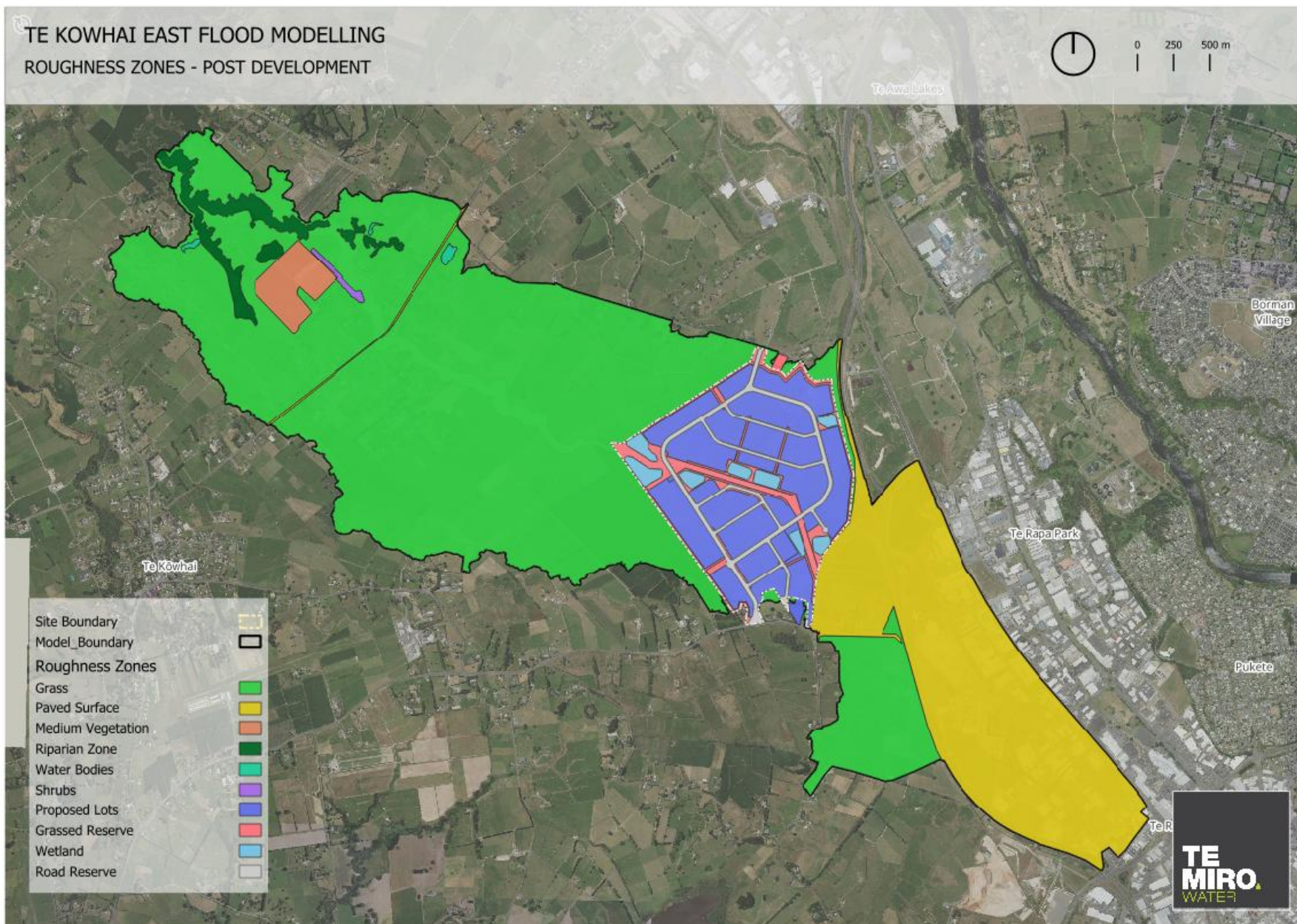


Figure 11 Mannings Roughness Map

4.3 TERRAIN MANIPULATION

Terrain modifications were undertaken where necessary to improve the representation of drainage features and hydraulic connectivity between the 1D and 2D domains. The following adjustments were applied:

- The development area has a significant number of drains which manage surface and groundwater and enable farming operations to occur. Site investigations show that the majority of the drains in the Te Kowhai East area are generally well maintained, typical of a managed farming operation. Accordingly, where drains were not well represented by the LiDAR, z-lines were used to better reflect the drains as 'maintained' drainage channels.
- Small culvert crossings of unknown size associated with farm tracks were replaced with z-lines representing a continuous drainage path.
- At major culvert inlets and outlets, Z lines and polygons were used to locally modify terrain levels at the inlets and outlets of significant culverts to reflect the adopted invert levels. These adjustments remove artificial hydraulic controls at structure interfaces and ensure consistency between the 1D culvert representation and the surrounding 2D terrain.
- No reticulation has been included in existing developed catchments (i.e. Te Rapa Industrial area). Z-lines have been used here to ensure excess surface ponding is directed to the existing wetlands.
- Z-lines were also applied where required to remove artificial blockages in the LiDAR surface associated with vegetation and to burn in farm crossings where asset or survey information was not available.
- Any large areas of residual surface ponding on the pre-developed site have been analysed and where appropriate z-lines have been used to create artificial channel links to the nearest drain. This replicates a typical farming situation where residual ponding would not be tolerated and reflects anecdotal evidence that the area is 'well-drained'.
- Similarly for the post-development case, the model does not include reticulation within the development and where residual ponding occurs within the surface low-points (road sags etc) z-lines have been used to create artificial links to the nearest drain or wetland. This more accurately reflects a reticulated developed situation where large areas of residual ponding would not occur.
- Lowering of areas immediately upstream and downstream of culverts was undertaken to help stabilise flows and remove any artificial blockages or impediments created by the LiDAR data.

Terrain modifications and SX connections were refined through an iterative calibration process to achieve stable hydraulic behaviour through the major culverts while preserving the underlying LiDAR surface as far as practicable. In particular, the localised lowering of terrain around the primary highway culverts was intentionally incorporated to facilitate stable flow transfer across the 1D/2D interface and to ensure consistent hydraulic performance between the pre- and post-development scenarios. The effect of refined SX lines and

increased SX storage can be seen in Figure 12 which shows the flows from the State highway culvert 503 before and after the application of terrain modification.

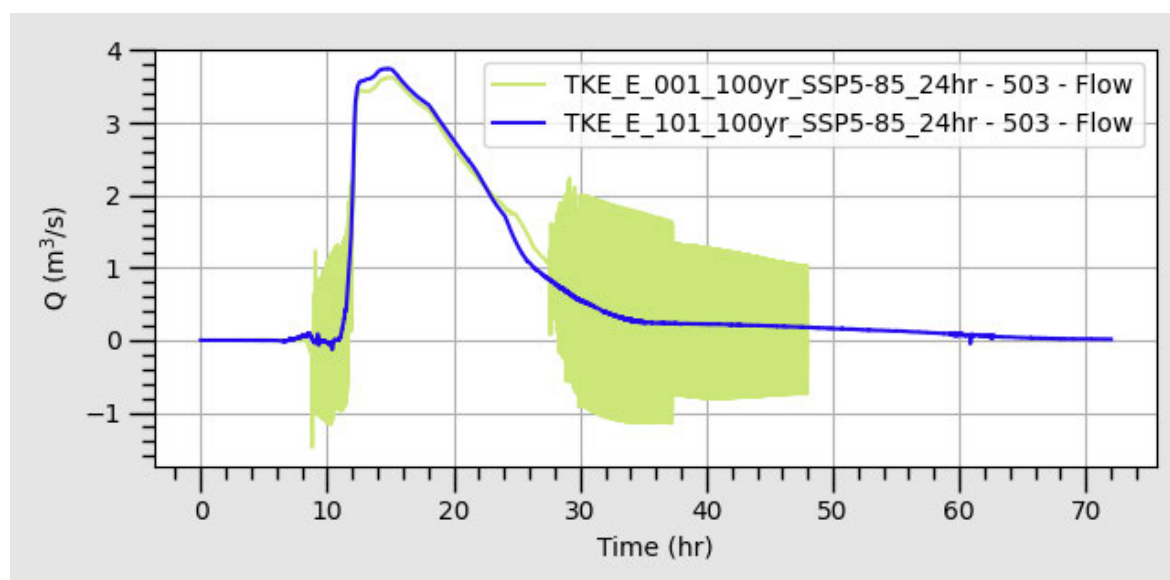


Figure 12; Effect of Terrain Modification at Culvert Inlets within the Model

Accordingly, larger local terrain adjustments are confined to specific culvert and drainage locations where they were necessary to represent hydraulic connectivity and structure performance.

4.4 HYDRAULIC STRUCTURES

4.4.1 CULVERTS AND BRIDGES

External to the proposed development area the following hydraulic structures have been included in the model as 1D elements.

- Two 1800mm dia. culverts associated with two private driveway crossings at 55 Hartstone Road (surveyed)
- Two 900mm dia. Culverts and twin 1200mm culverts underneath Horotiu Road (surveyed)
- Three private culverts (size unknown) associated with private driveway crossings downstream of Horotiu Road (not surveyed). These have been burned in the channel using z lines.
- Crawford Road Bridge (surveyed).
- Waikato Expressway (SH1C) culvert
- Koura Drive (SH39) Culverts x 2

Within the development area, the following hydraulic structures have been included in the post-developed model. These hydraulic structures have been provided by Maven.

- Culverts linking the various channels to the wetlands

- Two box culverts and bridges associated with the two road crossings proposed over Mangaheka channel
- Constructed wetland outlet structures

4.4.2 CHANNELS AND WATERWAYS

Open channels within the rain-on-grid model boundary, including the Mangaheka Stream and associated drainage channels, have been represented directly within the 2D domain. Channel geometry has been defined from the adopted terrain surface (LIDAR) and refined where required to maintain hydraulic connectivity through the drainage network. The Crawford Road Bridge has been included as a surveyed hydraulic control, with the bridge opening and adjacent road low point represented in the model as 2D layered structure to capture conveyance through the structure and overtopping of Crawford Road during larger events.

4.4.3 STORMWATER DEVICES

4.4.3.1 *Existing wetland basins in Te Rapa*

There are two existing wetlands located in the Te Rapa industrial area- the Porters wetland¹ located off Aurthur Porter Drive and the Hamilton Joint-Venture wetland² located off Hounsell Drive. There are several smaller ponds, swales and conveyance channels within the Te Rapa area which are linked to these two main wetlands.

The flood storage volume within the wetlands, basins and channels area well represented in the LiDAR surface and the ROG model directs surface water to these devices. Culvert outlets as per the asset data have been included in the model to discharge flows from the various stormwater devices to the downstream drains. The attenuation function of these devices is therefore not well represented; however, as the same approach has been applied in both the pre- and post-development scenarios, the approach is considered acceptable.

Where excess ponding remains on the surface in this built-up area, z-lines have been included to ensure as much surface water as practicable drains to the wetland devices. A sensitivity check has been undertaken to assess the effects of draining remaining residual ponding areas by increasing runoff from this area. This is summarised in Section 7 below.

¹ 119 Ruffell Road, Burbush, Hamilton (Lot 1 DP 607754)

² 52 Hounsell Road, Burbush, Hamilton (Lot 110 DP 570593)

4.4.3.2 Development Wetlands

The proposed development consists of eight (8) constructed wetlands which have been designed to provide flood storage and attenuation function above the extended detention water levels for their respective sub-catchments. Each wetland comprises an outlet structure comprising several orifices and broad crested weirs to attenuate influent flows. Refer Table 7 below for a list for the modelled wetland outlets included as 1D elements in the model. Each wetland has an emergency spillway for larger than design events, or culvert blockage. These spillways were included as 1D elements in the model for the culvert block hazard assessment only.

The wetland live storage volumes are included in the proposed ground surface and runoff from the ROG model drains directly into the basins via the drainage channels and roads located throughout the site. Where excess ponding is apparent z-lines have been included to ensure the majority of surface water is conveyed to the various devices.

Preliminary design of the various wetlands has been undertaken by Maven Ltd and has been tested in the hydraulic model. The attenuation devices within the development have been designed to attenuate site-specific post-development peak flows (incl. climate change adjusted rainfall) to meet site-specific pre-development peak flows (historic rainfall) as per the BoD and TR2020/07 for the following events (and targets);

- Extended Detention volume
- 80% of 2 -year ARI event
- 100% of 10 and 100-year ARI events.

Te Miro Water has adopted the proposed wetland hydraulic structures as per the Maven design. Final outlet configurations are presented in Table 8 below. Refer results section below for confirmation that the above attenuation targets are improved on in all storm events.

Further wetland design summary is documented in the Maven Ltd Stormwater Management Plan contained in the substantive application documentation.

Table 7 Development wetland outlet structures

WETLAND	ORIFICE (CIRCULAR)	WEIR 1	WEIR 2	EMERGENCY SPILLWAY
Wetland 1	5 x 180mm, IL23.95m	0.8m Long, IL24.85m	1.5m Long, IL25.10m	30m Long, IL25.7m
Wetland 2	5 x 150mm, IL24.90m	0.5m Long, IL25.55m	0.2m Long, IL25.85m	15m Long, IL26.65m
Wetland 3	5 x 180mm, IL26.70m	1.0m Long, IL27.40m	0.5m Long, IL27.65m	15m Long, IL28.2m
Wetland 4	5 x 180mm, IL24.00m	0.8m Long, IL24.90m	2.0m Long, IL25.15m	35m Long, IL26m
Wetland 5	4 x 150mm, IL25.60m	0.5m Long, IL26.55m	0.5m Long, IL26.75m	15m Long, IL27.3m
Wetland 6	5 x 180mm, IL25.90m	0.8m Long, IL26.60m	1.5m Long, IL27.00m	25m Long, IL27.45m
Wetland 7	5 x 130mm, IL28.50m	0.5m Long, IL28.90m	1.0m Long, IL29.20m	15m Long, IL29.6m
Wetland 8	5 x 100mm, IL26.80m	0.5m Long, IL27.60m	-	15m Long, IL28.35m

4.5 BOUNDARY CONDITIONS

4.5.1 HYDROLOGIC INPUTS

An inflow hydrograph representing an external sub-catchment discharging into the TKE model area has been applied as a boundary condition in the 100-year post-development model. The inflows are associated with the Rotokauri Greenway, which overtops via a spillway during the 100-year RCP8.5 scenario. Spillway flows discharge across undeveloped land, pass beneath Te Kowhai Road via an existing culvert, continue beneath Koura Drive, and ultimately enter the main Mangaheka channel. To align with the SSP5-8.5 climate change scenario adopted for the TKE model, the spillway hydrograph has been increased by 20% in the hydraulic model

to account for the additional runoff expected from the contributing sub-catchment in this climate change scenario.

The inflow hydrograph is represented in Figure 13 below.

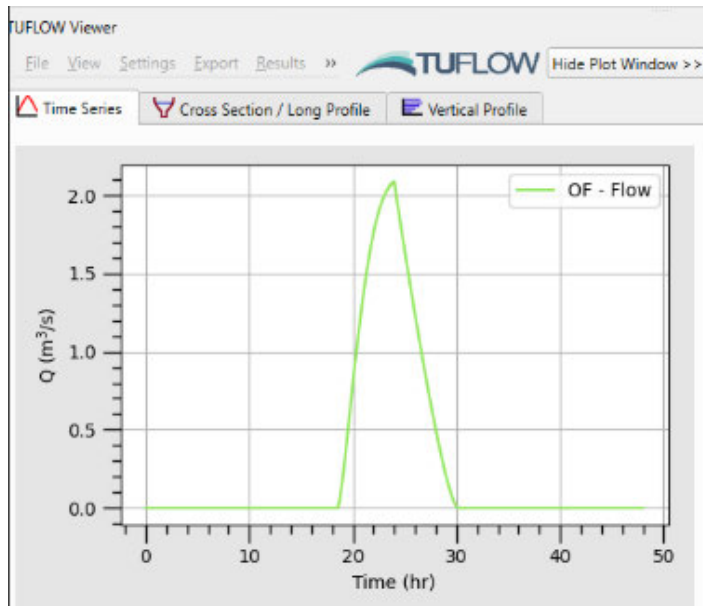


Figure 13 Rotokauri Greenway Spillway Hydrograph Applied as Boundary Condition in TKE Model

4.5.2 DOWNSTREAM BOUNDARY

The downstream boundary of the model is approximately 200m west of the Crawford Road Bridge, sufficient distance for any tailwater effects on the bridge to be represented in the model. The boundary has been represented as a normal boundary with a 0.5% slope representative of the existing channel slope in this area.

4.6 ASSESSMENT SCENARIOS

4.6.1 Pre-development Scenario

The pre-development or existing scenario assessment includes two scenarios to reflect the seasonal variation in groundwater levels found across the site. The geotechnical and hydrogeological investigations confirmed seasonal ground water fluctuations of up to 2m between the winter and summer months. Accordingly, as per section 3.2.2 above and in accordance with TR2020/06 a dual soil classification has been adopted, and a corresponding 'pre-development scenario' has been run for each groundwater condition as a baseline for comparison with the design case. The pre-development scenarios include climate change adjusted rainfall

The two scenarios have been defined as per below including model ID.

- Summer or Low groundwater scenario [E_101]
- Winter or High Ground water scenario [E_103]

As per Section 3.2.1 above, S-maps have classified a significant portion of the site as Soil Type D due to the elevated groundwater and this has been retained during the summer / low groundwater scenario, despite soil investigations showing the ground water does drop to around 2m deep. As a result, the high groundwater / winter scenario does not result in significant increases in peak flows or runoff volumes. Accordingly, the low-groundwater or summer scenario [E_101] has been adopted as the baseline "pre-development" scenario for comparisons with post-development model results. Comparisons with the winter or high groundwater scenario have been included for reference where appropriate to understand seasonal differences to the downstream environment.

4.6.2 Existing / Historical Scenario

An existing model scenario has been developed to determine existing or historical runoff from the subject site only i.e. excludes external inflows. This scenario uses historical rainfall without any adjustment for climate change and has been used to measure the effectiveness of the attenuation devices within the site as per BoD and TR2020/07.

4.6.3 Post Development Scenario

Various post-development scenarios have been assessed to measure the effectiveness of the proposed stormwater management measures within the development.

Additional post-development scenarios have been simulated in the model to predict the effects of various stormwater mitigation options considered within the development and these are summarised in Table 8 below.

Table 8 Post-Development Model Scenarios and ID

SCENARIO AND ID		MITIGATION BASIS	DESCRIPTION
Desing P_303	Case	Minimum retention requirement	Represents the minimum 10 mm or equivalent Initial Abstraction depth retention achieved across the development.
Sensitivity P_305	Check	Partial soakage / re-use	Represents distributed soakage or re-use provided where practicable across the development.
Sensitivity P_306	Check	2-year ARI volume neutrality	Represents full 2-year ARI volume neutrality, with post-development runoff retained or re-used at source to match the pre-development summer runoff volume.

4.6.4 MODEL REPORTING POINTS

The reporting points within the hydraulic model extents are described in Table 9 below.

Table 9. Hydraulic Model Reporting Points

REPORTING POINT ID	LOCATION	REPORTING POINT ID	LOCATION
A	Main Mangaheka Channel Immediately upstream of Horotiu Road	G	Immediately downstream of Horotiu Road
B	Side branch of WRC managed drainage scheme	H	Main Mangaheka Channel downstream of Horotiu Road
C	Side branch of WRC managed drainage scheme	I	Main Mangaheka Channel downstream of Horotiu Road
D	On main Mangaheka Stream immediately downstream of site.	J	Main Mangaheka Channel downstream of Horotiu Road
E	Upstream of Koura Drive	L	(Crawford Road Bridge)
F	Between Point D and A on main channel	M	Main Mangaheka Channel Downstream model boundary

The Te Rapa Stream reporting point is located at the upstream end of the Waikato Expressway culvert. Results are not presented for every reporting point; only locations relevant to the assessment outputs have been included. The reporting point locations are shown in Figure 14 below. Additional reporting point information is available on request.

TE KOWHAI EAST FLOOD MODELLING

MODELLING EXTENT

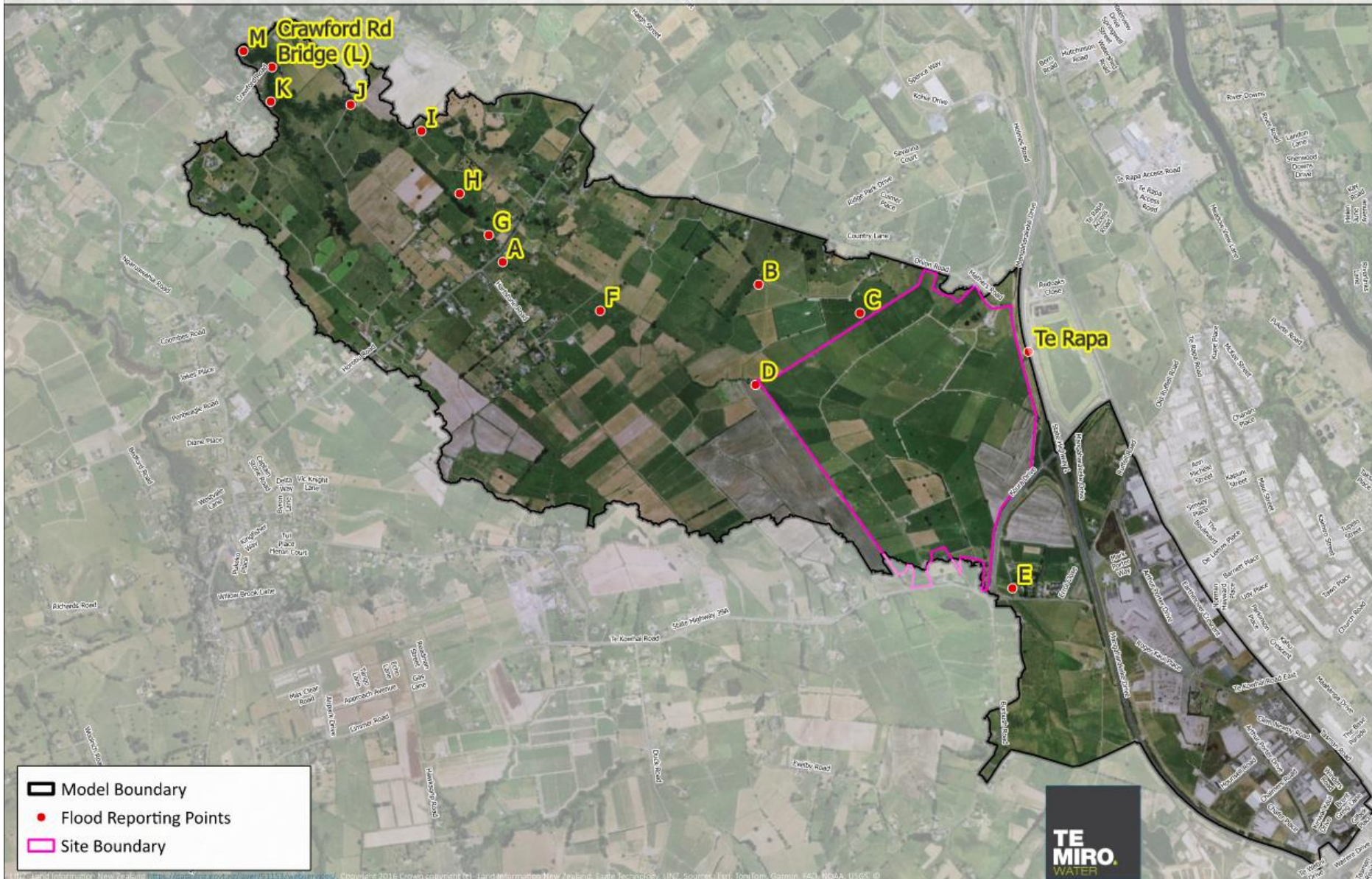


Figure 14 Model Boundary and Reporting Points

5 RESULTS

The following sections present the hydraulic model results.

- 2-year ARI Event: Peak flow, Volume and Velocity
- 10-year ARI Event: Peak Flow and drain down/ pasture clearance times
- 100-year ARI Event: Peak flow and Water Level

5.1 PRE-DEVELOPMENT

5.1.1 EXISTING / HISTORICAL RESULTS

Existing or historical peak flow results generated from the site area only are presented in Table 10.

Table 10 Historical Rainfall Peak flows -Site Extent only

ARI EVENT	PEAK FLOW (M ³ /S)	
	MANGAHEKA STREAM CATCHMENT (MEASURED AT POINT D)	TE RAPA STREAM CATCHMENT
2 -Year	2.30	0.35
10-Year	4.93	0.79
100-Year	9.93	1.37

5.1.2 PRE-DEVELOPMENT SCENARIO

Pre-development peak flow, velocity and water level results measured at various locations downstream of the site are presented in Table 11, Table 12 and Table 13.

Table 11 Pre-Development 2-year ARI Peak Flow and Velocity

CATCHMENT / LOCATION	PEAK FLOW [E_101], Q (M ³ /S)	EXISTING VELOCITY, V (M/S)	WINTER [E_103] PEAK FLOW, Q (M ³ /S)
Mangaheka - Point D	3.65	0.57	3.94
Mangaheka - Point A	5.08	0.23	5.29
Mangaheka - Point I	5.66	0.28	5.84
Mangaheka – Crawford Road Bridge	6.81	1.47	7.00

Mangaheka – Point M	6.83	0.46	7.01
Te Rapa Stream - WEX culvert	0.36	1.28	0.60

Table 12 Existing 10-year ARI Peak Flow results

CATCHMENT / LOCATION	PEAK FLOW [E_101], Q (M ³ /S)	WINTER [E_103] PEAK FLOW, Q (M ³ /S)
Mangaheka - Point D	7.24	8.06
Mangaheka - Point A	9.22	9.62
Mangaheka - Point I	10.64	10.88
Mangaheka – Crawford Road Bridge	14.11	14.22
Mangaheka – Point M	14.16	14.27
Te Rapa Stream - WEX culvert	0.88	1.18

Table 13 Existing 100-year ARI Peak Flow and Water Level results

CATCHMENT / LOCATION	PEAK FLOW [E_101], Q (M ³ /S)	EXISTING WATER LEVEL, RL (M)	WINTER [E103] PEAK FLOW, Q (M ³ /S)
Mangaheka - Point D	15.86	24.77	17.48
Mangaheka - Point A	16.21	23.34	18.25
Mangaheka - Point I	18.42	17.31	20.70
Mangaheka – Point L (Crawford Road Bridge)	26.07	15.01	26.17
Mangaheka – Point M	26.20	14.18	26.30
Te Rapa Stream - WEX culvert	1.41	N/A	1.56

Refer to Figure 15, Figure 16, Figure 17 and Figure 18 for model results maps showing water levels and velocities in the pre-development scenario across the model extent.

1. Maximum Water Levels – 2, 10 and 100-year ARI Events (Figure 15, Figure 16, Figure 17)
2. Velocity – 2-year ARI Event (Figure 18).

Zoomed maps are located in Appendix A.

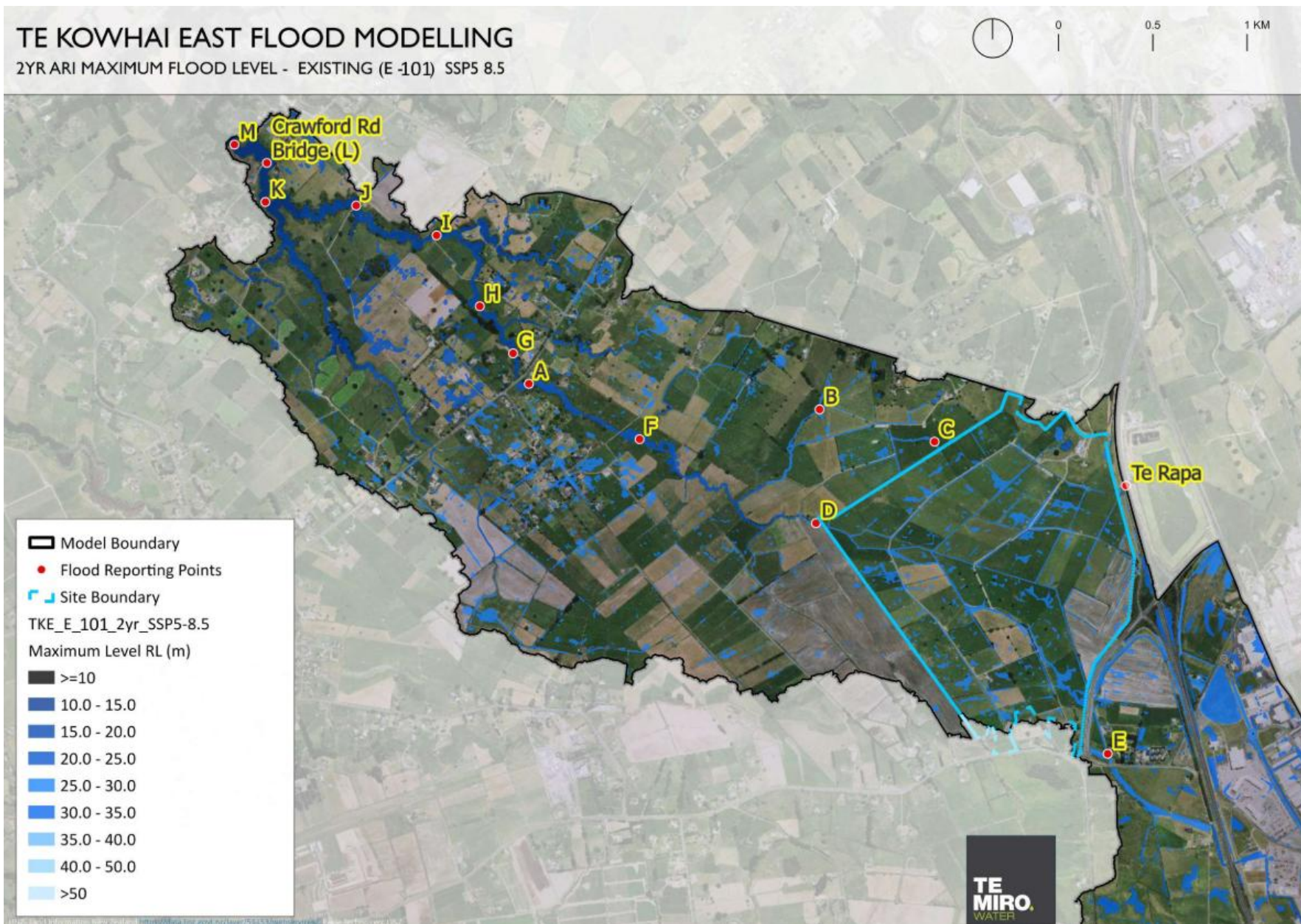


Figure 15 Pre-Development (with CC) 2-year ARI Flood Level

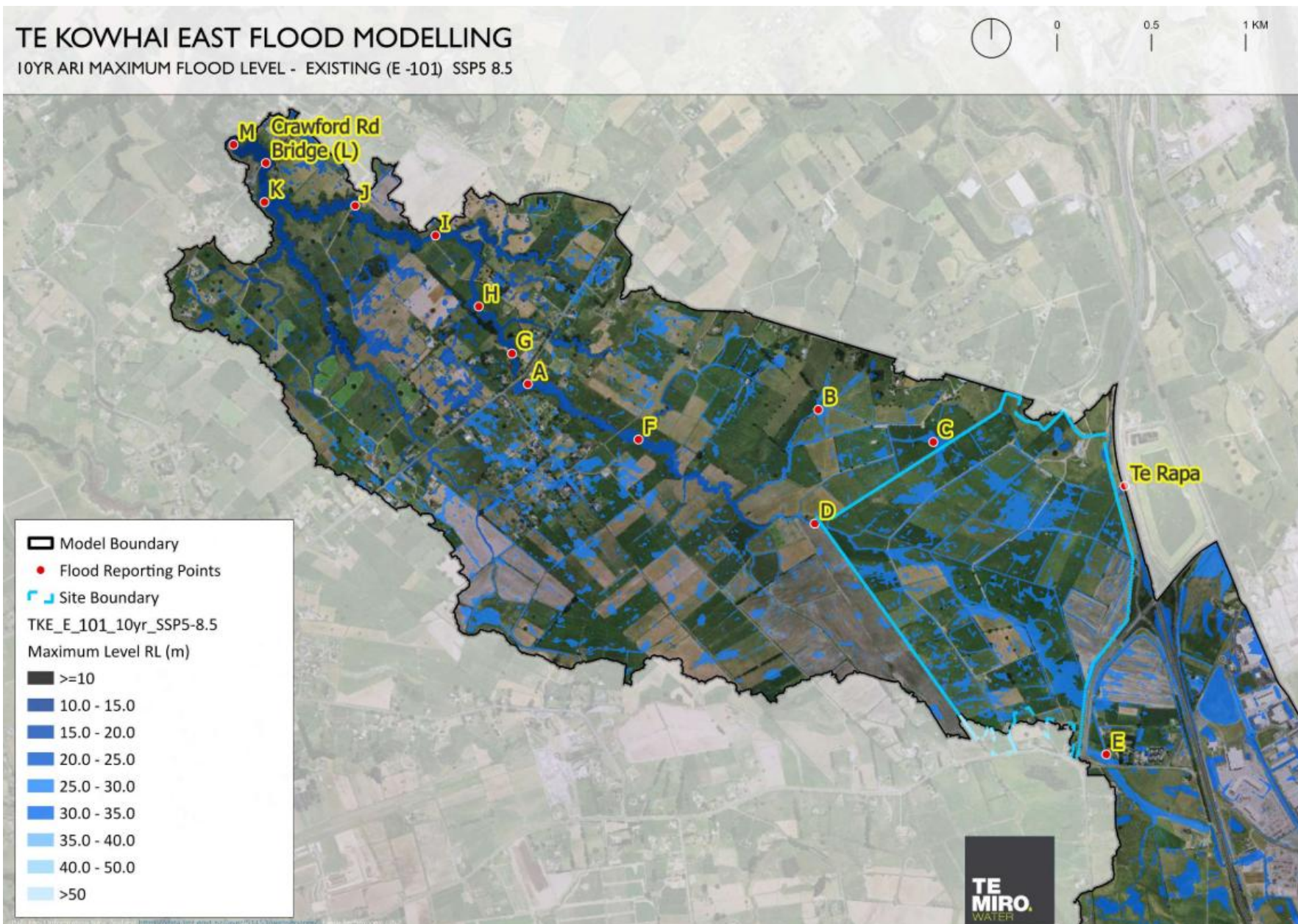


Figure 16 Pre-Development (with CC) 10-year ARI Flood Level

TE KOWHAI EAST FLOOD MODELLING

100YR ARI MAXIMUM FLOOD LEVEL - EXISTING (E -101) SSP5 8.5

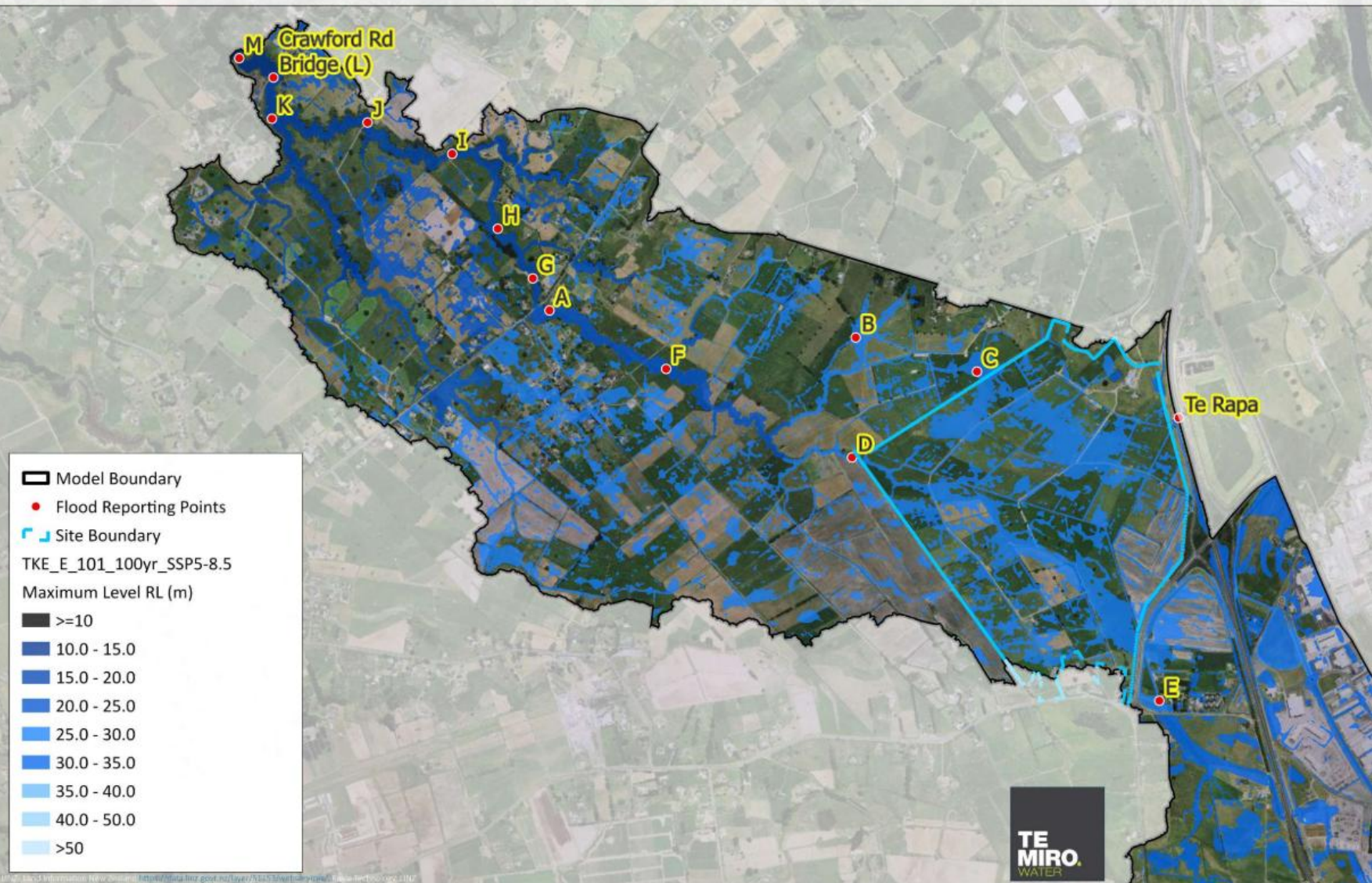


Figure 17 Pre-Development (with CC) 100-year ARI Flood Level

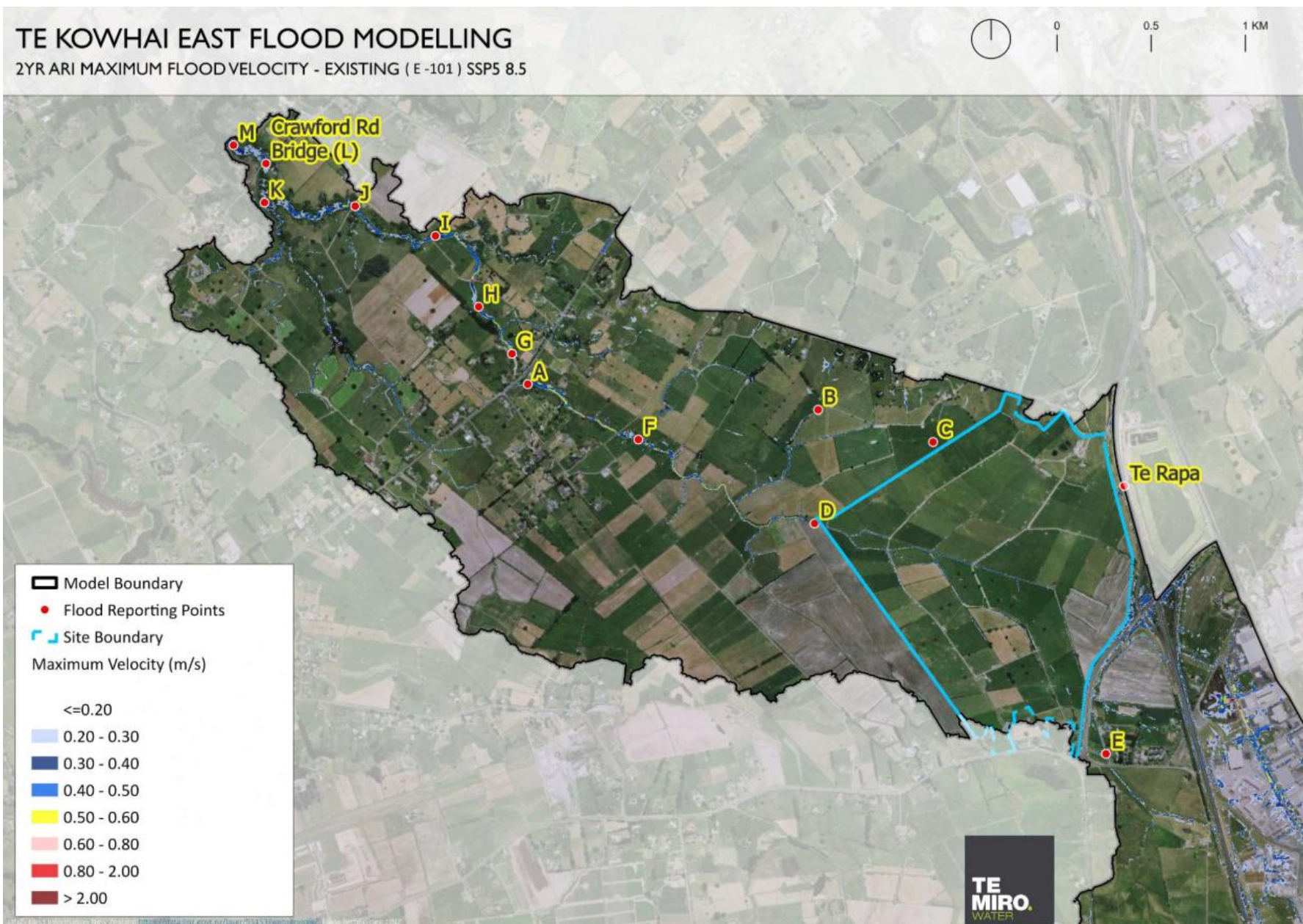


Figure 18 Pre-Development 2-Year ARI (with CC) Velocity

5.2 POST DEVELOPMENT RESULTS

Post development results from the site sub-catchment only (excludes any upstream flows) are presented in Table 14 below.

Table 14 Post-development Peak Flows (Site Only)

ARI EVENT	PEAK FLOW (M ³ /S)	
	MANGAHEKA STREAM CATCHMENT (MEASURED AT POINT D)	TE RAPA STREAM CATCHMENT
2 -Year	1.51	0.25
10-Year	3.39	0.55
100-Year	9.34	1.10

Table 15, Table 16, Table 17 present the post-developed P_303 results at various reporting locations downstream of the site. These flows present total catchment flows.

Table 15 Post developed 2-year ARI Peak Flows and velocity [P_303]

ASSESSMENT LOCATION	PEAK FLOW, Q (M ³ /S)	VELOCITY, V (M/S)
Mangaheka - Point D	2.89	0.54
Mangaheka - Point A	3.68	0.22
Mangaheka - Point I	4.08	0.27
Mangaheka - Point L (Crawford Road Bridge)	5.16	1.47*
Mangaheka - Point M	5.17	0.40
Te Rapa Stream - WEX culvert	0.36	1.28**

* Peak velocities across whole crossing- see below for further explanation

** Peak flows measured within the culvert

Table 16 Post developed 10-year ARI Peak Flows [P_303]:

ASSESSMENT LOCATION	PEAK FLOW, Q (M ³ /S)
Mangaheka - Point D:	6.95
Mangaheka - Point A:	8.45
Mangaheka - Point I:	9.66
Mangaheka - Point L (Crawford Road Bridge):	12.90
Mangaheka - Point M:	12.95
Te Rapa Stream - WEX culvert:	0.75

Table 17 Post developed 100-year ARI Peak Flows and Max water level [P_303]

LOCATION	PEAK FLOW, Q (M ³ /S)	MAXIMUM WATER LEVEL RL, (M)
Mangaheka – Point D	13.90	24.69
Mangaheka – Point A	13.80	23.24
Mangaheka – Point I	15.88	17.23
Mangaheka – Point L (Crawford Road Bridge)	25.13	15.00
Mangaheka – Point M	25.26	14.16
Te Rapa Stream – WEX Culvert	1.39	N/A

5.3 RESULTS DISCUSSION

5.3.1 Peak Flow

- Post development peak flows from the site are less than the existing/historical case in all events in both the Mangaheka and Te Rapa discharge locations. Post development peak flows to the Mangaheka channel are 66%, 69% and 94% of existing/historical flows in the 2, 10 and 100-year ARI events respectively. Post development peak flows to the Te Rapa are 73%, 68% and 80% of existing/ historical peak flows in the 2, 10 and 100-year ARI events respectively. This confirms the BoD targets are met and improved on in all cases.
- At all reporting points on the Mangaheka Stream the post-development peak flows are below the pre-development peak flows under low-ground water/summer conditions for all events (and less runoff compared to winter conditions). Over-attenuation of post development flows has been achieved with peak flows equal to 80%, 95% and 88% of pre-development flows (with climate change) in the 2, 10 and 100-year ARI events respectively.
- Post development flows to the Te Rapa Stream are attenuated to meet 100%, 85% and 100% of pre-development flows in the 2, 10 and 100-year ARI events respectively.

5.3.2 Velocity

- Post-development flow velocities are generally maintained at levels comparable to pre-development conditions in the 2-year ARI event, in all reporting locations.
- Pre- and post-development velocities are all generally lower than the limiting velocity of 0.61m/s for the adopted silty loam (noncolloidal) soil type as per TR2020/07 Table 7-1 at the reporting locations. The model results indicate there are discrete sections of the channel which have elevated velocities in the pre- and post-development conditions, which is consistent with anecdotal erosion visible within the existing channel.
- The exception is the Crawford Road Bridge location, where modelled velocities exceed the limiting value in both the pre- and post-development scenarios (1.27 m/s) with no change post-development. Analysis of the model outputs shows that this reported value is an aggregated velocity of flows over and under the bridge associated with the 2D structure modelled in TUFLOW. To ascertain actual velocities in the channel beneath the bridge a HY-8 model was set up, and this is discussed below.
- Analysis of results immediately upstream and downstream of the Crawford Road Bridge show the velocities are low, typical of elsewhere along the stream channel, hence the elevated velocities are an isolated occurrence at the bridge.

5.3.3 Water Level

- Peak water levels along the main channel in the post-development scenario are generally less than pre-development water levels in all events i.e. there is no afflux at the private driveway crossings indicating a reduced level of service. Water levels however remain elevated for extended periods due to the increased runoff volume in the larger storm events.
- Crawford Road Bridge: The bridge and adjacent road low point overtops in all existing and post-developed model scenarios. Post-development water levels are lower than pre-development levels in all storm events. There is a 40mm decrease in 10yr, 20mm decrease in the 100yr and 50mm decrease in the 2yr ARI Event peak levels.
- To the west of the site at 169 Onion Road the proposed development results in a significant decrease in 100-year ARI event flooding on farmland due to the proposed perimeter drain constructed on the western boundary of the site. This drain diverts a significant catchment from within the site away from the existing drain on the adjacent property.
- The proposed widening and deepening of the central Mangaheka Stream channel through the site, as far as Point D, results in a decrease in tailwater and associated restrictions on the WEX culverts under Koura Drive (SH39). Accordingly, post development flows at the culvert outlets are elevated in all events with a corresponding reduction in water levels upstream of these culverts. Culverts are proposed within the modified portion of the stream within the site to provide a level of online attenuation. The culverts and additional storage capacity provided by the widened and deepened channel assists to mitigate peak flows from the upstream catchment back to pre-development levels downstream of the site for the 2-Year, 10-Year and 100-Year ARI storm events.

Refer to Figure 19 - Figure 25 below for post development result maps of the water levels, velocities and pre-and post-development comparisons. Further zoomed in plots are in Appendix A.

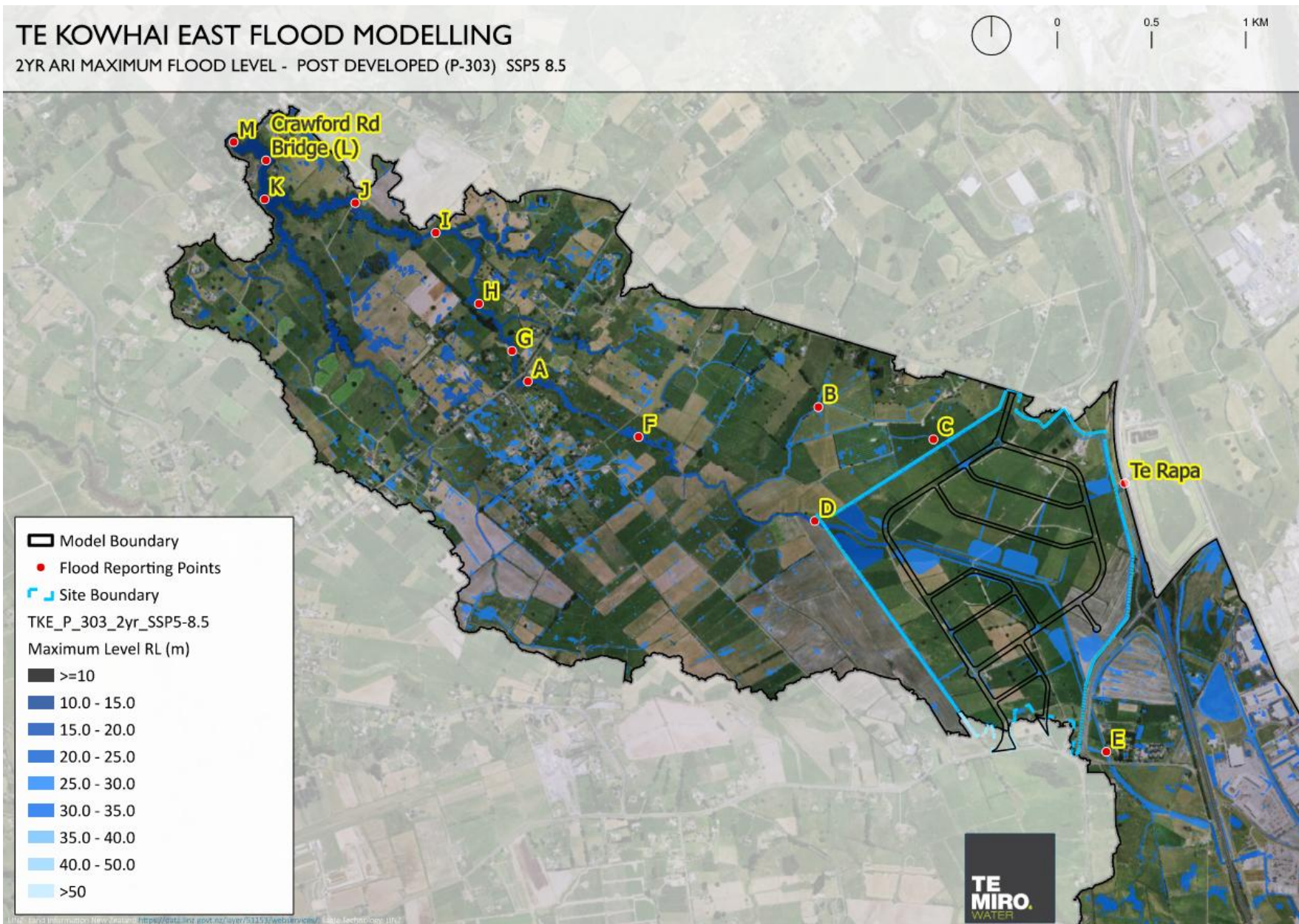


Figure 19 Maximum Water Level Post Development- 2-year ARI Event

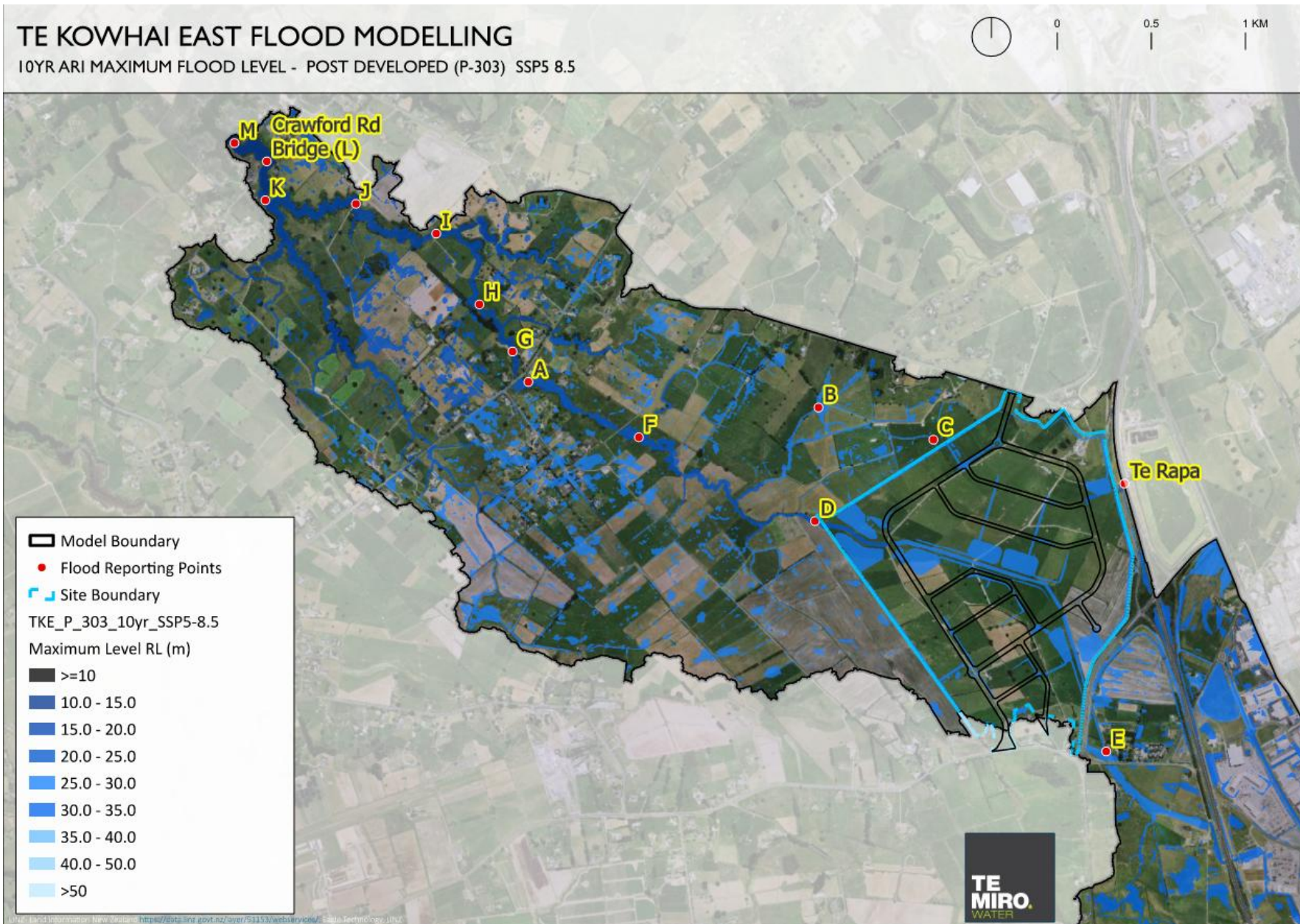


Figure 20 Maximum Water Level Post Development- 10-year ARI Event

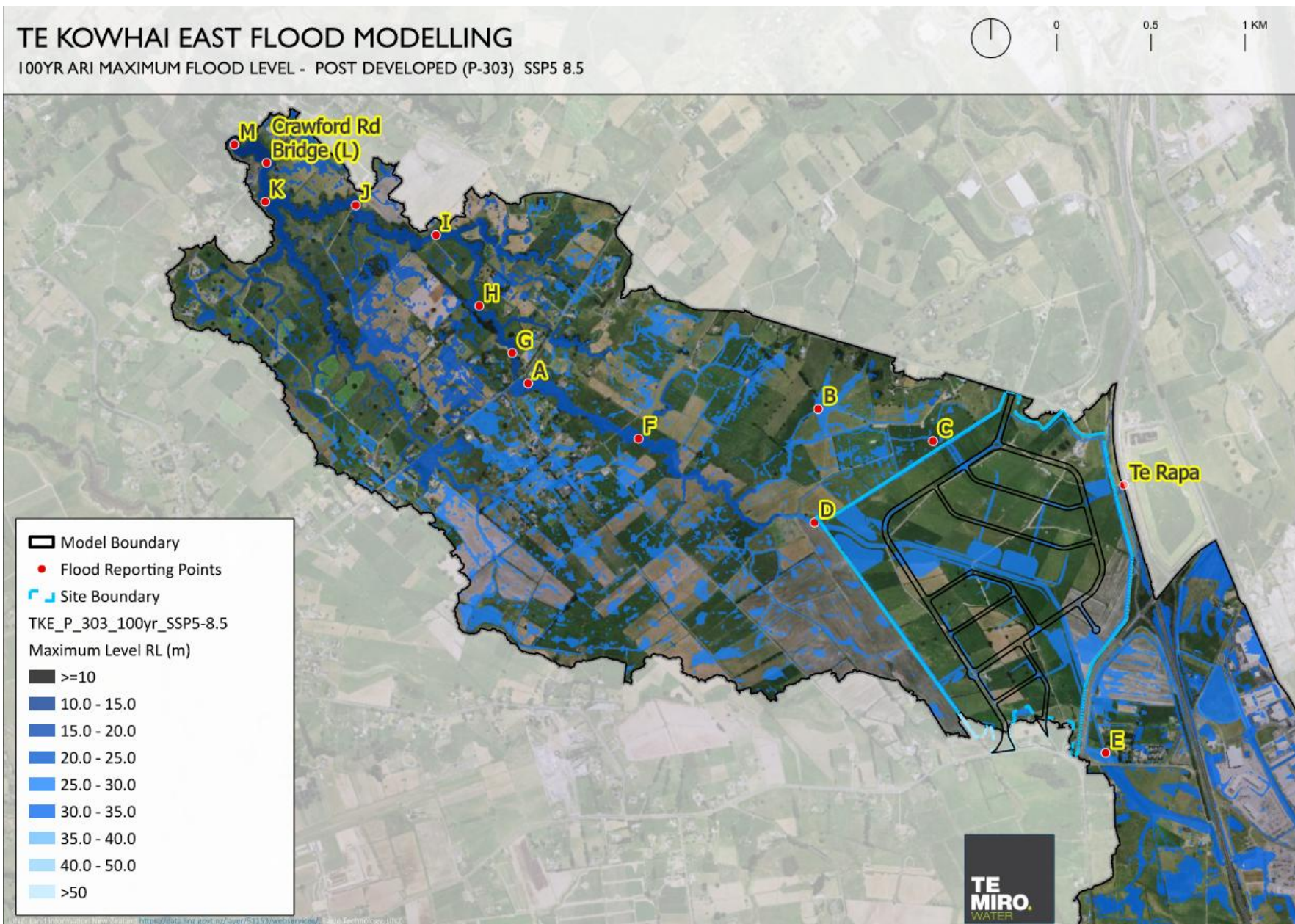


Figure 21 Maximum Water Level Post Development- 100-year ARI Event

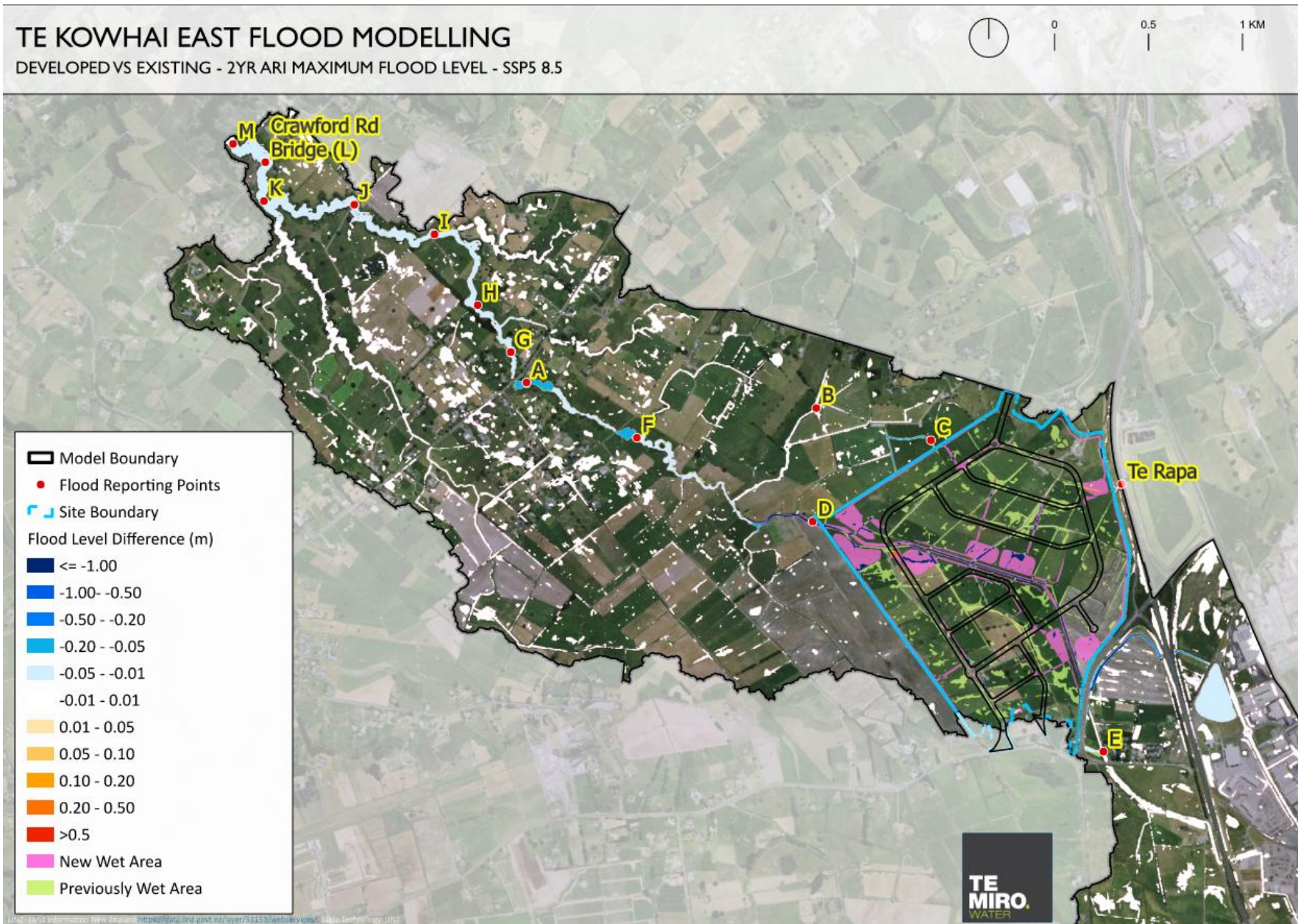


Figure 22 Maximum Water Level Comparison Map- 2-year ARI Event

TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5

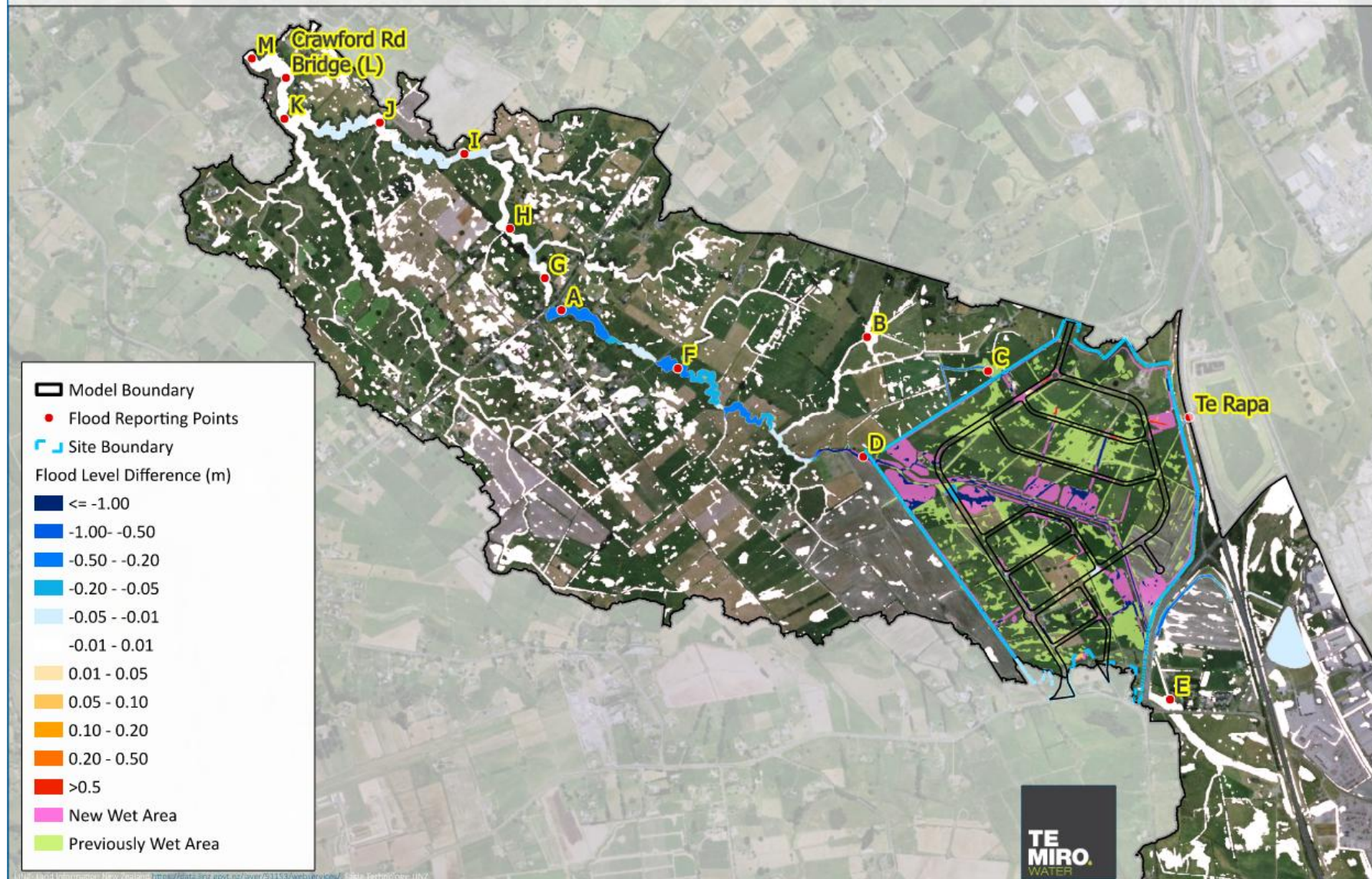


Figure 23 Maximum Water Level Comparison Map- 10-year ARI Event

TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5

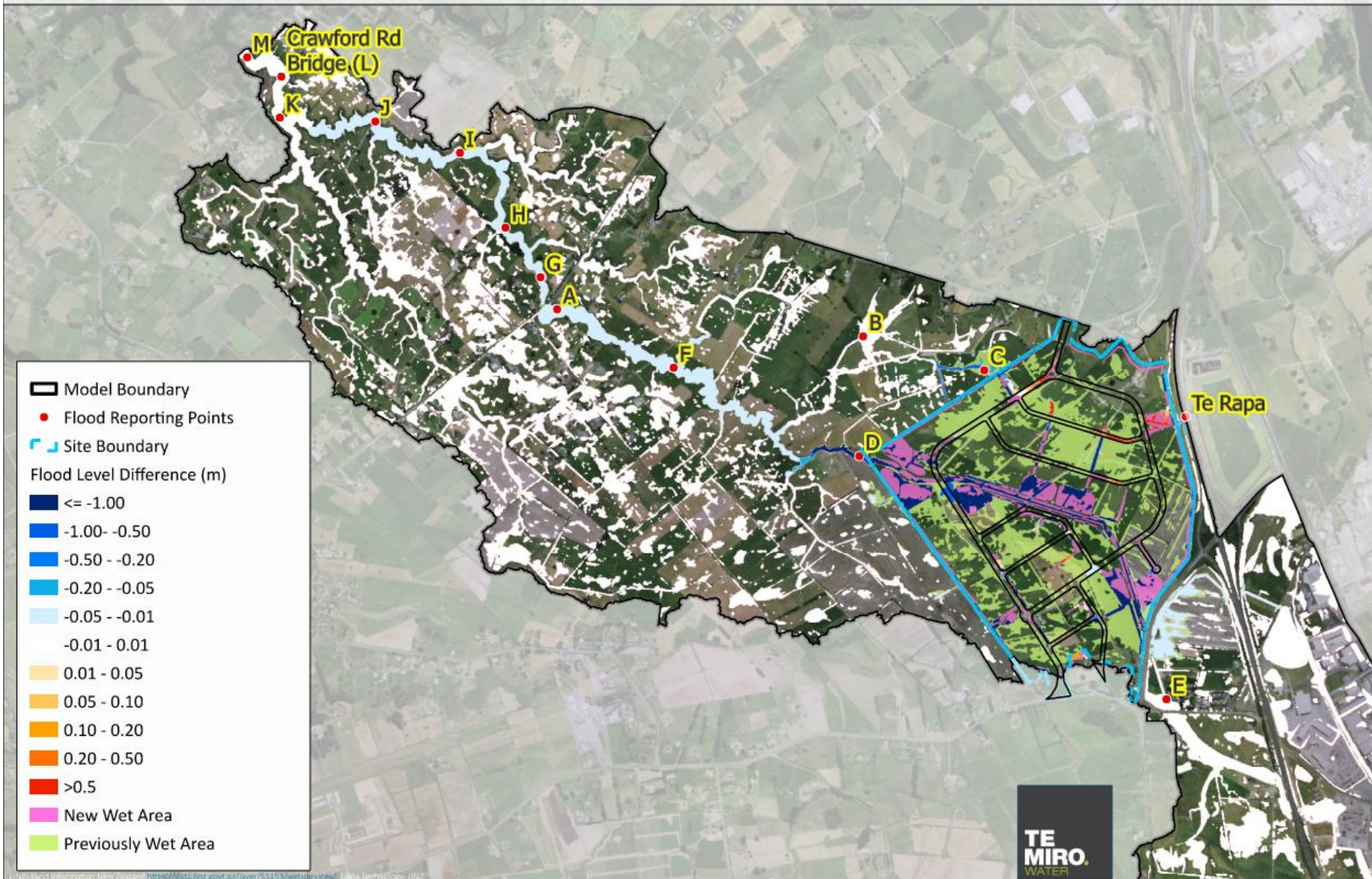


Figure 24 Maximum Water Level Comparison Map- 100-year ARI Event

TE KOWHAI EAST FLOOD MODELLING

2YR ARI MAXIMUM FLOOD VELOCITY - POST DEVELOPED (P -303) SSP5 8.5

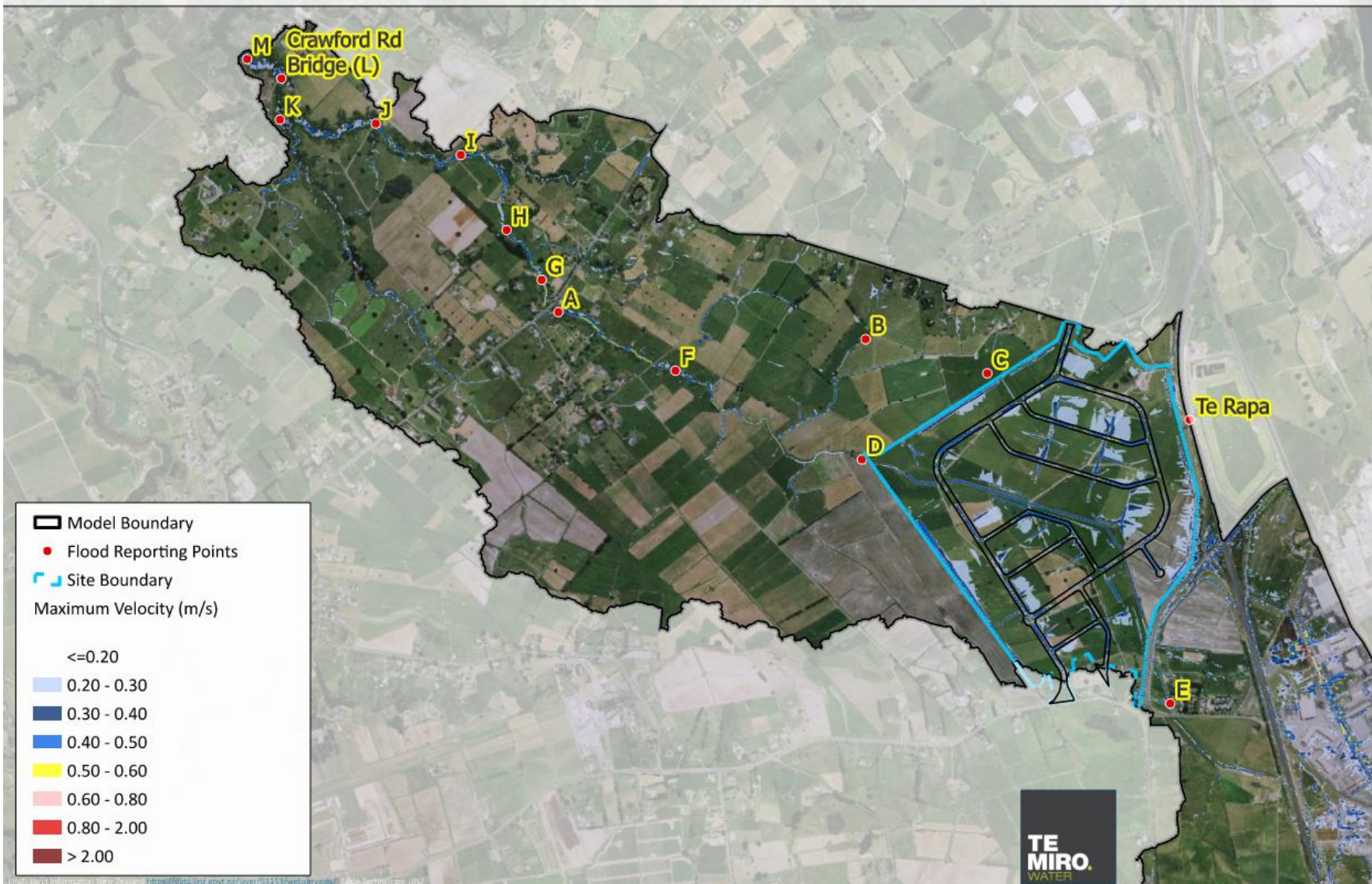


Figure 25 Maximum Velocity Post Development- 2-year ARI Event

5.4 WATER LEVEL DRAIN DOWN TIMES

The model confirms that excess runoff volume occurs under post-development conditions for the 10-year and 100-year ARI storm events. This excess volume is most evident along the Mangaheka Stream channel and at hydraulic restrictions, such as culverts and bridges. The model results confirm that while there is no increase in peak water levels, the increased run-off volume post-development results in prolonged elevated water levels compared with pre-development conditions within the central channel.

5.4.1 McBeth Drain

The Mangaheka Stream channel between Koura Drive and Horotiu Road is a WRC managed drain (McBeth Drain) and accordingly has a level of service (LOS) to drain the runoff volume from a 10-year ARI 24-hour event within 3-days (72-hours), to prevent pasture die-off. As per the BoD, post-development drainage conditions are required to meet this level of service. Table 18 below confirms the 10-year ARI drain down times at Points A and F along the drain downstream of the site. A top of stream bank elevation has been taken from the LIDAR surface and used as a point of reference to measure drain down times. Note this assessment point is conservatively adopted as the top of the low-flow channel located at the base of the incised gully. The drain down time measures the time taken for flows to recede back “within bank”. Post development results show the drainage channel still meets the WRC level of service and the drain down times are still under the 3-day (72-hour) limit.

Table 18: 10-year ARI Drain Down Times in McBeth Drain

REPORTING POINT	TOP OF BANK, RL (M)	PRE-DEV DRAIN DOWN TIME (HRS)	POST DEV DRAIN DOWN TIME (HRS)	DIFFERENCE (HOURS)
A	19.5	21.63	31.15	+9.52
F	21.5	14.63	17.30	+2.67

Refer Figure 26 and Figure 27 below for level hydrographs at Point F and Point A along the WRC managed drain showing the reduced peak water level but prolonged drain down time. The top of bank reference point is shown with a bold black line.

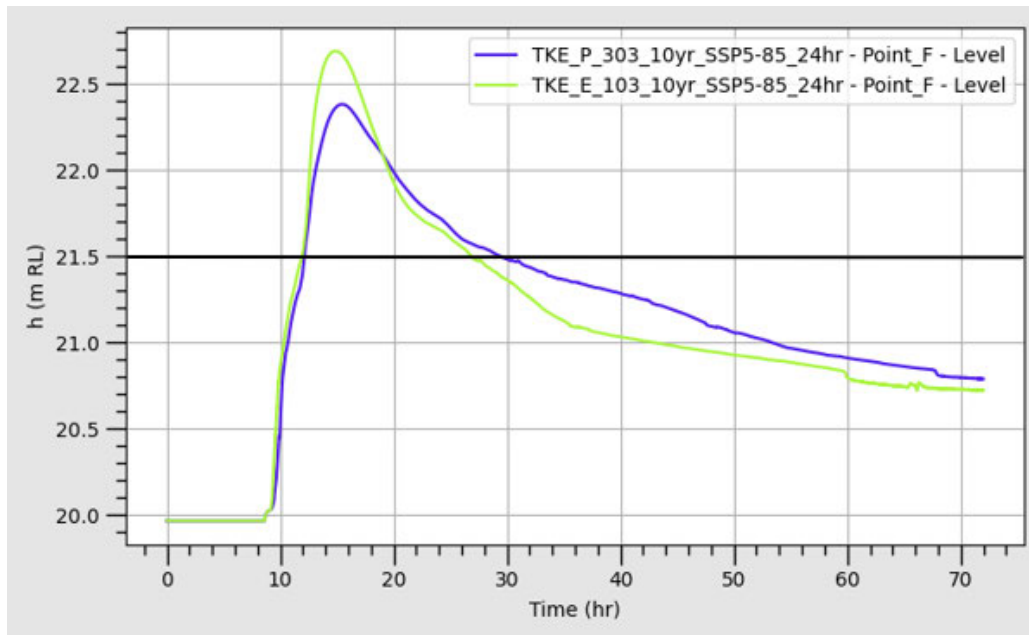


Figure 26 10-Year ARI Level Hydrograph -Point F

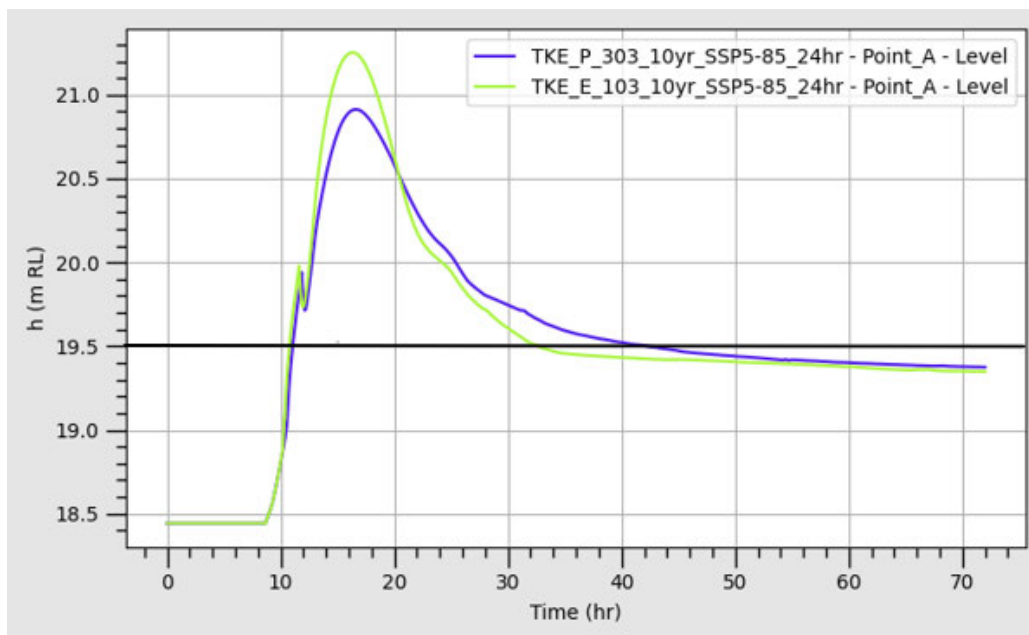


Figure 27 10-Year ARI Level Hydrograph-Point A

The reference point for the drain down time comparison is the top of the bank of the low-flow channel which sits within the incised gully area. Whilst the low-lying gully floor is occasionally used for grazing, the main farming activities within the WRC managed drain catchment occur on the elevated ground well above the maximum 10-year ARI flood level. Accordingly, the increased drain down time is not expected to adversely affect the majority of farming activities within the drain catchment.

Figure 28 below illustrates the incised nature of the McBeth drain and assessment point (RL19.5m) at Point A.

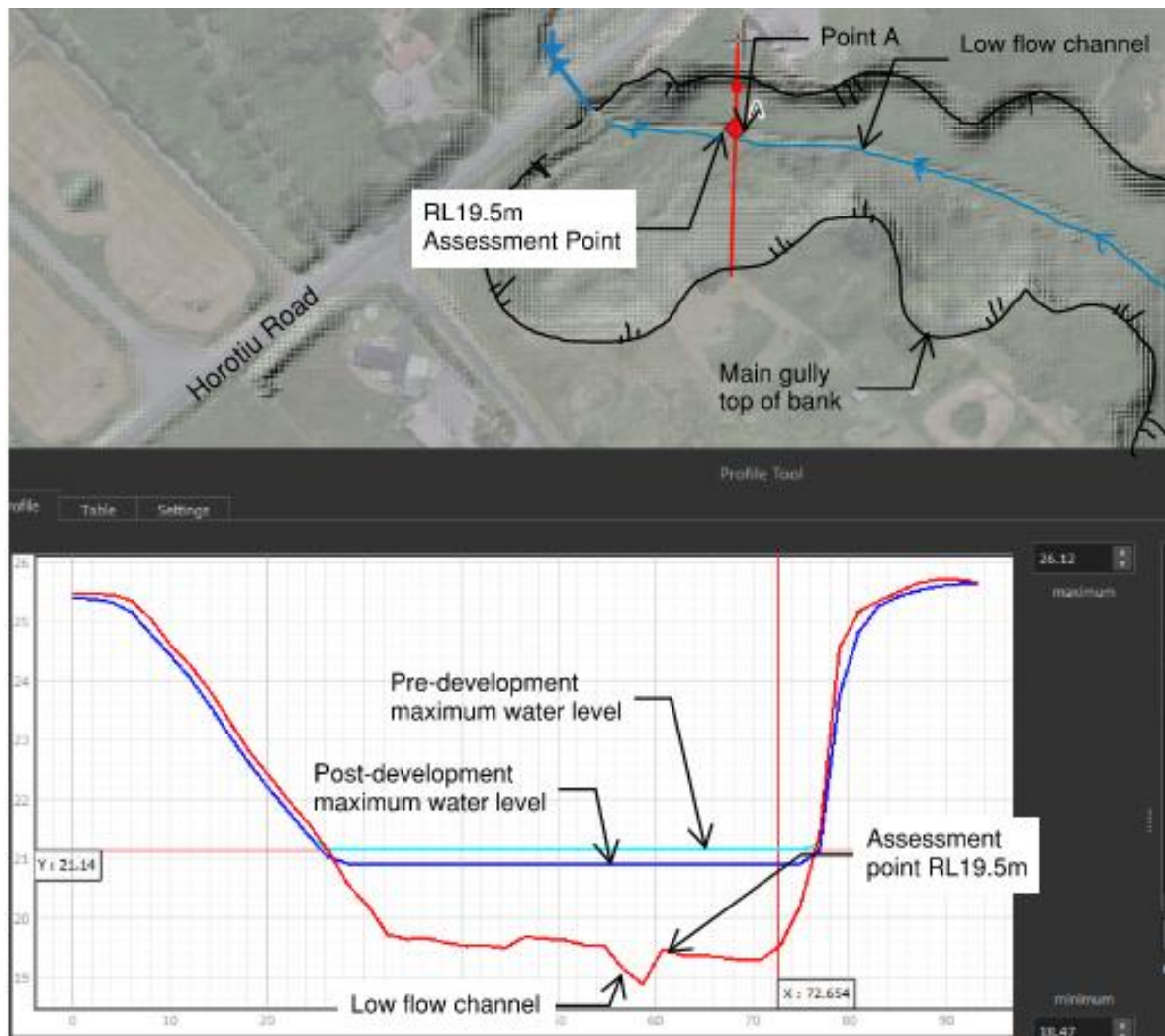


Figure 28 Section at Reporting Point A showing low-flow channel located within the incised gully.

Downstream of Horotiu Road (outside the WRC drain area) the effects of the excess post-development runoff volume in the 10- and 100-year ARI events become less noticeable. Peak water levels are all lower than the pre-development scenario, however water levels remain elevated for extended periods. Refer to Table 19 below which summarises the peak water levels and drain down times to a nominal bank line in places. As for Points A and F upstream, the results show the drain down times are well under the 72-hour limit.

Table 19 10-year ARI event drain down times downstream of Horotiu Road

REPORTING POINT	TOP OF BANK, RL (M)	PRE-DEV DRAIN DOWN TIME (HRS)	POST DEV DRAIN DOWN TIME (HRS)	DIFFERENCE (HOURS)
I	15.90	36.23	45.60	+9.37
L (Crawford Bridge)	14.60*	8.25	8.98	+0.73
M	13.55	23.13	30.88	+7.75

* - Bridge Deck level

5.5 CRAWFORD ROAD BRIDGE

The model results show the Crawford Road bridge acts as a hydraulic restriction on the Mangaheka channel with the model confirming the water levels exceeds the bridge deck level in the existing and post-developed 10 and 100-year ARI events. The model shows the 2-year ARI event overtops a low point in the road adjacent to the bridge in both the pre- and post-development scenarios.

Refer to Figure 29 and Figure 30 below for hydrographs of the 10 and 100-year ARI water levels (pre and post-development) at the Crawford Road Bridge. The figures demonstrate the reduction in peak water level post-development compared to the pre-development case. There is a slight increase in the duration of ponding on the road in the 100-year ARI event in the post-development case with water levels shown to be above the deck level for 20 hours post-development compared with 18 hours pre-development (an increase of 2 hours). There is a nominal increase in ponding duration on the bridge in the 10-year ARI event shown in Table 19 and Figure 29 below.

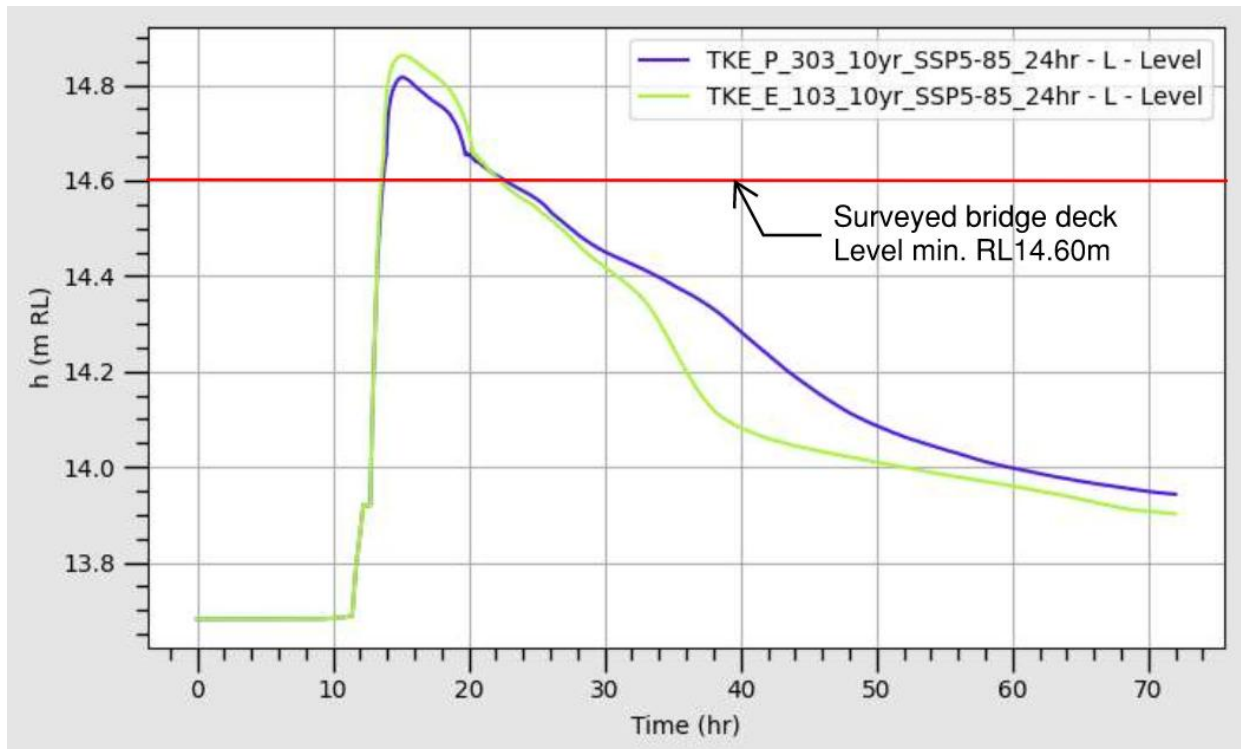


Figure 29 10-year ARI water levels at Crawford Road Bridge

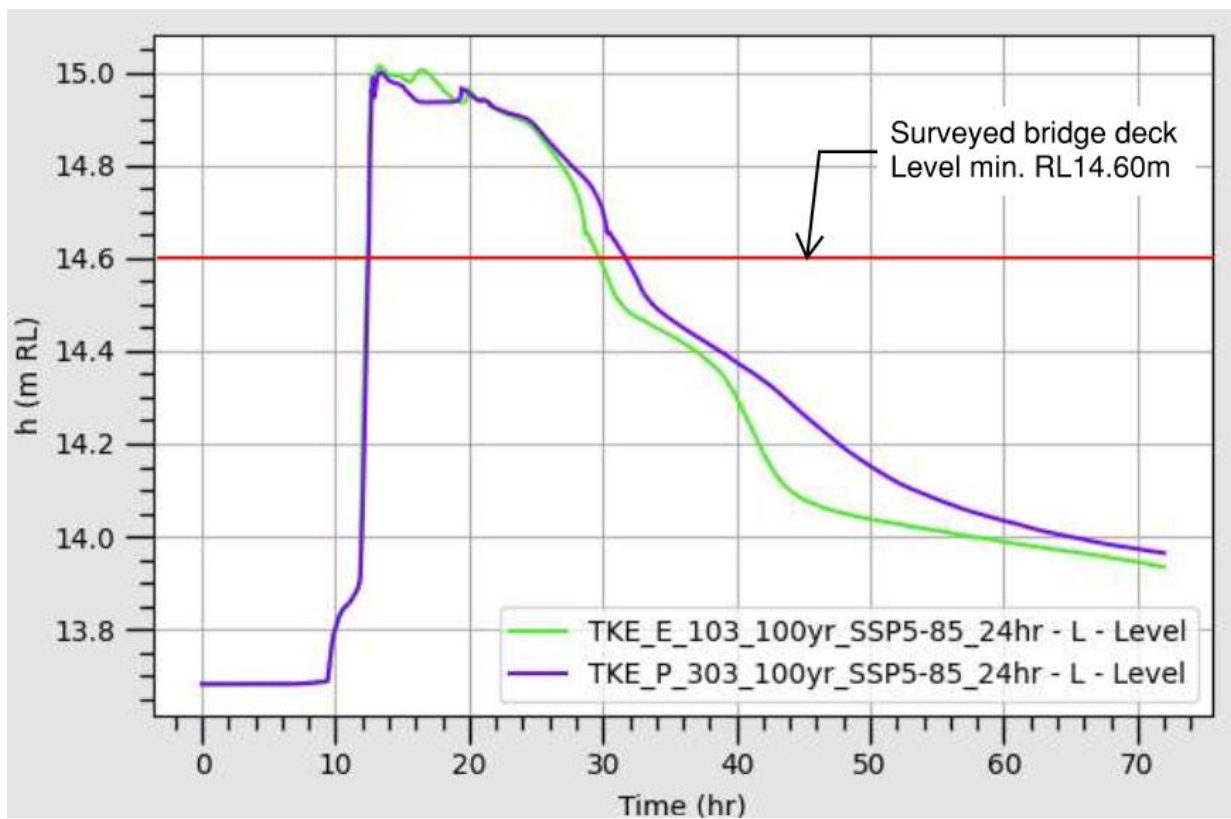


Figure 30 100-year ARI Water Level at Crawford Road Bridge

Model results show that peak velocities at the bridge are elevated compared to the upstream and downstream sections of the channel, however as the bridge was modelled as a 2-dimensional structure, TufLOW returns depth-average velocities and it is not possible to distinguish peak flows or velocities for flows under the bridge to those over the bridge deck. Accordingly, a HY8 model was set up to model the bridge as a 1D hydraulic structure for comparison with TUFLOW results. The opening beneath the bridge was modelled as an embedded box culvert with the road crossing above. Bridge details were taken from survey undertaken by Maven Ltd. Surveyed bridge deck levels are similar to the LIDAR levels, so the lidar surface was adopted to give a complete road profile over the full width of the road flow.

Refer to Figure 31 below for a photo of the bridge looking downstream.



Figure 31 Photo of Crawford Road Bridge

A snip of the HY8 model is in Figure 32 below.

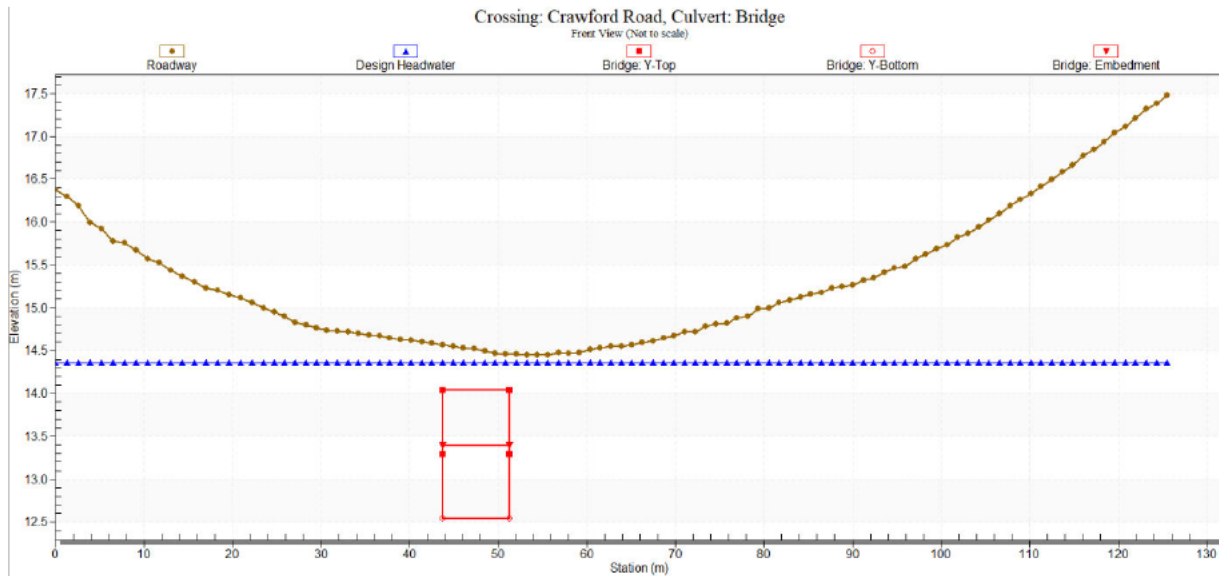


Figure 32 HY8 model of Crawford Road Bridge opening

The HY8 model was run for the 2year ARI event post-development flows. The results were compared to the TufLOW model results in Table 20 below.

Table 20 Crawford Road 2yr Velocity Results Comparison -Post Development

METRIC	TUFLOW MODEL RESULTS	HY8 RESULTS
Total crossing flow (m ³ /s)	5.18	5.18
Peak water level, RL(m)	14.56	14.14
Culvert flow (m ³ /s)	-	5.18
Velocity (m/s)	1.47 (averaged)	1.07 (within culvert)
Tailwater Velocity (m/s)	0.50	0.43

The HY8 model results are comparable to the TufLOW results and confirm that velocities beneath the Crawford Road bridge are likely to be elevated above typical channel values during the 2-year ARI event, with a velocity above the permissible value of 0.61m/s. The HY8 assessment and TufLOW model demonstrate that velocities downstream quickly return to the lower values typically seen elsewhere along the channel.

The model results show post- development velocities do not exceed pre-development velocities under the bridge.

5.6 PARTIAL AND FULL VOLUME NEUTRALITY

Several sensitivity checks were simulated in the model to understand the effects that partial or full volume neutrality for the 2-year ARI event would have on the peak flows and velocities in the downstream receiving environment.

Table 21 describes the modelled cases ID's and descriptions.

Table 21 Volume Sensitivity Runs

SENSITIVITY CHECK	DESCRIPTION
Partial volume retention _[P305]	The partial soakage option provides approximately 23,000 m ³ soakage or reuse in the Mangaheka catchment of the site and approximately 2,000 m ³ in the Te Rapa Stream catchment of the site.
Full volume neutrality _[P306]	Full neutrality. Approximately 44,000m ³ soakage or re-use in the Mangaheka catchment of the site. Approximately 4,000m ³ in the Te Rapa Stream catchment of the site.

The soakage /reuse volume was simulated within the model by using polygons in select locations with a constant loss rate applied (in m³/s) to the polygon. The constant loss value was modified via a trial-and-error process until the approximate desired loss volume was achieved. The hydrology for these model runs included the 10mm Initial Abstraction loss.

The model results show that increased volume reduction at source over and above the 10mm retention does result in a reduction in peak flows and velocities in the receiving environments. Refer Tables 22 and 23 below for a summary of model results at select reporting locations.

Table 22 Partial Volume Reduction Results- Post-development 2-Year ARI [P_305]

CATCHMENT / LOCATION	PEAK FLOW, Q (M ³ /S)	VELOCITY, V (M/S)
Mangaheka - Point D	2.74	0.53
Mangaheka - Point A	3.63	0.21
Mangaheka - Point I	3.93	0.26
Mangaheka – Point L Crawford Road Bridge	5.00	1.28*
Mangaheka – Point M	5.01	0.40
Te Rapa Stream - WEX culvert	0.35	1.27**

* Peak velocities across whole crossing- see above for further explanation

** Peak flows measured within the culvert

Table 23 Full Volume Neutrality Results -Post-Development 2-Year ARI [P_306]

CATCHMENT / LOCATION	PEAK FLOW, Q (M ³ /S)	VELOCITY, V (M/S)
Mangaheka - Point D	2.58	0.51
Mangaheka - Point A	3.57	0.21
Mangaheka - Point I	3.78	0.25
Mangaheka – Point L Crawford Road Bridge	4.84	1.28*
Mangaheka – Point M	4.85	0.39
Te Rapa Stream - WEX culvert	0.34	1.26**

* Velocity across whole crossing- see above for further explanation

** Velocity measured within the WEX culvert, not the downstream channel/stream

5.6.1 Velocity Duration

The effect of increased runoff volume on flow hydrographs shows up as a delayed or extended recession limb compared to the pre-development scenario. The effect of the extended volumetric discharges can also be seen on a velocity hydrograph which illustrates the duration of elevated velocities that occurs due to the extended duration of flow.

Figure 33 and Figure 34 below plot the velocity against time at several locations downstream of the development, for the various post-development and retention scenarios that have been modelled.

The plots demonstrate the effect that increased retention has on peak velocities. The plots demonstrate the velocities are below the maximum velocity for the soil type i.e. 0.61m/s as per TR2020/07 however demonstrate the increased duration of (lower) velocity within the drains.

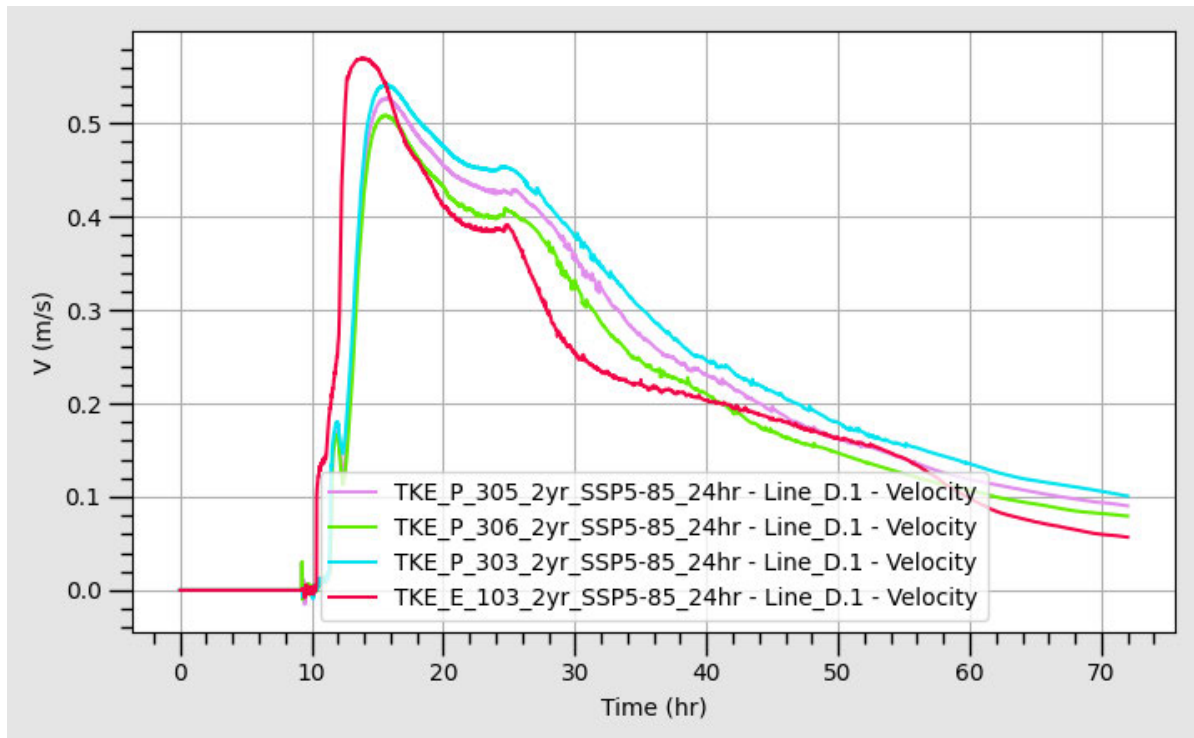


Figure 33 Velocity Hydrograph at Point D immediately downstream of the site

At Point D, P_303 results in an increase of 3 hours at 0.50m/s, an increase of 7 hours at 0.40m/s and an increase of 7 hours at 0.30m/s.

At Point M (the downstream end of the model), the volume effect is still noticeable, albeit reduced. P_303 has reduced velocities compared to the pre-development case and the increased duration at lower velocities is less noticeable. P_303 has an increased duration of approximately 4 hours at 0.3m/s, approximately 4.5 hours at 0.2m/s and approximately 5 hours at 0.1m/s.

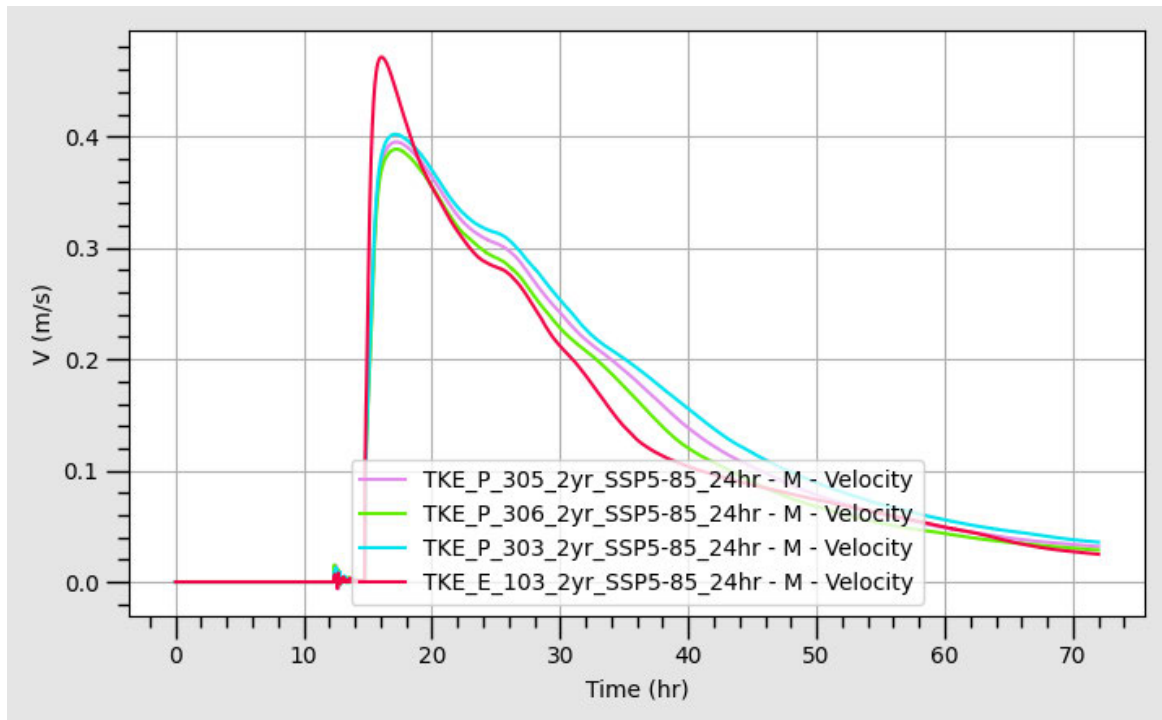


Figure 34 Velocity Hydrograph at Point M (downstream of Crawford Road)

5.6.2 Retention Methods

To achieve full volume neutrality over the development site, significant soakage or re-use will be required. To match pre -development volume, the sub-catchment draining to the Mangaheka Stream requires the equivalent soakage or re-use of approximately 26 mm depth over the 169.7Ha contributing catchment. For the approximate 9.83Ha sub-catchment which discharges to the Te Rapa Stream, 2-year ARI volume neutrality requires the soakage and reuse of approximately 38mm depth. In volumetric terms of reuse/storage per hectare of development, this equates to approximately 260m³/Ha and 407m³/Ha in the Mangaheka and Te Rapa sub-catchments respectively. Detailed simulation of soakage accounting for infiltration will likely reduce the actual volume of storage required.

6 SENSITIVITY CHECKS

The following sensitivity runs were undertaken following initial model reviews from HCC's Stormwater Subject Matter Expert (SME).

6.1 EFFECTS OF RESIDUAL PONDING IN THE MODEL

The ROG model does not include the stormwater reticulation network, which has resulted in excess ponding or depression storage, being retained in the existing built-up areas such as the Te Rapa industrial area. To determine whether runoff volume resulted in increases downstream of the culverts, the following adjustments were applied to the model as a sensitivity run:

- Additional z-lines were introduced to represent pipes draining the larger ponded areas to adjacent wetlands or channels.
- The rainfall hydrograph over this area was increased by 20% to generate additional runoff volume.

The results of this sensitivity analysis confirmed that the additional rainfall and additional z-lines resulted in additional runoff volume which was conveyed downstream. Peak flows within the culverts also increased as a result of the additional runoff with increases of between 10-40% compared with the base model.

Increases were more noticeable for the larger events. Peak flow comparisons are presented in Figure 35 and Figure 36 and were taken at the Koura Drive culverts.

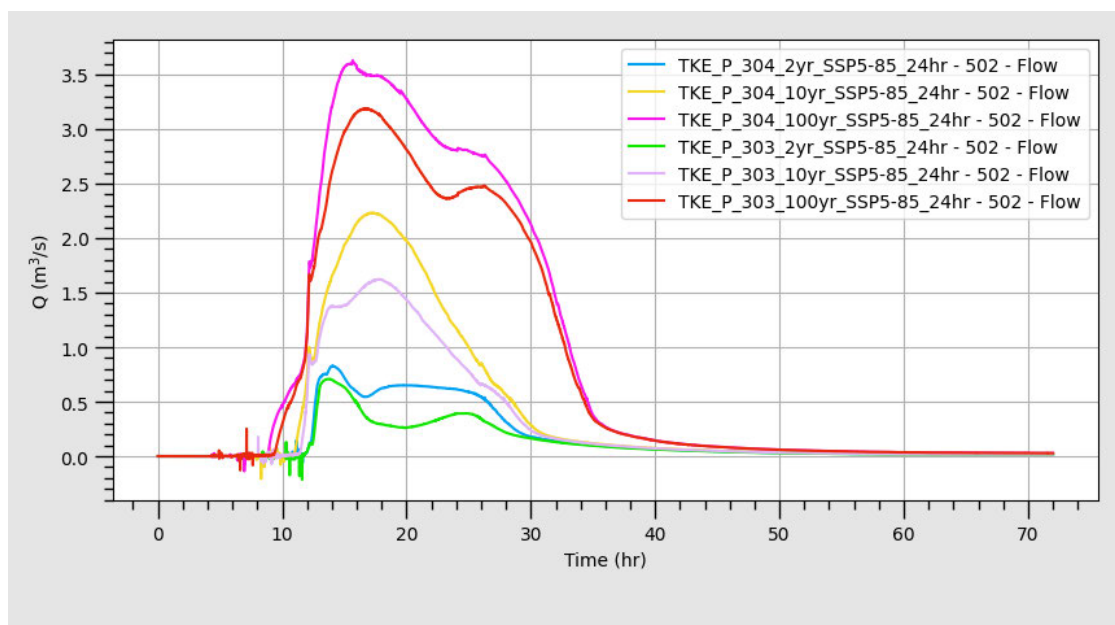


Figure 35 Peak flows at Culvert 502 Koura Drive comparing Design case vs Sensitivity

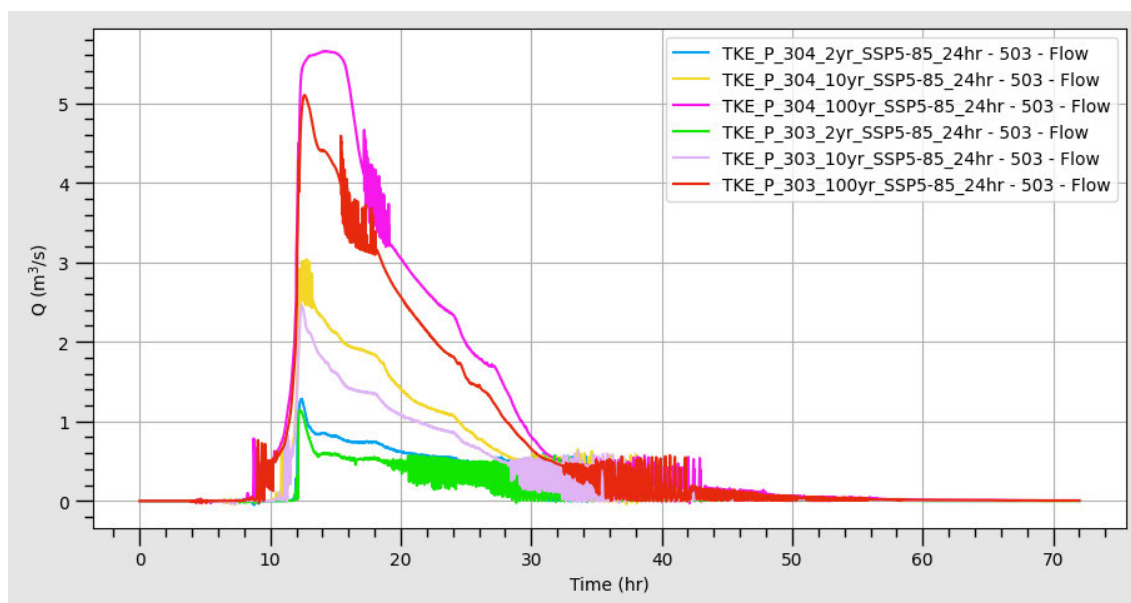


Figure 36 Peak flows at Culvert 503 Koura Drive comparing Design case vs Sensitivity

The sensitivity check confirmed that additional runoff volume generated within the Te Rapa area does result in slight increases in flows and through the Koura Drive culverts and this signal is more noticeable for the larger storm events. The signal dissipates downstream of the site as shown in Figure 37 (Point D). Detailed design should consider these increased flows from the upper catchment in the design of the Mangaheka Stream channel through the development and associated drainage system (culverts etc).

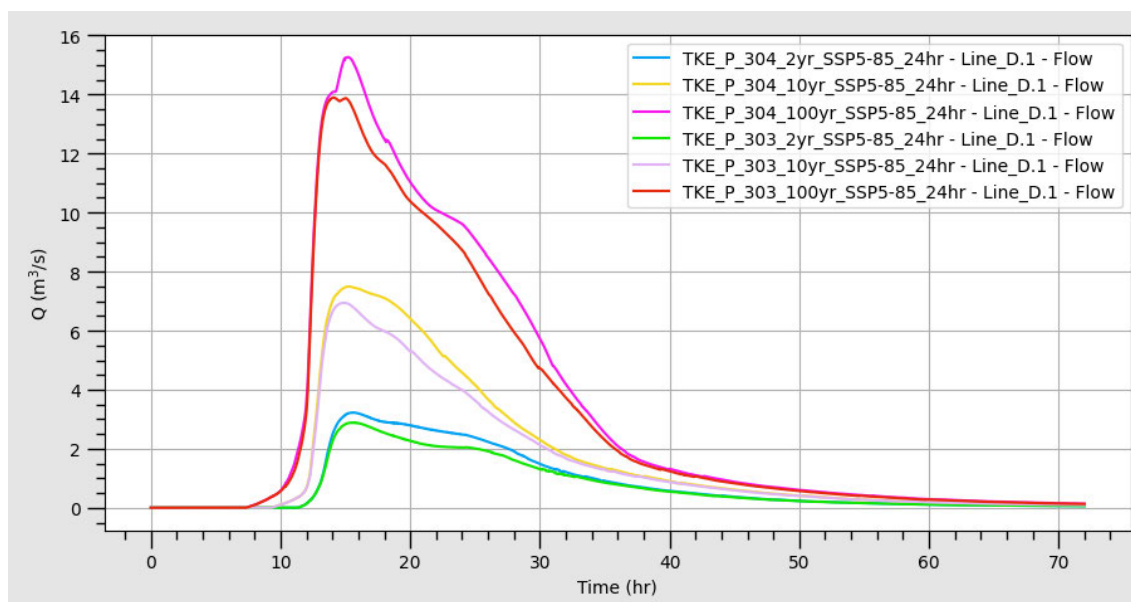


Figure 37 Peak flows at Point D comparing Design case vs Sensitivity

6.2 ROTOKAURI GREENWAY EMERGENCY SPILLWAY FLOWS

As discussed in Section 4.4.1 above, the Rotokauri Greenway located to the south of the proposed Te Kowhai East development area includes an emergency spillway. In the 100-year ARI post-development scenario with RCP8.5 climate change adjusted rainfall, the spillway is activated and discharges overflows to the north, through Burbush Road properties, under Te Kowhai Road via the culvert, and into the Te Kowhai East model area.

Refer Figure 38 below for a snip of the Rotokauri greenway flood model and spillway location in relation to the Te Kowhai East area.

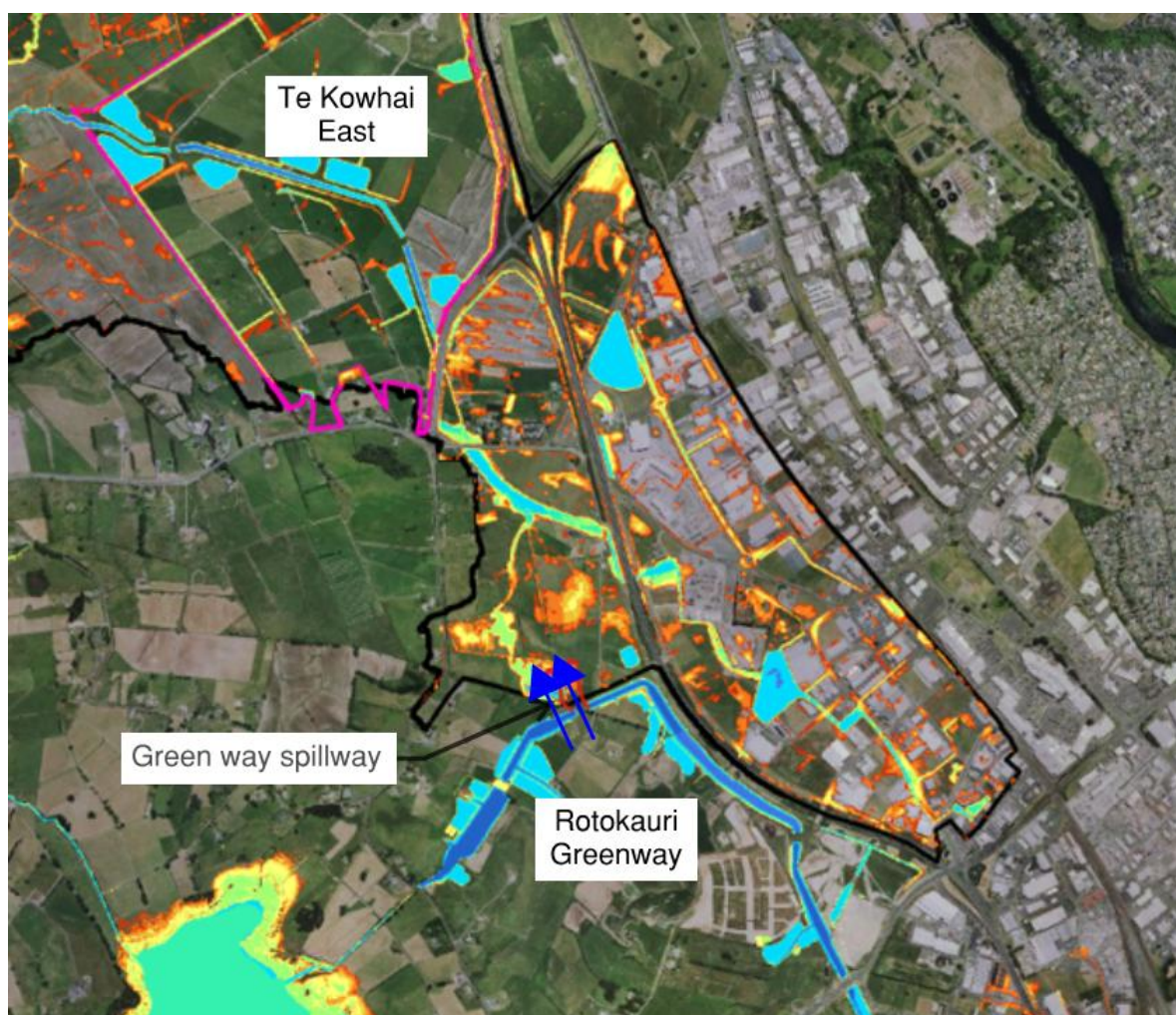


Figure 38 Snip showing Rotokauri Greenway Spillway flows in relation to Te Kowhai East Area

The hydrograph from the Greenway spillway was inputted into the Te Kowhai East model as a boundary condition as per Section 4.4.1 above. Due to the timing and magnitude of peak flows from the emergency spillway there were no peak flow effects of the downstream however the additional volume from the spillway

overflow is conveyed into the Mangaheka channel and is evident in the model result. Figure 39 and Figure 40 below contain post-development 100-year hydrographs at Point 502 which is the southern Koura Drive culvert and Point 503, the northern Koura Drive culvert. The hydrographs compare the peak flows from model runs P_303 and P_302 (with and without the spillway flows, respectively). P_303 (red line) clearly shows the additional peak from the spillway at time 26.2hrs. In total about 48000 m³ additional volume passes through the Koura Drive culverts because of the spillway flows. The entirety of this volume is conveyed though the central modified channel within the development site and to the downstream Mangaheka Channel.

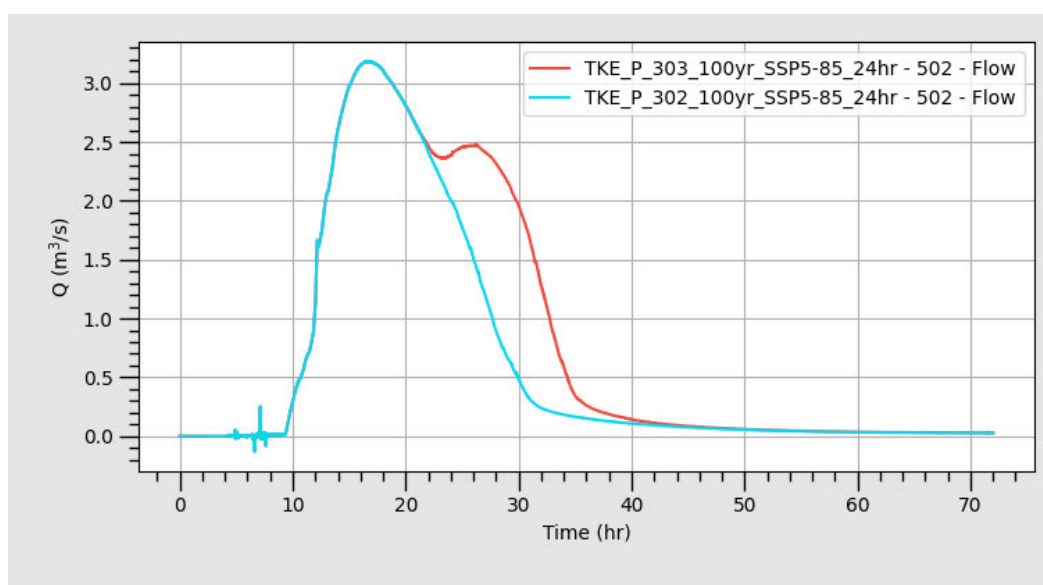


Figure 39 Point 502 (Southern Koura Drive Culvert) comparing flows with and without spillway flows

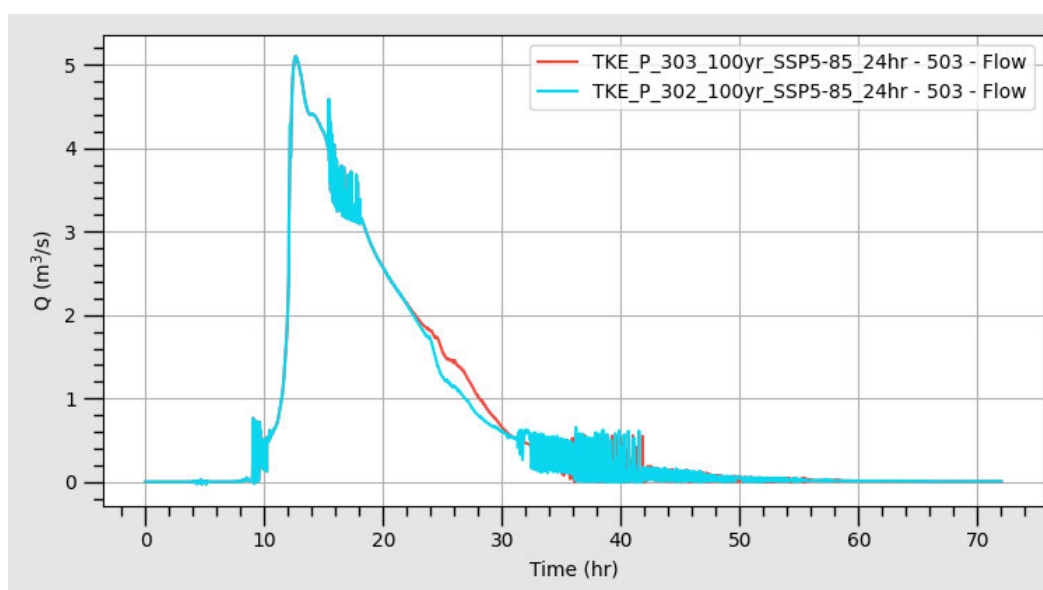


Figure 40 Point 503 (Northern Koura Drive Culvert) comparing flows with and without spillway flows

Figure 41 and Figure 42 below show the effect of this volume at Point D (Mangaheka Stream channel immediately downstream of the development site). In summary the Rotokauri Greenway flows result in no changes to peak flows or maximum levels, however, they result in a slight flattening of the recession limb indicating water levels remain elevated within the channel for approximately 3 hours.

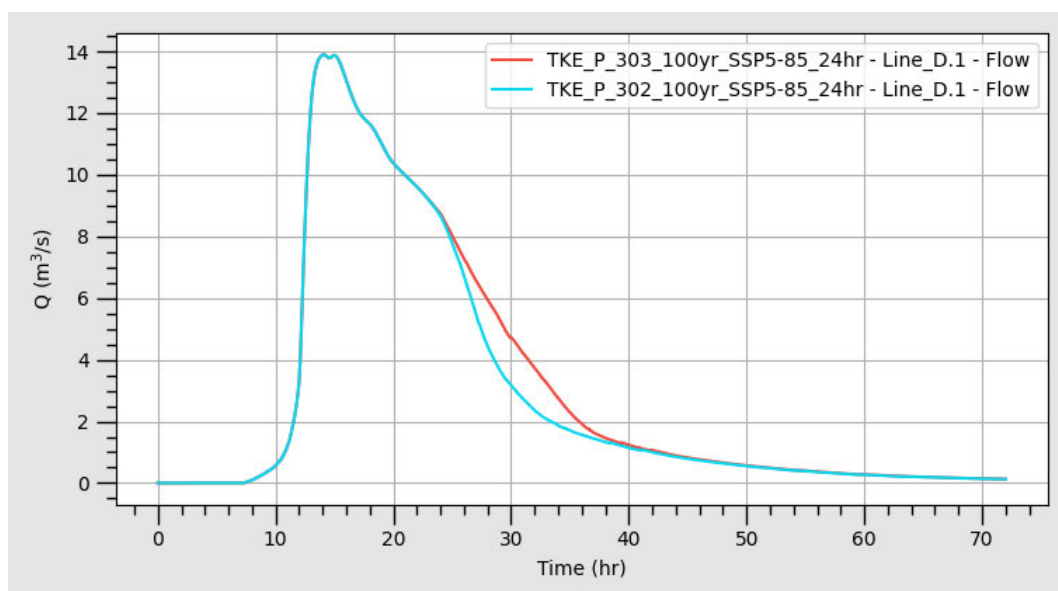


Figure 41 Peak Flow Hydrograph at Point D.

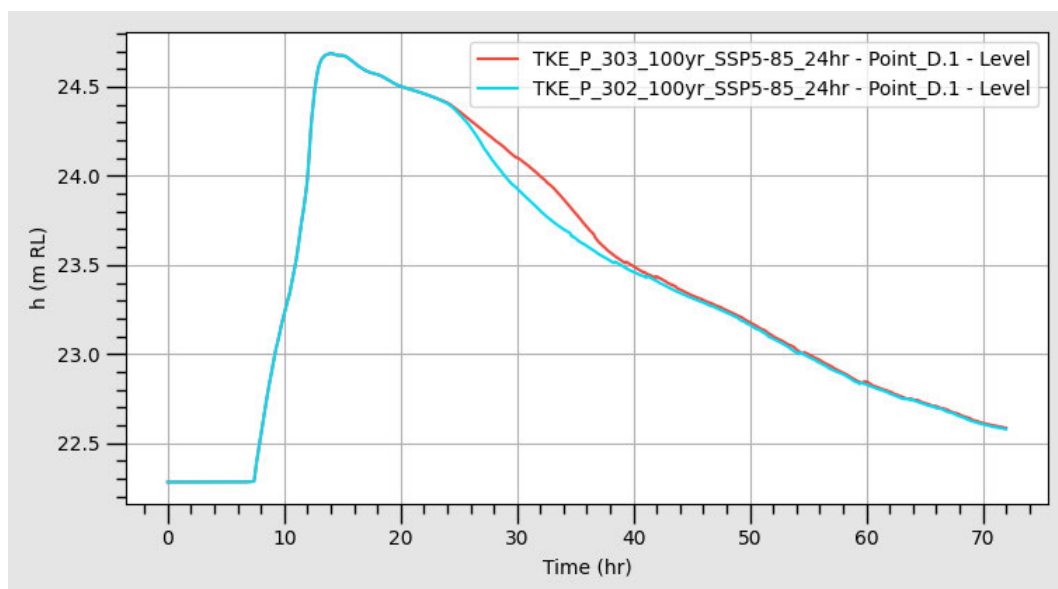


Figure 42 Maximum Water level at Point D

Table 24 Drain downtime at Point D

DRAIN TOP OF BANK LEVEL AT POINT D , RL (M)	DRAIN DOWN TIME– P_302 (HOURS)	DRAIN DOWN TIME– P_303 (HOURS)	DIFFERENCE (HOURS)
24.0	16.90	19.93	+3.03

The significance of this additional volume on development channel design will be negligible however to ensure any potential effects are captured in design, the design base case P_303 has retained the spillway flows.

6.3 WAIKATO REGIONAL COUNCIL MANAGED DRAIN PERFORMANCE

Related to the drawn down time of the 10-year 24 storm event clearing pasture within 72 hours, Waikato Regional Council Stormwater Management Plan (TR2020/07) has a drainage standard for their managed drains. In accordance with Table 15-1 in TR2020/07 'other' drains in the Waikato require 38mm of runoff to be cleared within 24hours. A check was done on the post-developed runoff from the Mangaheka catchment including the development site to determine the runoff depth the main McBeth drain discharges over 24 hours. The assessment was undertaken at the downstream end of the drain at reporting Point A. The assessment is summarised below.

- Total catchment to Point A = 776.7Ha
- Total runoff volume at Point A (in the first 24hrs) = 295,920m³ (extracted from the TufLOW model)
- Runoff Depth = Runoff Volume/ Catchment Area = 0.03809m or 38mm.

Accordingly, the drainage standard is being achieved at the downstream end of the WRC managed McBeth drain in the post-development scenario.

7 PEER REVIEW

89 Degrees Ltd have been engaged to undertake peer review of the hydraulic model. Their scope included the following;

- Model Schematisation and Fitness for Purpose
- Input Data Review
- Model Setup and Numerical Implementation
- Robustness, Sensitivity and Uncertainty
- Reporting and Interpretation of results

The peer review found the TUFLOW model to be generally suitable for its intended pre- and post-development assessment, subject to the below comments being addressed.

1. No fundamental issues were identified with the model methodology or its use for the assessed design events.
2. The model results showed generally coherent hydraulic behaviour and good numerical performance.
3. Flow paths, flood depths, water levels, velocities, hydraulic structures and representative 1D/2D coupling were generally consistent with the expected hydraulic response.
4. The post-development 100-year ARI results showed attenuation of peak flows and a smoother, more distributed downstream hydrograph response, consistent with the proposed storage and flow-control system.
5. Changes in depths, water levels and velocities were mainly associated with areas modified by the proposed development, such as wetlands and storage areas.
6. No material adverse effects were identified outside the areas modified by the proposed development, based on the results reviewed.

Key implementation matters requiring confirmation or refinement included:

- MIN/GULLY terrain-modification treatment used with SGS, where local saw-tooth effects were identified.
- SX connections using Z-flag adjustments, including locations with relatively large level adjustments between 1D connection levels and 2D terrain.
- Localised oscillatory behaviour at some culvert connections, requiring confirmation of connection geometry, levels and nearby 2D storage representation.
- Adopted initial water levels, including whether they are appropriate for the intended antecedent design-event conditions.

Te Miro Water have reviewed the comments provided by 89 Degrees Ltd and have undertaken several sensitivity tests to verify whether the recommended changes resulted in any differences to the model results. In all cases the sensitivity tests didn't result in any material differences to results. Accordingly, the original parameters and methodology have been considered fit for purpose. TMW have responded to all received comments and 89 Degrees has reviewed these responses and confirmed that there are no outstanding issues.

The peer review summary report and is included in Appendix B.

8 SUMMARY AND CONCLUSION

The hydrologic and hydraulic assessment has established the baseline runoff, flood behaviour and receiving environment conditions for the Te Kowhai East site, including the WRC-managed McBeth Drain, Mangaheka Stream, Te Rapa Stream and downstream hydraulic controls such as Horotiu Road and Crawford Road Bridge. The model has been developed using LiDAR-derived terrain, rain-on-grid hydrology, SCS Curve Number rainfall losses, adopted seasonal groundwater soil classifications, modelled hydraulic controls, proposed development earthworks, proposed constructed wetlands, modified channels and climate-change-adjusted design rainfall. The hydrologic and hydraulic model assessment is considered suitable for evaluating the relative stormwater effects of the proposed development and the performance of the proposed mitigation strategy.

Peak Flow

In summary, the post-development modelling demonstrates that the proposed stormwater management strategy is capable of controlling peak flows, water levels and velocities in the downstream receiving environments. For the Mangaheka Stream, post-development peak flows are below the existing summer baseline at all assessed reporting points for the 2-year, 10-year and 100-year ARI events, with peak flows generally reducing to approximately 80%, 95% and 88% of pre-development flows respectively. For the Te Rapa Stream, post-development peak flows are also controlled to approximately 100%, 85% and 100% of pre-development flows for the same events.

Peak flows from the development site only are well below the existing or historical event peak flows (no climate change rainfall) improving the basis of design targets in all events.

Velocity

Post-development velocities are generally comparable to, or lower than, pre-development velocities in the 2-year ARI event and remain below the adopted limiting velocity of 0.61 m/s at the main Mangaheka Stream reporting locations. The exception is the Crawford Road Bridge location, where elevated velocities occur under both existing and post-development conditions due to the existing hydraulic restriction at the bridge. A HY-8 assessment concludes that there are elevated velocities, but these are localised to the bridge opening, with downstream velocities returning to lower values consistent with the wider channel.

Water Levels and drain down times

The results show that peak water levels in the post-development scenario are generally lower than pre-development water levels, including at key hydraulic restrictions such as Horotiu Road and Crawford Road Bridge.

At Crawford Road Bridge, peak water levels reduce by approximately 50 mm in the 2-year ARI event, 40 mm in the 10-year ARI event and 20 mm in the 100-year ARI event. No adverse afflux or reduction in flood level of service has been identified as a result of the proposed development.

Within the WRC managed drainage channel (McBeth Drain) increased drain down times post-development have been identified. The 10-year ARI drain-down time increases by approximately 9.5 hours at Point A and 2.7 hours at Point F.

Sensitivity testing

Sensitivity testing indicates that additional at-source volume reduction, through soakage or re-use, would further reduce downstream peak flows and velocities. Full 2-year ARI volume neutrality would require approximately 44,000 m³ of soakage or re-use in the Mangaheka catchment and approximately 4,000 m³ in the Te Rapa Stream catchment. While this would provide additional hydraulic benefit, the base design case incorporating the minimum 10 mm retention allowance and proposed attenuation is shown to achieve the required peak flow and water level reductions, and acceptable velocity outcomes.

A sensitivity assessment was also undertaken to test the effect of additional runoff volume from the existing Te Rapa Industrial Area, recognising that the rain-on-grid model does not explicitly include the full reticulated stormwater network and therefore retains some residual surface ponding within existing developed areas. The sensitivity run applied additional z-lines to improve drainage of ponded areas to existing wetlands or channels and increased the rainfall input over this area by 20% to generate additional downstream runoff volume. This assessment confirmed that additional runoff volume could be conveyed downstream and that peak flows through the Koura Drive culverts increased relative to the base model, with increases generally more noticeable in larger storm events. However, downstream peak flow conveyance generally remained constrained by the Koura Drive culverts, and the sensitivity results do not alter the overall conclusion that the proposed development stormwater strategy can manage peak flow and flood level effects. The outcomes of this sensitivity check should be carried forward into detailed design, where the proposed modified channel, culvert and wetland arrangements can be refined to appropriately account for potential additional runoff volume and peak flow from the upstream Te Rapa Industrial Area.

The Rotokauri Greenway emergency spillway sensitivity assessment confirms that spillway flows do not increase downstream peak flows or maximum water levels, although they add runoff volume to the modified Mangaheka channel and extend the recession limb of the hydrograph by approximately three hours at Point D. This sensitivity case has been retained in the design base case to ensure the potential volume effect is captured in the proposed channel and wetland design.

Residual effects

The principal residual effect identified by the modelling is the increase in runoff volume and associated longer duration of elevated water levels, particularly in the 10-year and 100-year ARI events, and increased duration of (lower) velocity flows in the 2-year ARI event

Within the WRC-managed McBeth Drain, the 10-year ARI drain-down time increases by approximately 9.5 hours at Point A and 2.7 hours at Point F, however the model confirms that the drain continues to achieve the required level of service, including clearance of the 10-year 24-hour event within 72 hours and discharge of approximately 38 mm runoff depth within the first 24 hours at the downstream end of the managed drain. The increased duration of ponding is within the low-lying area or floodplain of the incised gully and is not expected to adversely affect farming activities in the elevated land outside the gully. The slight increase in the duration of ponding on the Crawford Road bridge during larger events is not considered to materially worsen the existing flood hazard or level of service.

The remaining residual effect is primarily the increased runoff volume, which extends the duration of low-velocity flows. Because these flows are generally below erosion threshold velocities, the altered flow regime is not expected to materially increase erosion risk within the downstream receiving environment.

Additional mitigation could still be adopted to manage potential residual velocity effects. This may include investigating at-source soakage opportunities at each development stage, following earthworks and on-site infiltration testing, and monitoring the receiving stream for actual effects. Any instream works required to address effects attributable to the development could then be implemented if the monitoring identifies a need. We recommend these matters be managed through consent conditions and addressed during subsequent design stages, including further consideration of practical soakage and re-use opportunities within the site.

Overall, the proposed stormwater management strategy, comprising constructed wetlands, modified channel with on-line attenuation, and retention or re-use where practicable, is considered technically appropriate for the Te Kowhai East development. The modelling demonstrates that the proposed mitigation can manage development-related increases in runoff, avoid increases in downstream peak flows and flood levels, maintain peak velocities generally at or below pre-development conditions, and preserve the required level of service for the WRC-managed drain.

The background image shows a scenic view of a lake or fjord surrounded by dense green ferns in the foreground. In the distance, a large mountain range is visible under a blue sky with scattered white clouds.

ĀPITI HANGA APPENDICES

TE KOWHAI EAST FLOOD MODELLING

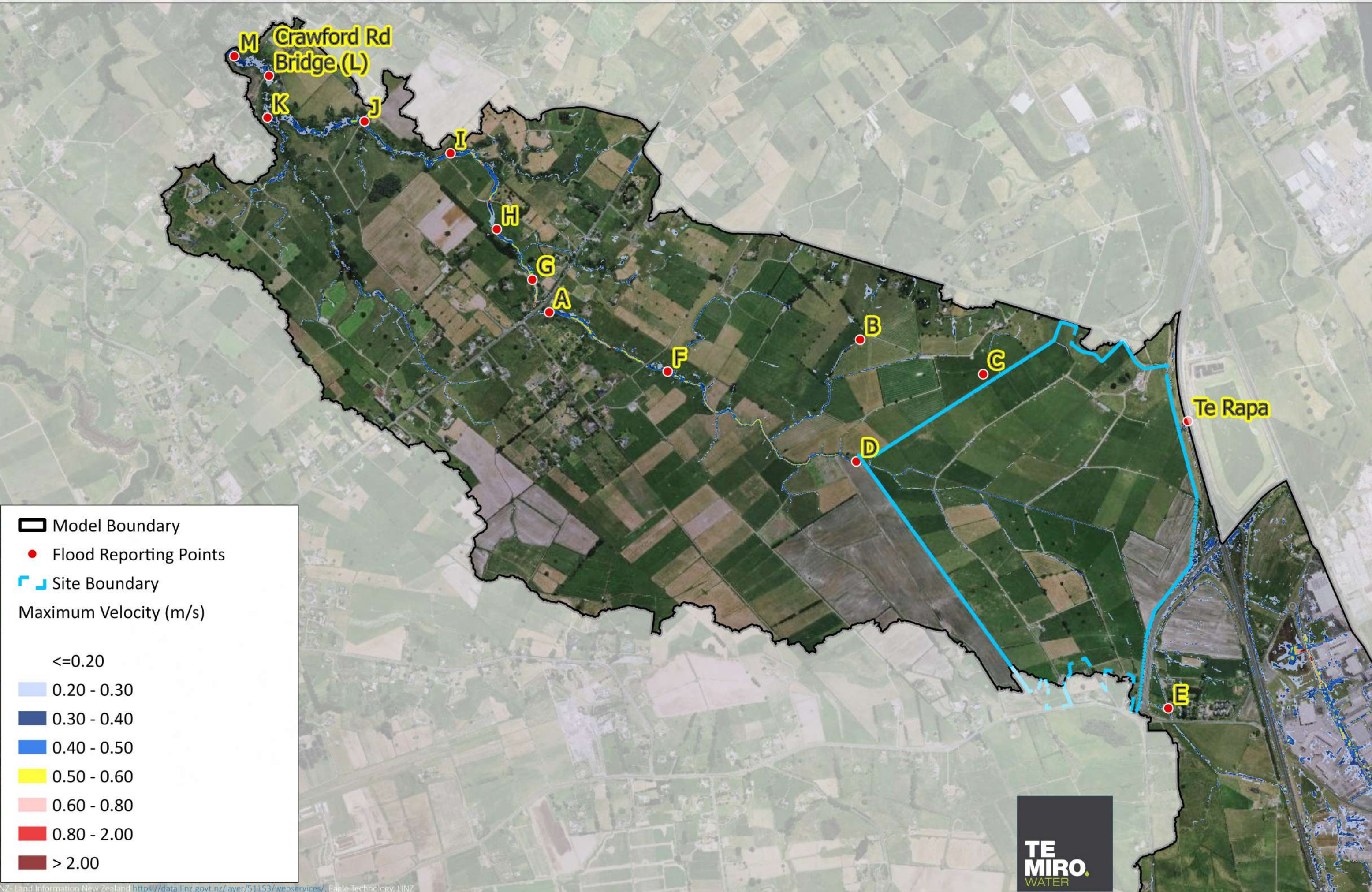
2YR ARI MAXIMUM FLOOD VELOCITY - EXISTING (E -101) SSP5 8.5



0

0.5

1 KM



- Model Boundary
- Flood Reporting Points
- Site Boundary
- Maximum Velocity (m/s)

	≤ 0.20
	0.20 - 0.30
	0.30 - 0.40
	0.40 - 0.50
	0.50 - 0.60
	0.60 - 0.80
	0.80 - 2.00
	> 2.00



TE KOWHAI EAST FLOOD MODELLING

2YR ARI MAXIMUM FLOOD VELOCITY - EXISTING (E - 101) SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

Maximum Velocity (m/s)

- 0.10 - 0.20
- 0.20 - 0.30
- 0.30 - 0.40
- 0.40 - 0.50
- 0.50 - 0.60
- 0.60 - 0.80
- 0.80 - 2.00
- > 2.00



TE KOWHAI EAST FLOOD MODELLING

2YR ARI MAXIMUM FLOOD VELOCITY - EXISTING (E - 101) SSP5 8.5



0

0.13

0.25 KM

Model Boundary

Flood Reporting Points

Maximum Velocity (m/s)

0.10 - 0.20

0.20 - 0.30

0.30 - 0.40

0.40 - 0.50

0.50 - 0.60

0.60 - 0.80

0.80 - 2.00

> 2.00



TE KOWHAI EAST FLOOD MODELLING

2YR ARI MAXIMUM FLOOD VELOCITY - EXISTING (E - 101) SSP5 8.5



0

0.13

0.25 KM

Model Boundary

Flood Reporting Points

Site Boundary

Maximum Velocity (m/s)

0.10 - 0.20

0.20 - 0.30

0.30 - 0.40

0.40 - 0.50

0.50 - 0.60

0.60 - 0.80

0.80 - 2.00

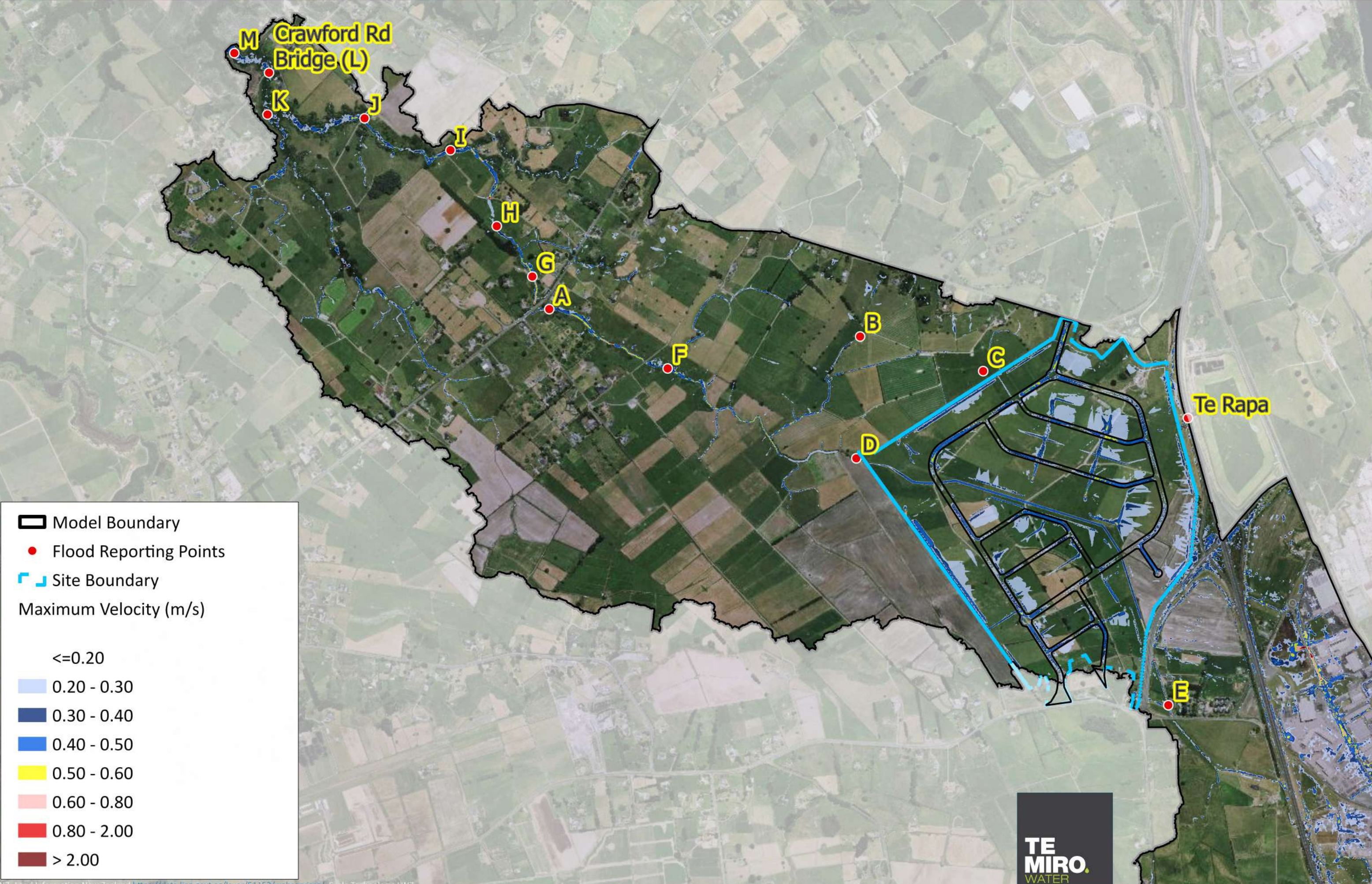
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Post Development Velocity

TE KOWHAI EAST FLOOD MODELLING

2YR ARI MAXIMUM FLOOD VELOCITY - POST DEVELOPED (P -303) SSP5 8.5



Model Boundary

Flood Reporting Points

Site Boundary

Maximum Velocity (m/s)

	<=0.20
	0.20 - 0.30
	0.30 - 0.40
	0.40 - 0.50
	0.50 - 0.60
	0.60 - 0.80
	0.80 - 2.00
	> 2.00



TE KOWHAI EAST FLOOD MODELLING

POST DEVELOPMENT - 2YR ARI MAXIMUM FLOOD VELOCITY - SSP5 8.5



Model Boundary

Flood Reporting Points

Maximum Velocity (m/s)

0.10 - 0.20
0.20 - 0.30
0.30 - 0.40
0.40 - 0.50
0.50 - 0.60
0.60 - 0.80
0.80 - 2.00
> 2.00



TE KOWHAI EAST FLOOD MODELLING

POST DEVELOPMENT - 2YR ARI MAXIMUM FLOOD VELOCITY - SSP5 8.5



Model Boundary

Flood Reporting Points

Maximum Velocity (m/s)

	0.10 - 0.20
	0.20 - 0.30
	0.30 - 0.40
	0.40 - 0.50
	0.50 - 0.60
	0.60 - 0.80
	0.80 - 2.00
	> 2.00



TE KOWHAI EAST FLOOD MODELLING

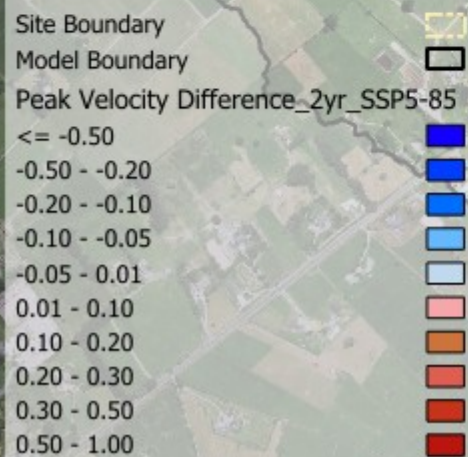
POST DEVELOPMENT - 2YR ARI MAXIMUM FLOOD VELOCITY - SSP5 8.5



- Model Boundary
 - Flood Reporting Points
 - Site Boundary
- Maximum Velocity (m/s)
- | |
|-------------|
| 0.10 - 0.20 |
| 0.20 - 0.30 |
| 0.30 - 0.40 |
| 0.40 - 0.50 |
| 0.50 - 0.60 |
| 0.60 - 0.80 |
| 0.80 - 2.00 |
| > 2.00 |

TE KOWHAI EAST FLOOD MODELLING

2YR ARI MAXIMUM FLOOD VELOCITY COMPARATIVE



Pre Development Flood Levels

TE KOWHAI EAST FLOOD MODELLING

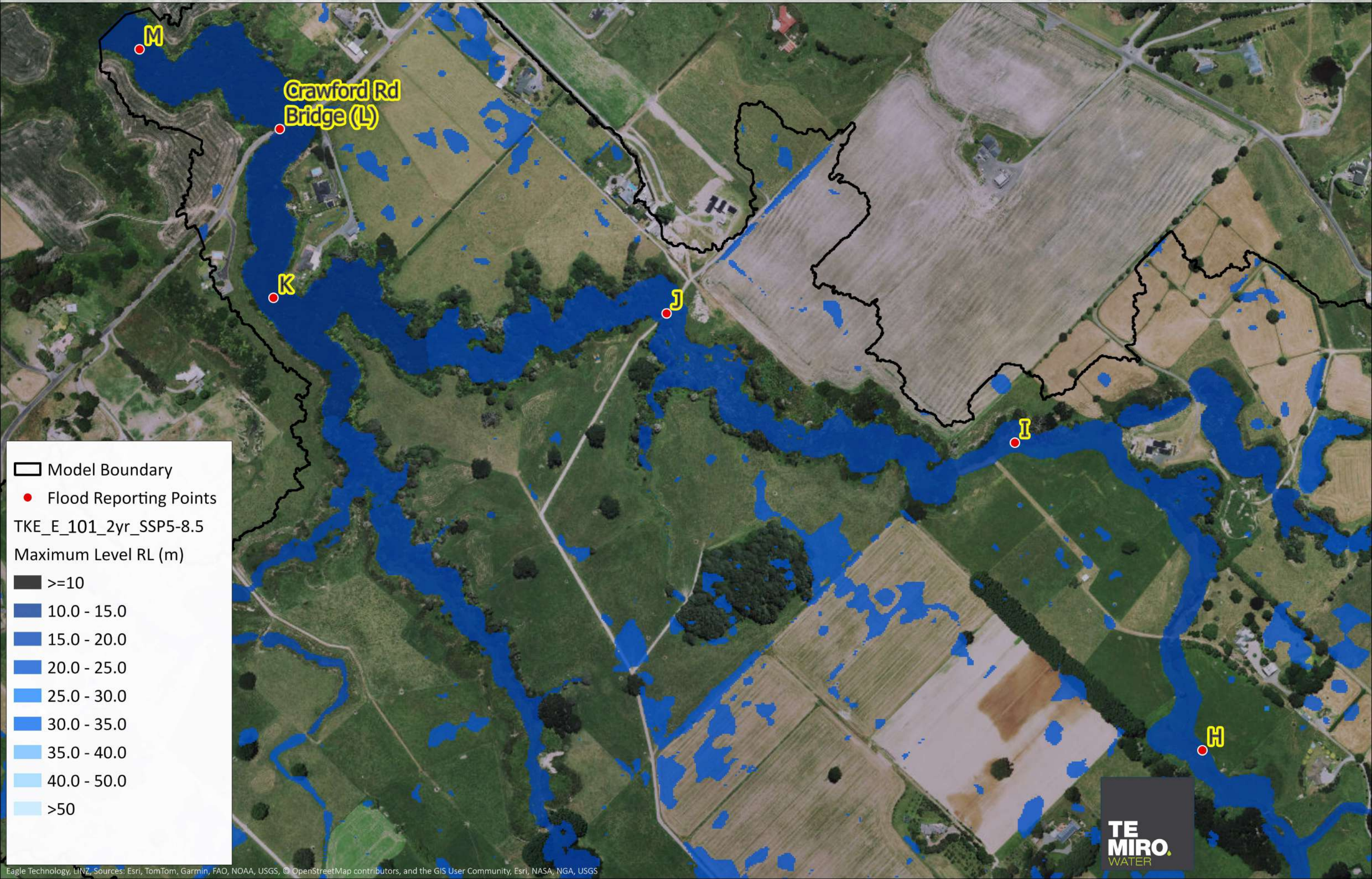
EXISTING - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

TKE_E_101_2yr_SSP5-8.5

Maximum Level RL (m)

- ≥ 10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- > 50



TE KOWHAI EAST FLOOD MODELLING

EXISTING - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

TKE_E_101_2yr_SSP5-8.5

Maximum Level RL (m)

	>=10
	10.0 - 15.0
	15.0 - 20.0
	20.0 - 25.0
	25.0 - 30.0
	30.0 - 35.0
	35.0 - 40.0
	40.0 - 50.0
	>50



TE KOWHAI EAST FLOOD MODELLING

EXISTING - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

Model Boundary

Flood Reporting Points

Site Boundary

TKE_E_101_2yr_SSP5-8.5

Maximum Level RL (m)

>=10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

40.0 - 50.0

>50



TE KOWHAI EAST FLOOD MODELLING

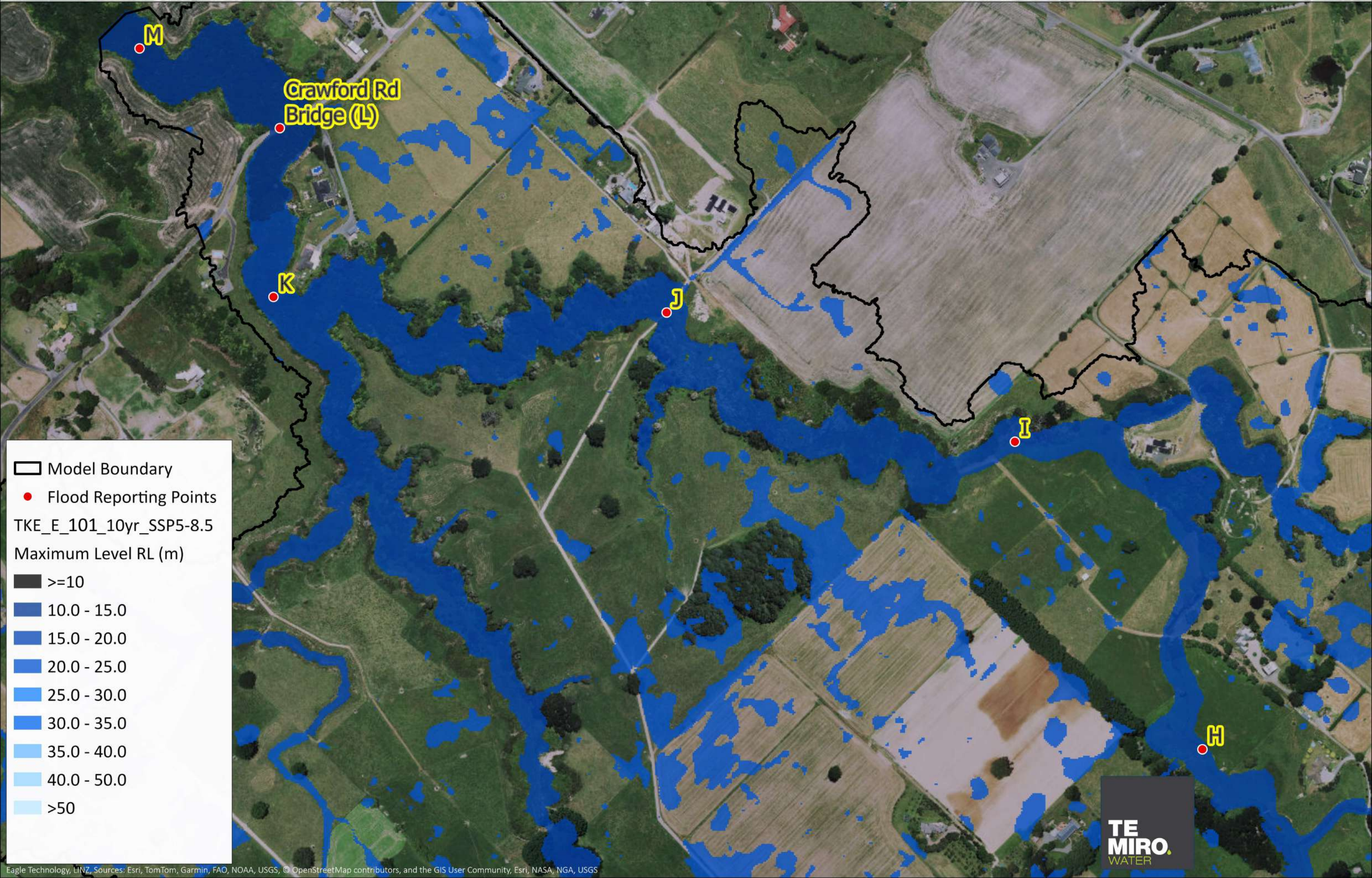
EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

TKE_E_101_10yr_SSP5-8.5

Maximum Level RL (m)

- ≥ 10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- > 50

TE
MIRO.
WATER

TE KOWHAI EAST FLOOD MODELLING

EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

TKE_E_101_10yr_SSP5-8.5

Maximum Level RL (m)

>=10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

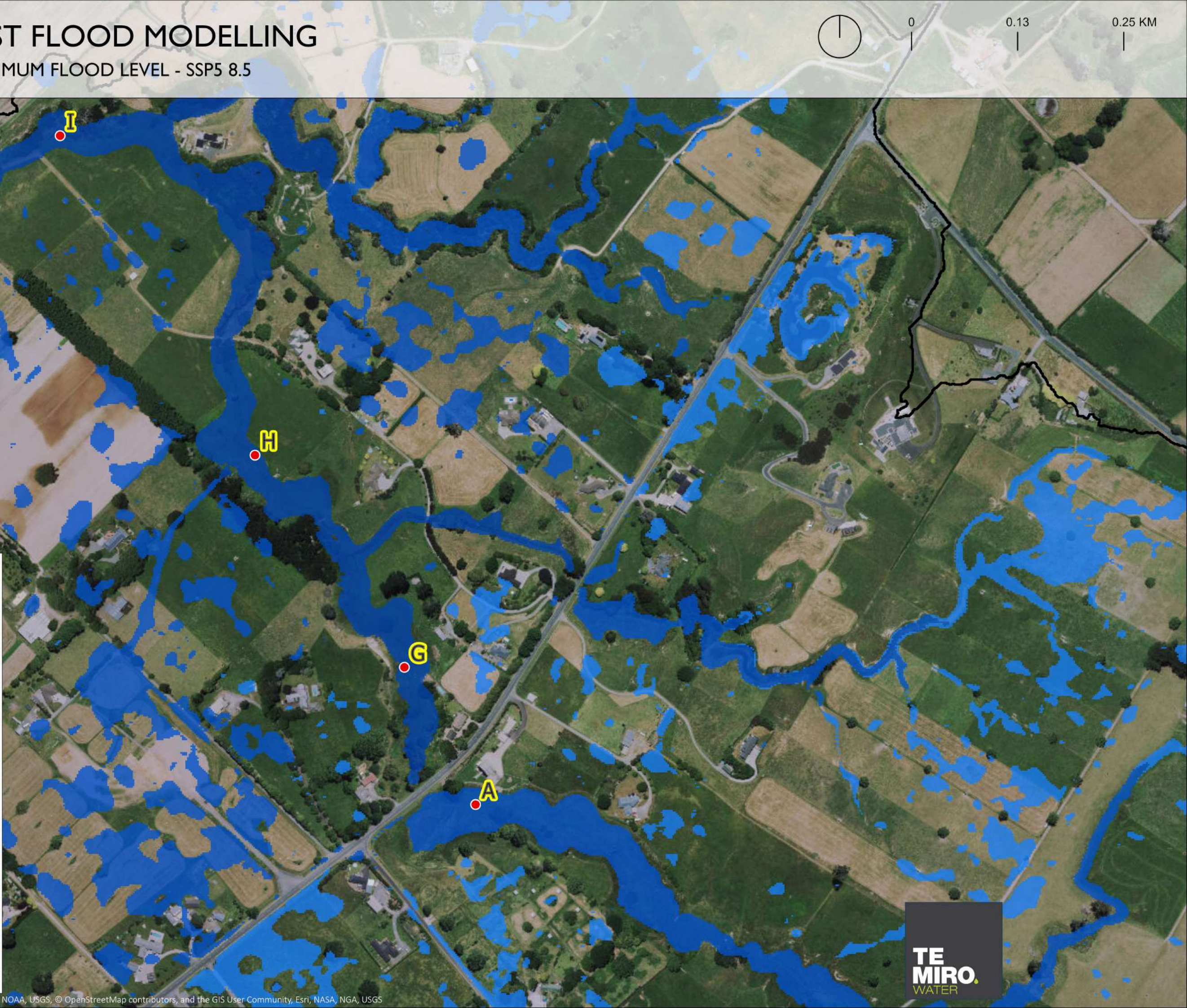
25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

40.0 - 50.0

>50



TE KOWHAI EAST FLOOD MODELLING

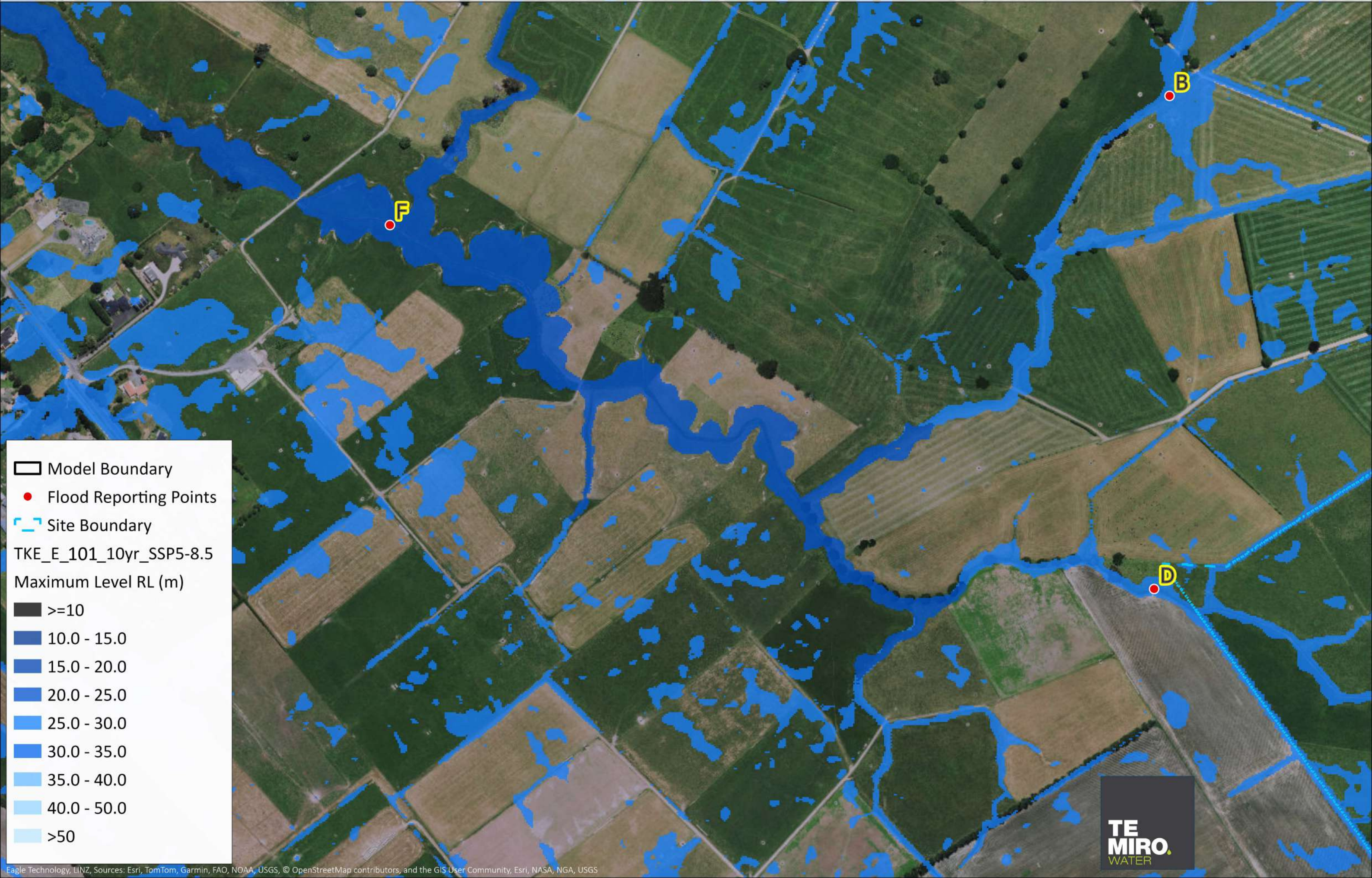
EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

Site Boundary

TKE_E_101_10yr_SSP5-8.5

Maximum Level RL (m)

≥ 10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

40.0 - 50.0

>50



TE KOWHAI EAST FLOOD MODELLING

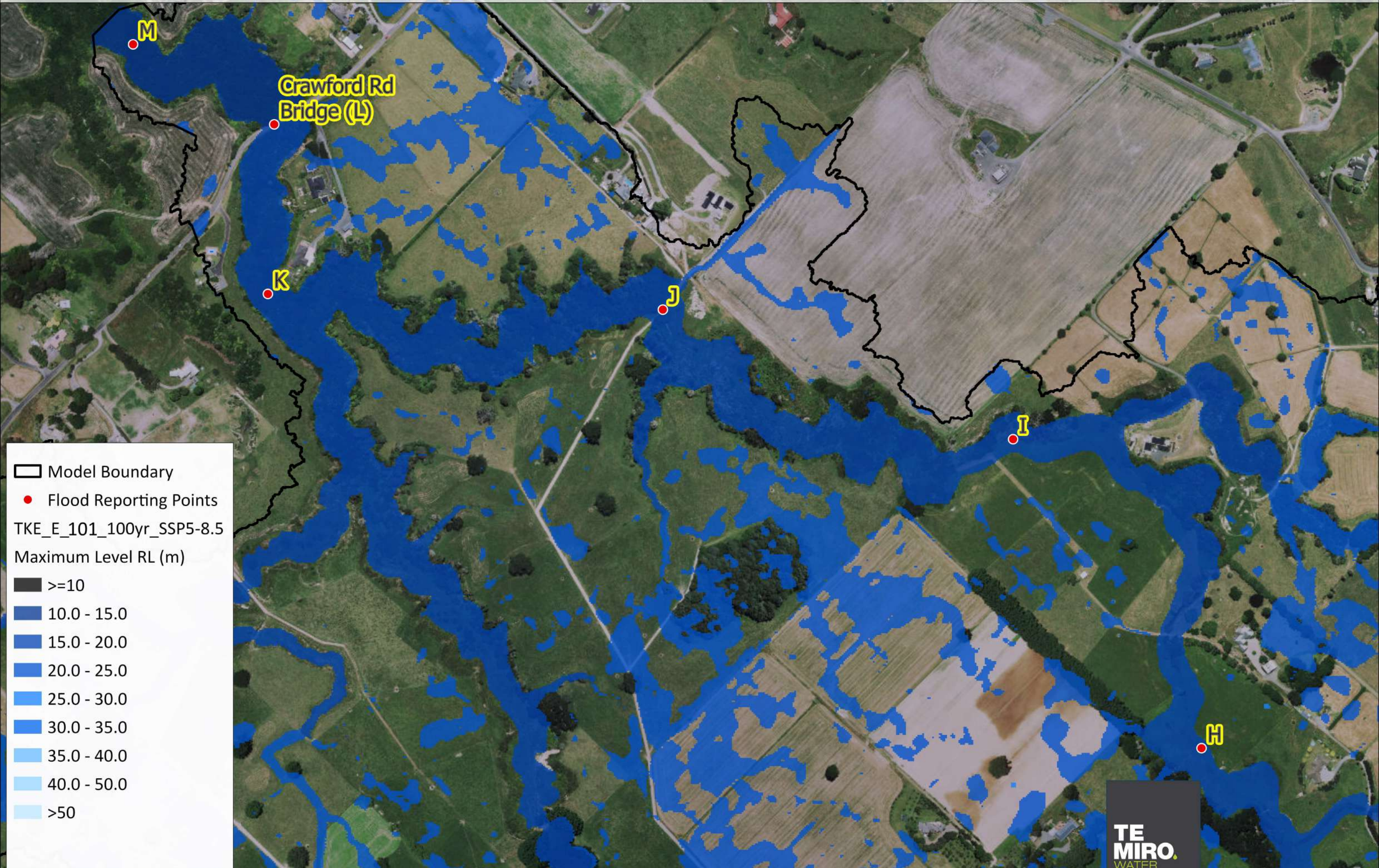
EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



- Model Boundary
- Flood Reporting Points
- TKE_E_101_100yr_SSP5-8.5
- Maximum Level RL (m)
- ≥ 10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- > 50



TE KOWHAI EAST FLOOD MODELLING

EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

Model Boundary

Flood Reporting Points

TKE_E_101_100yr_SSP5-8.5

Maximum Level RL (m)

- >=10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- >50



TE KOWHAI EAST FLOOD MODELLING

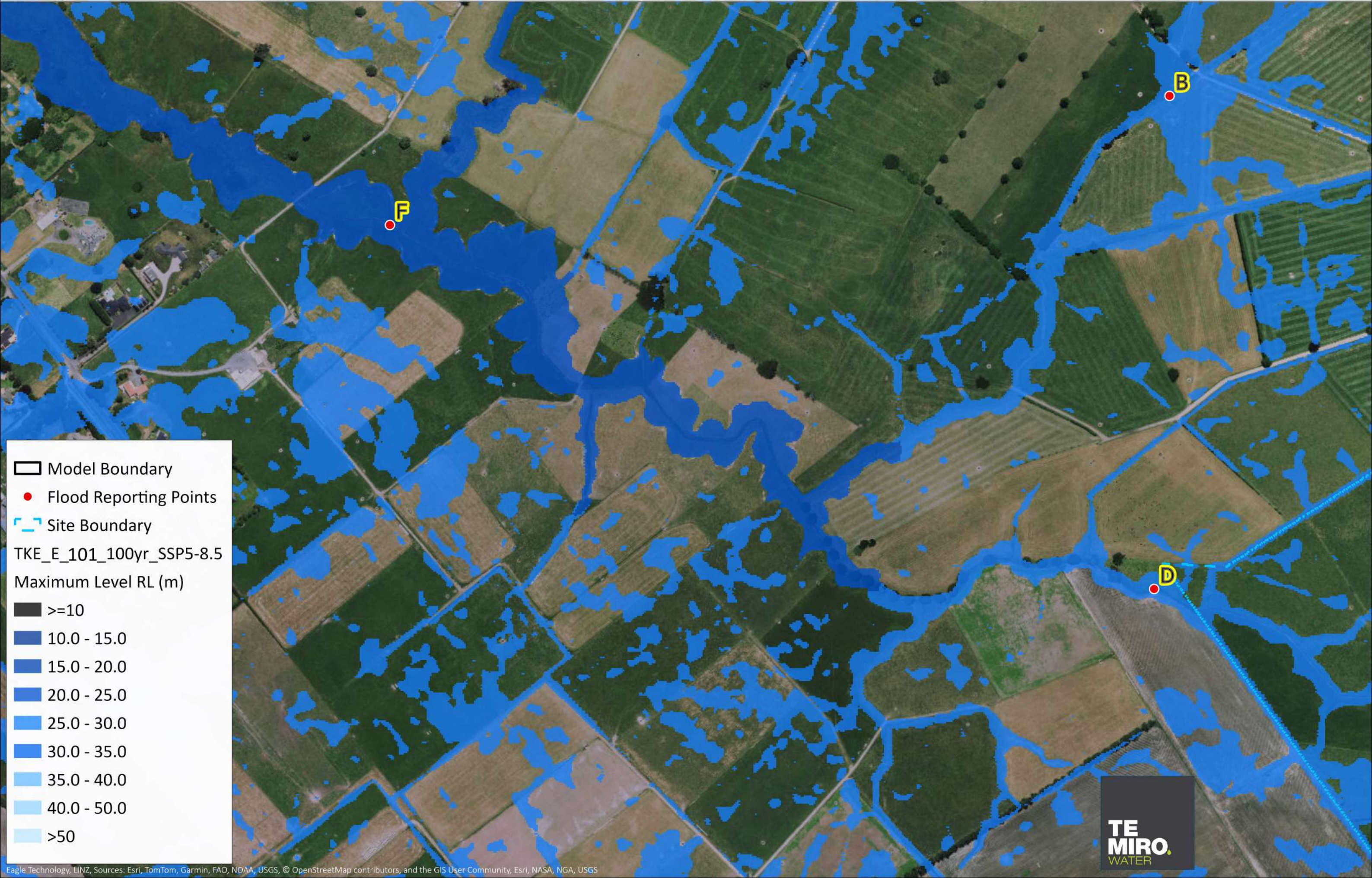
EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

Site Boundary

TKE_E_101_100yr_SSP5-8.5

Maximum Level RL (m)

≥ 10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

40.0 - 50.0

>50



Post Development Flood Levels

TE KOWHAI EAST FLOOD MODELLING

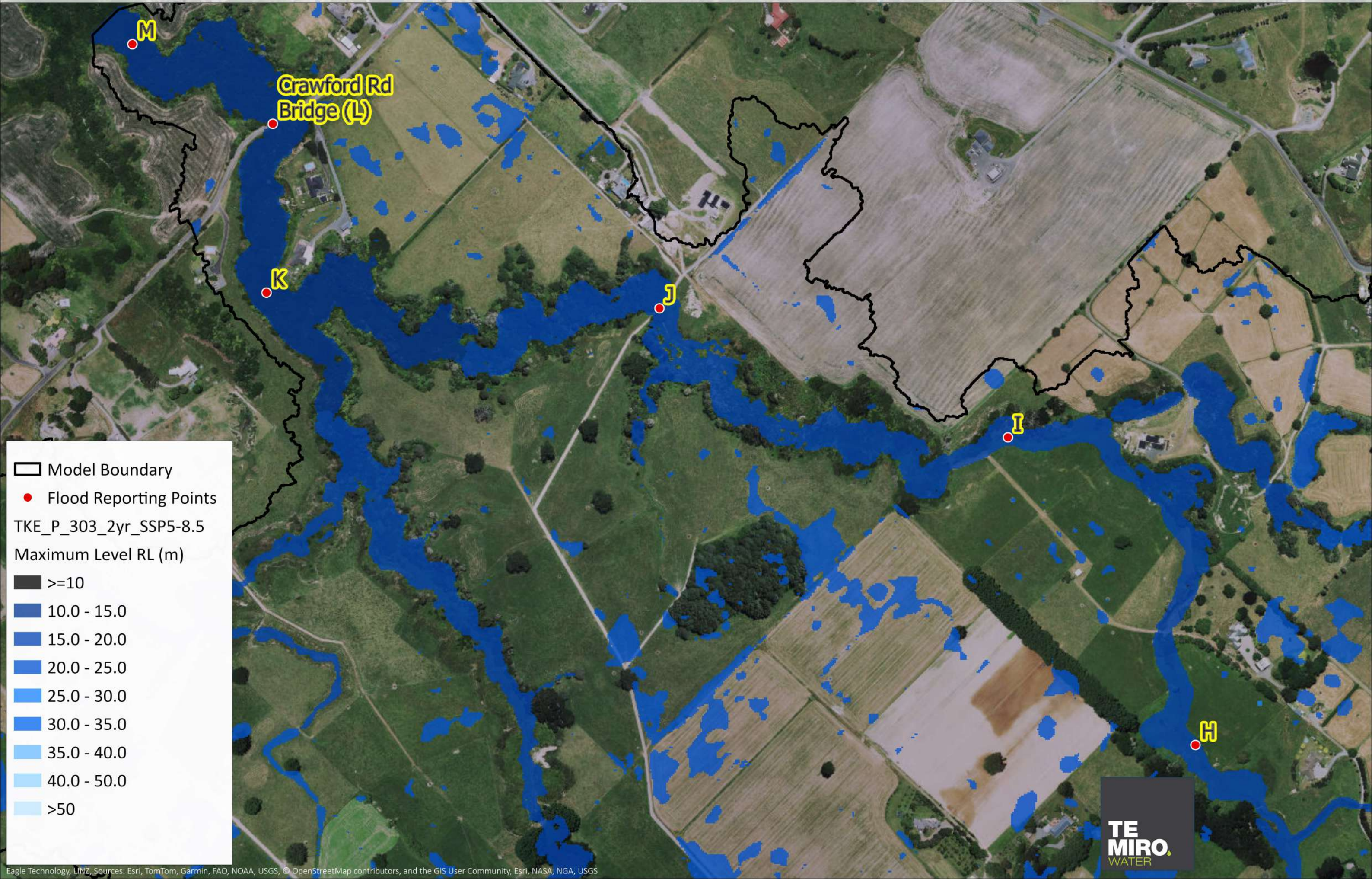
DEVELOPED - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

TKE_P_303_2yr_SSP5-8.5

Maximum Level RL (m)

- ≥ 10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- > 50



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

Model Boundary

Flood Reporting Points

TKE_P_303_2yr_SSP5-8.5

Maximum Level RL (m)

>=10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

40.0 - 50.0

>50



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

Model Boundary

Flood Reporting Points

Site Boundary

TKE_P_303_2yr_SSP5-8.5

Maximum Level RL (m)

 ≥ 10

 10.0 - 15.0

 15.0 - 20.0

 20.0 - 25.0

 25.0 - 30.0

 30.0 - 35.0

 35.0 - 40.0

 40.0 - 50.0

 >50

TE
MIRO.
WATER

TE KOWHAI EAST FLOOD MODELLING

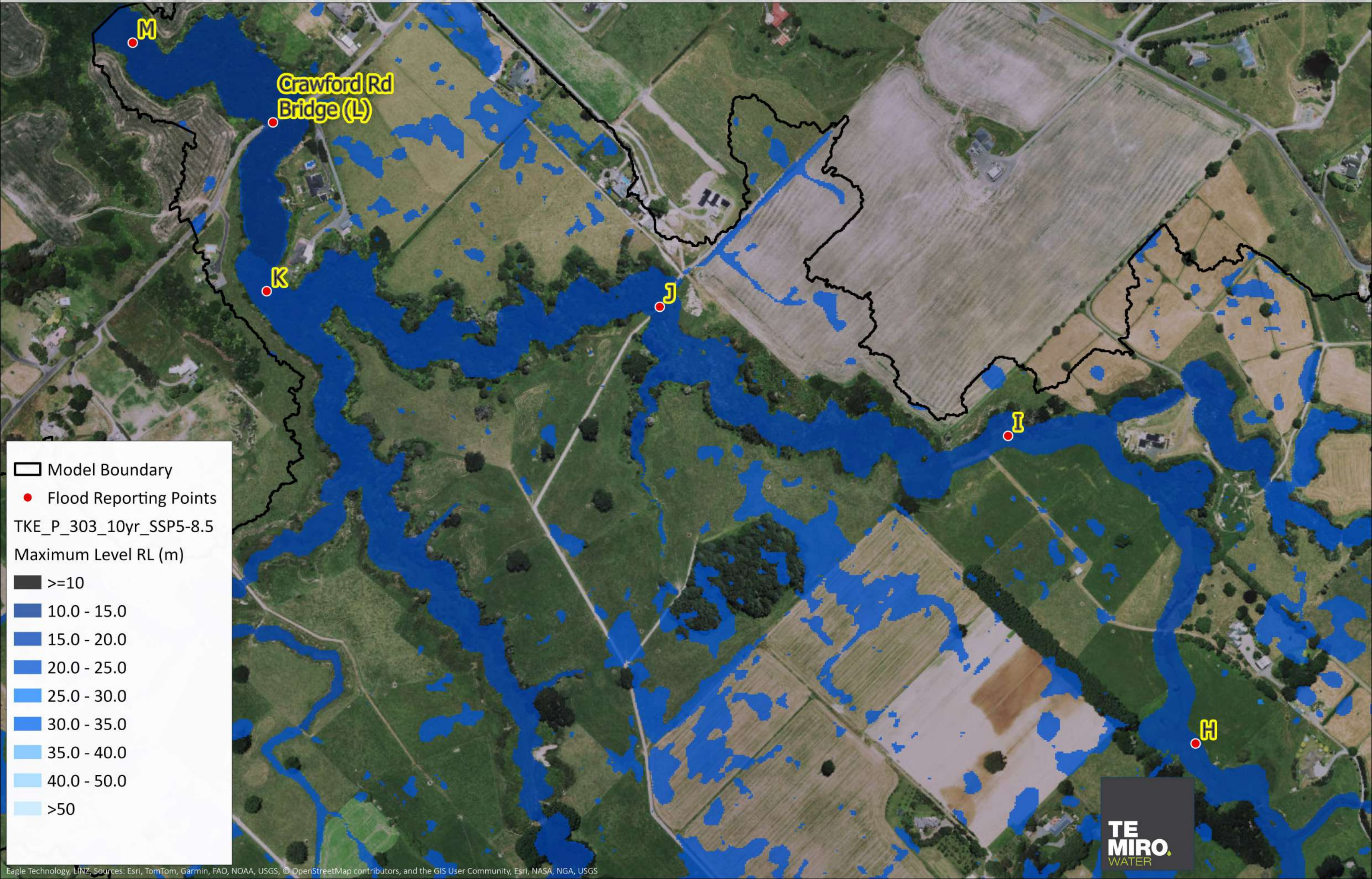
DEVELOPED - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

TKE_P_303_10yr_SSP5-8.5

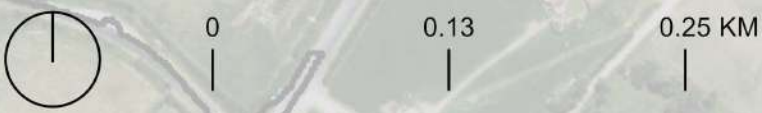
Maximum Level RL (m)

- ≥ 10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- > 50



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

TKE_P_303_10yr_SSP5-8.5

Maximum Level RL (m)

>=10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

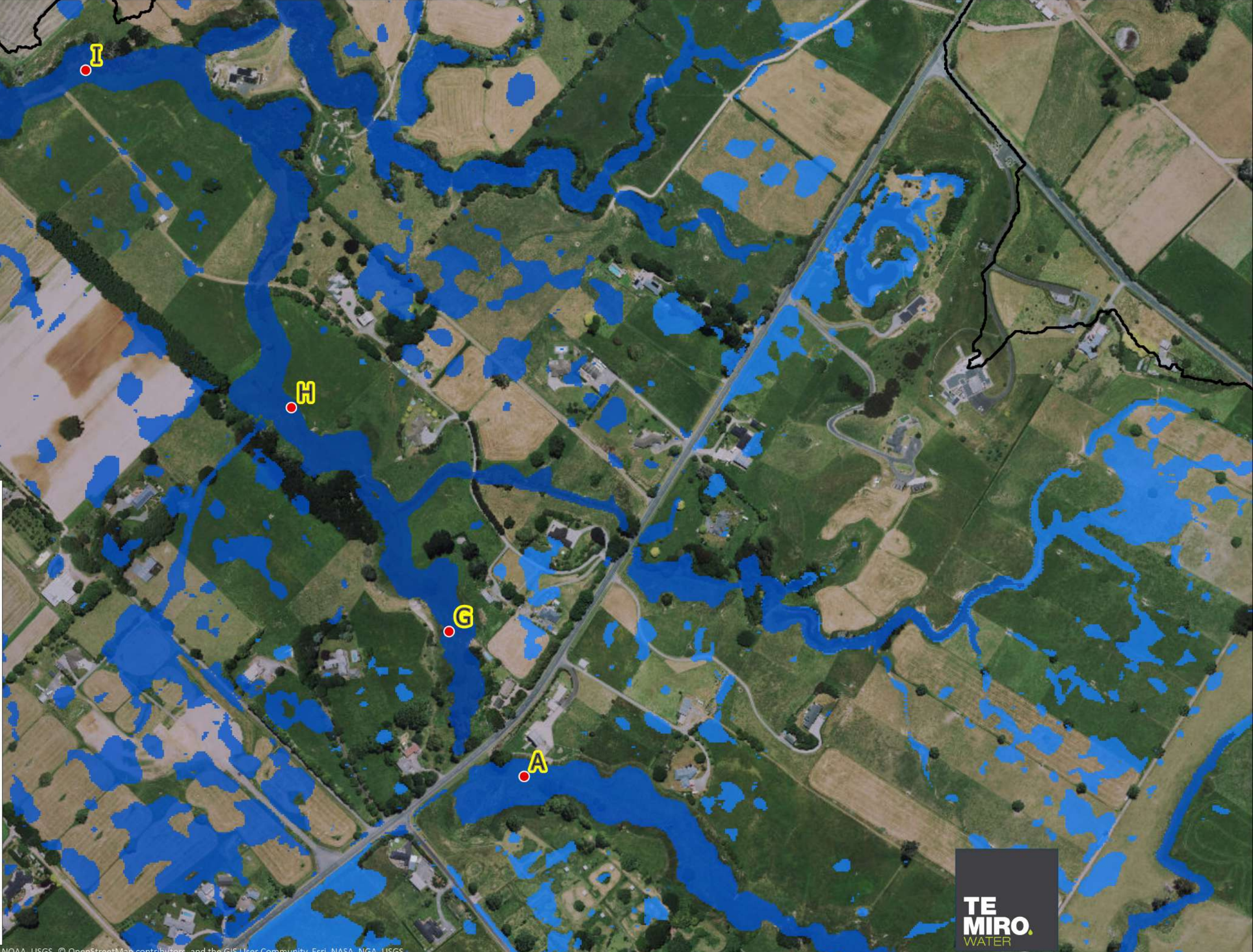
25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

40.0 - 50.0

>50



TE KOWHAI EAST FLOOD MODELLING

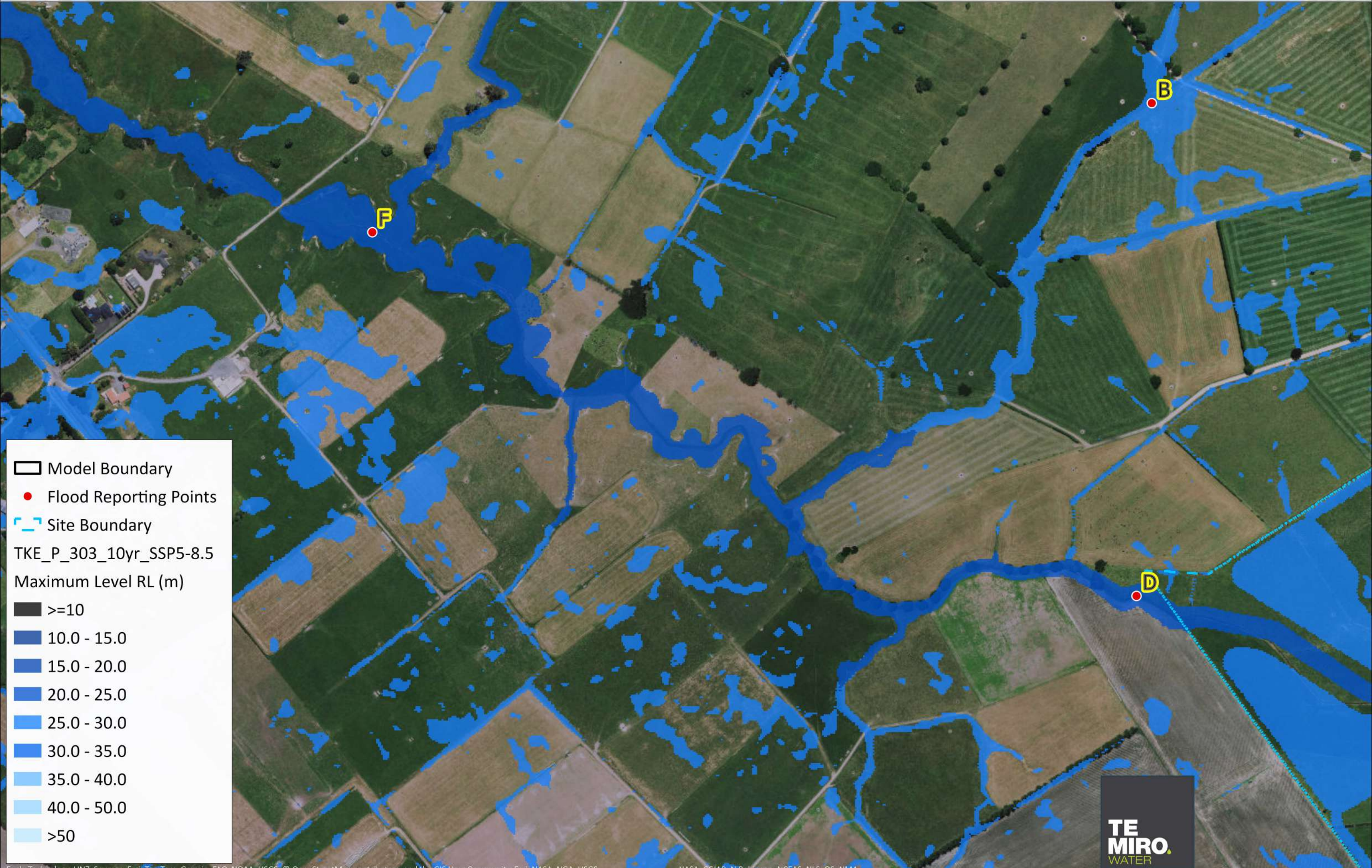
DEVELOPED - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



- Model Boundary
- Flood Reporting Points
- Site Boundary
- TKE_P_303_10yr_SSP5-8.5
- Maximum Level RL (m)
- ≥ 10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- > 50



TE KOWHAI EAST FLOOD MODELLING

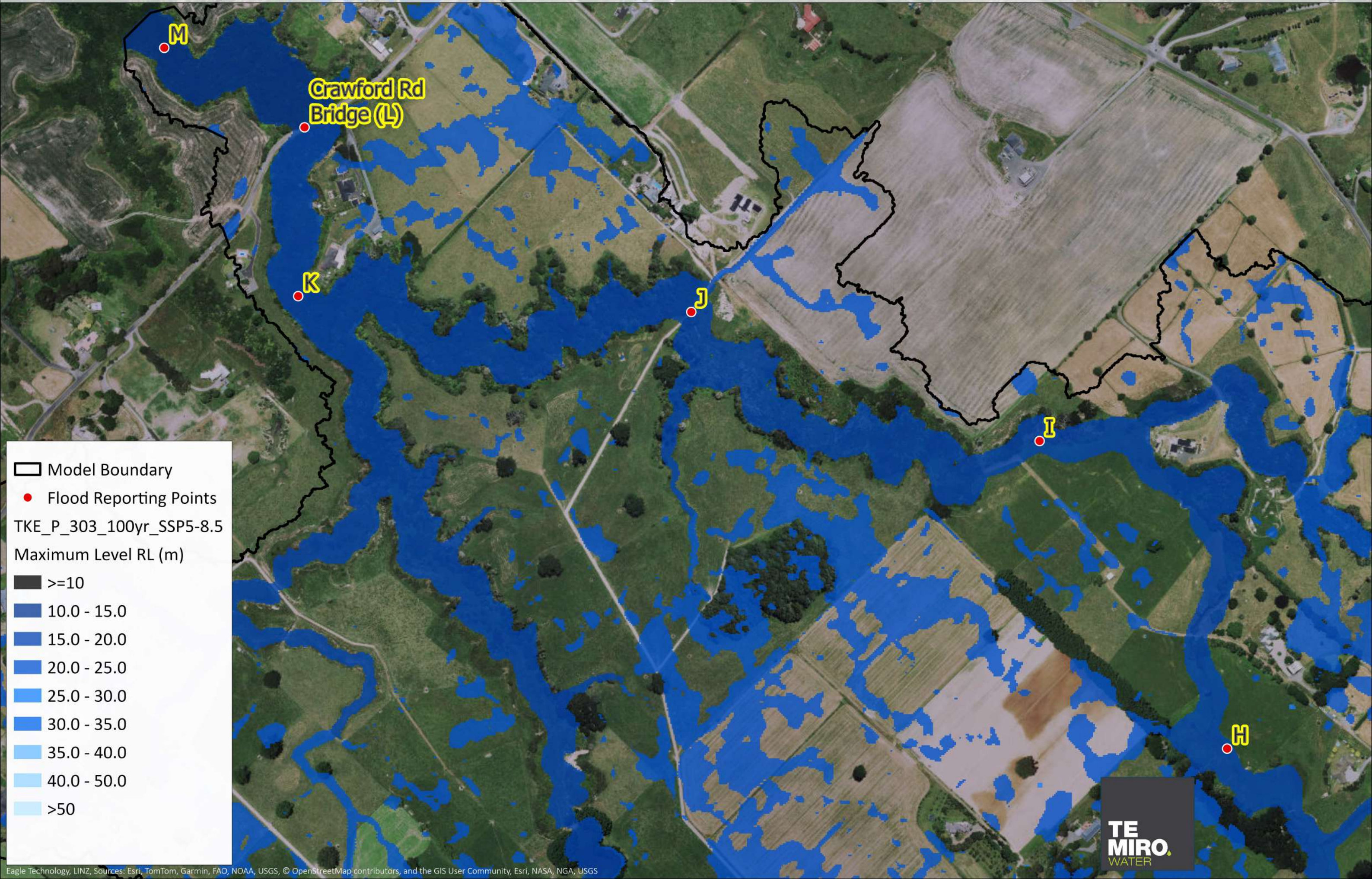
DEVELOPED - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

TKE_P_303_100yr_SSP5-8.5

Maximum Level RL (m)

>=10

10.0 - 15.0

15.0 - 20.0

20.0 - 25.0

25.0 - 30.0

30.0 - 35.0

35.0 - 40.0

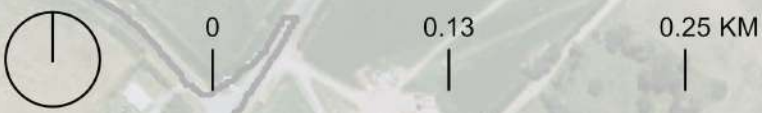
40.0 - 50.0

>50



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



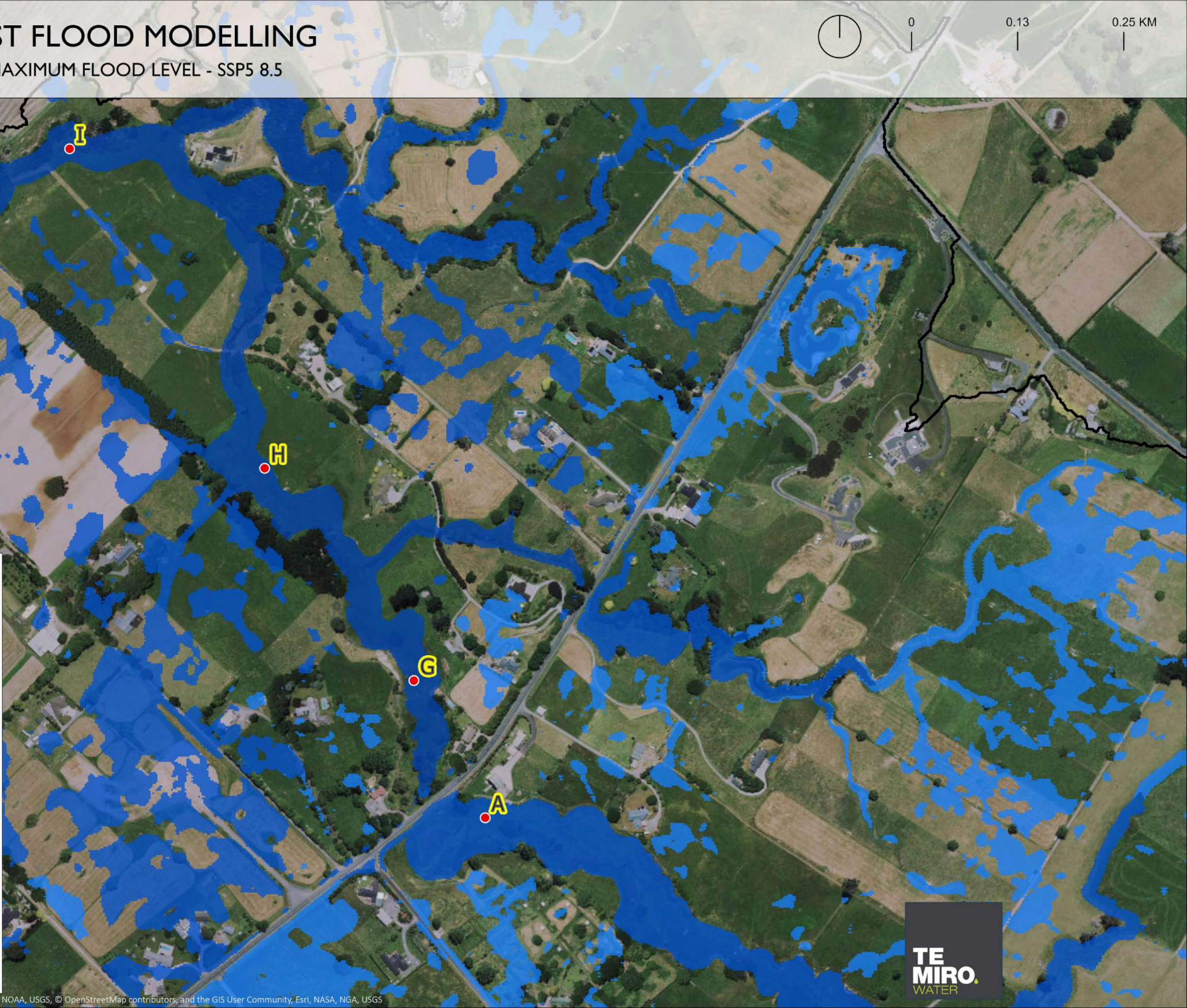
Model Boundary

Flood Reporting Points

TKE_P_303_100yr_SSP5-8.5

Maximum Level RL (m)

	>=10
	10.0 - 15.0
	15.0 - 20.0
	20.0 - 25.0
	25.0 - 30.0
	30.0 - 35.0
	35.0 - 40.0
	40.0 - 50.0
	>50



TE KOWHAI EAST FLOOD MODELLING

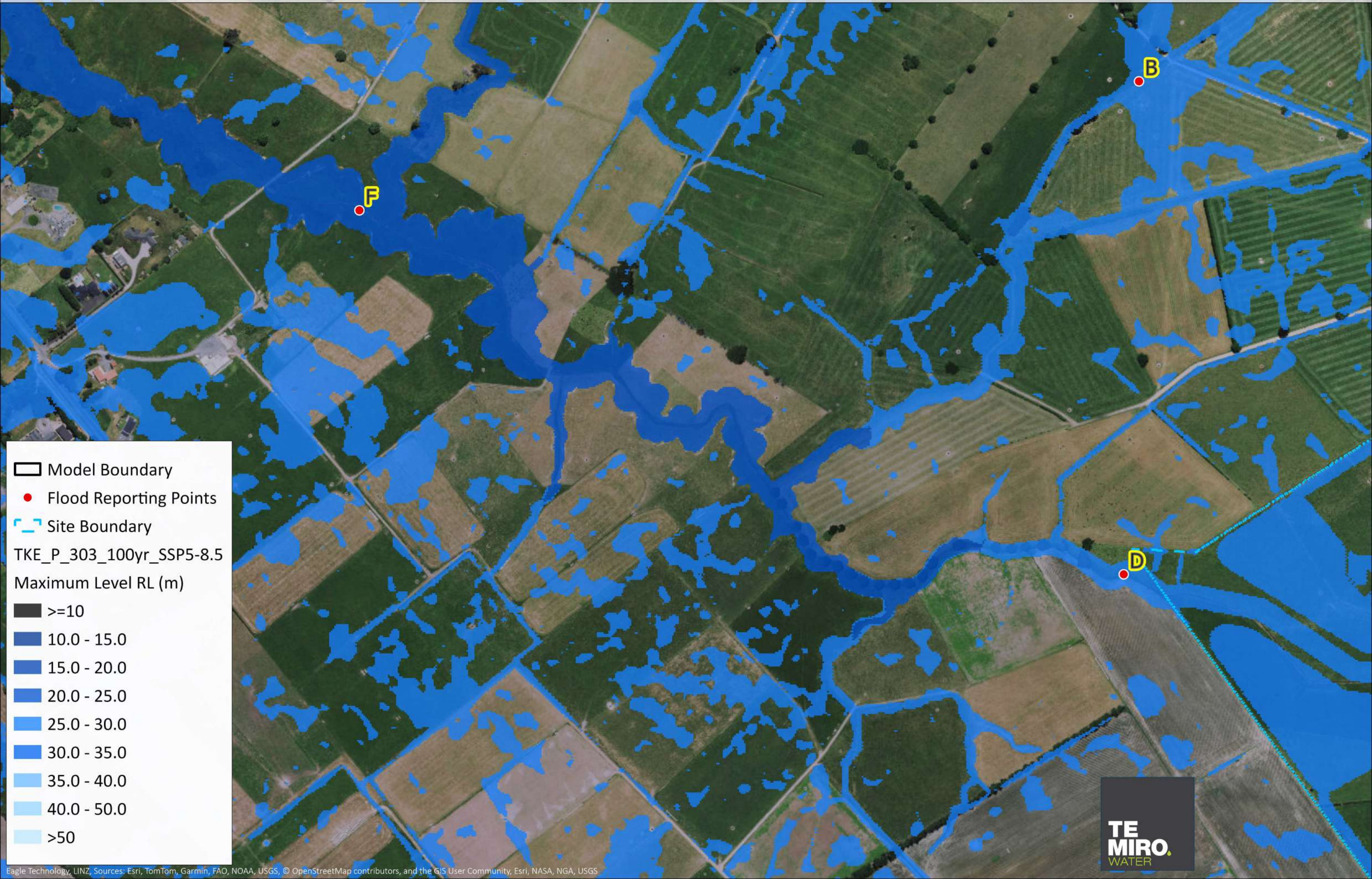
DEVELOPED - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM



Model Boundary

Flood Reporting Points

Site Boundary

TKE_P_303_100yr_SSP5-8.5

Maximum Level RL (m)

- >=10
- 10.0 - 15.0
- 15.0 - 20.0
- 20.0 - 25.0
- 25.0 - 30.0
- 30.0 - 35.0
- 35.0 - 40.0
- 40.0 - 50.0
- >50



Comparison Flood Level Maps

TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

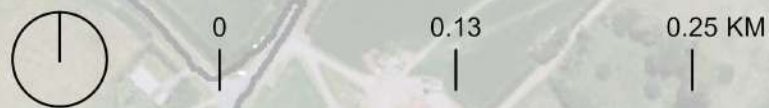


- Model Boundary
- Flood Reporting Points
- Flood Level Difference (m)
- ≤ -1.00
- 1.00 - -0.50
- 0.50 - -0.20
- 0.20 - -0.05
- 0.05 - -0.01
- 0.01 - 0.01
- 0.01 - 0.05
- 0.05 - 0.10
- 0.10 - 0.20
- 0.20 - 0.50
- >0.5
- New Wet Area
- Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5

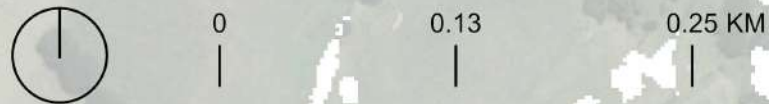


- Model Boundary
- Flood Reporting Points
- Flood Level Difference (m)
 - ≤ -1.00
 - 1.00 - -0.50
 - 0.50 - -0.20
 - 0.20 - -0.05
 - 0.05 - -0.01
 - 0.01 - 0.01
 - 0.01 - 0.05
 - 0.05 - 0.10
 - 0.10 - 0.20
 - 0.20 - 0.50
 - >0.5
- New Wet Area
- Previously Wet Area

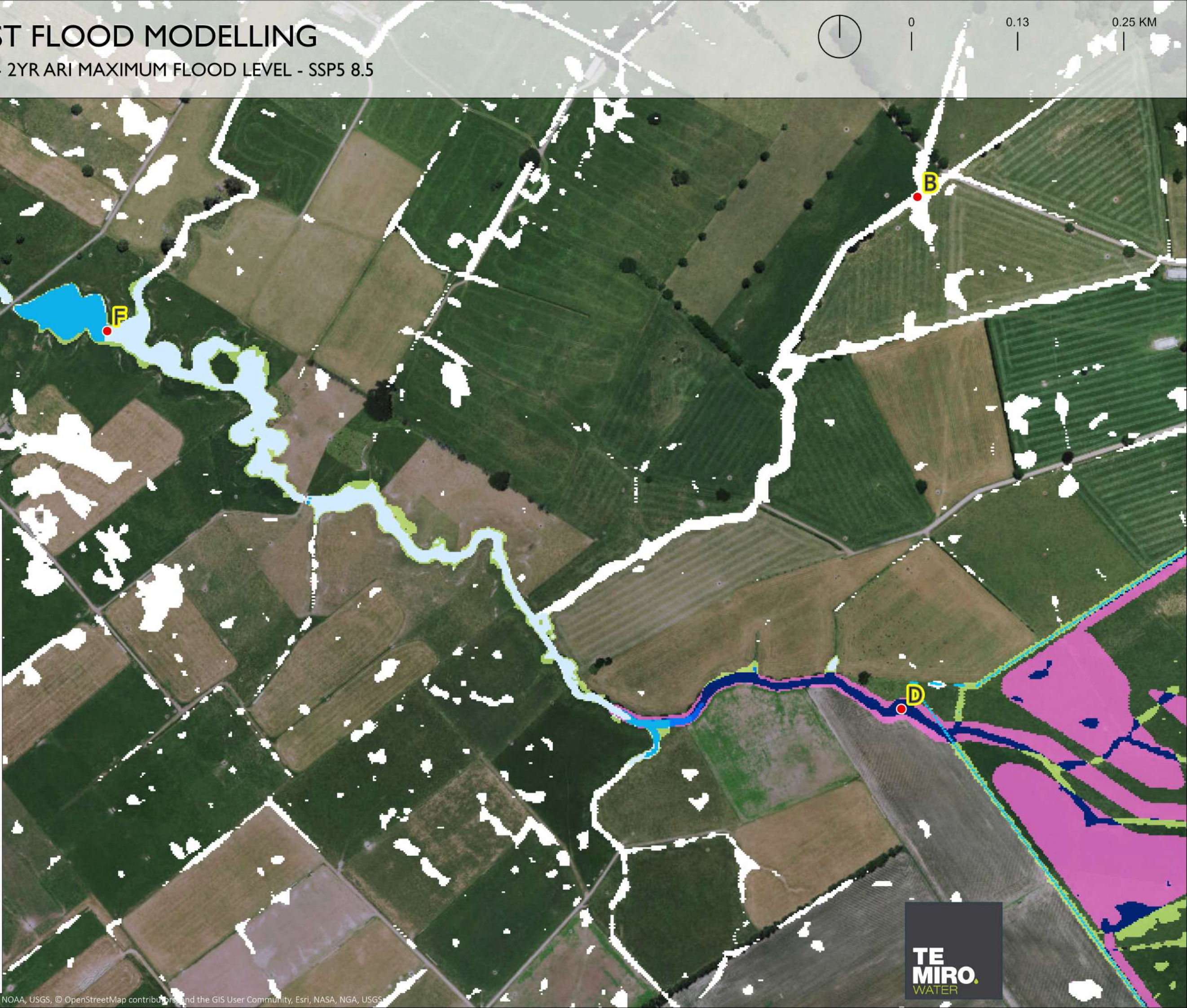


TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 2YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



- Model Boundary
- Flood Reporting Points
- Site Boundary
- Flood Level Difference (m)
 - ≤ -1.00
 - 1.00 - -0.50
 - 0.50 - -0.20
 - 0.20 - -0.05
 - 0.05 - -0.01
 - 0.01 - 0.01
 - 0.01 - 0.05
 - 0.05 - 0.10
 - 0.10 - 0.20
 - 0.20 - 0.50
 - >0.5
- New Wet Area
- Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

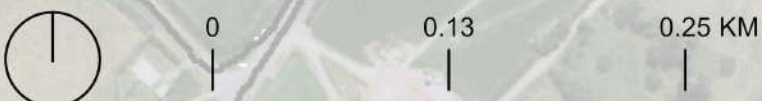


- Model Boundary
- Flood Reporting Points
- Flood Level Difference (m)
 - ≤ -1.00
 - 1.00 - -0.50
 - 0.50 - -0.20
 - 0.20 - -0.05
 - 0.05 - -0.01
 - 0.01 - 0.01
 - 0.01 - 0.05
 - 0.05 - 0.10
 - 0.10 - 0.20
 - 0.20 - 0.50
 - >0.5
- New Wet Area
- Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

Flood Level Difference (m)

<= -1.00

-1.00- -0.50

-0.50 - -0.20

-0.20 - -0.05

-0.05 - -0.01

-0.01 - 0.01

0.01 - 0.05

0.05 - 0.10

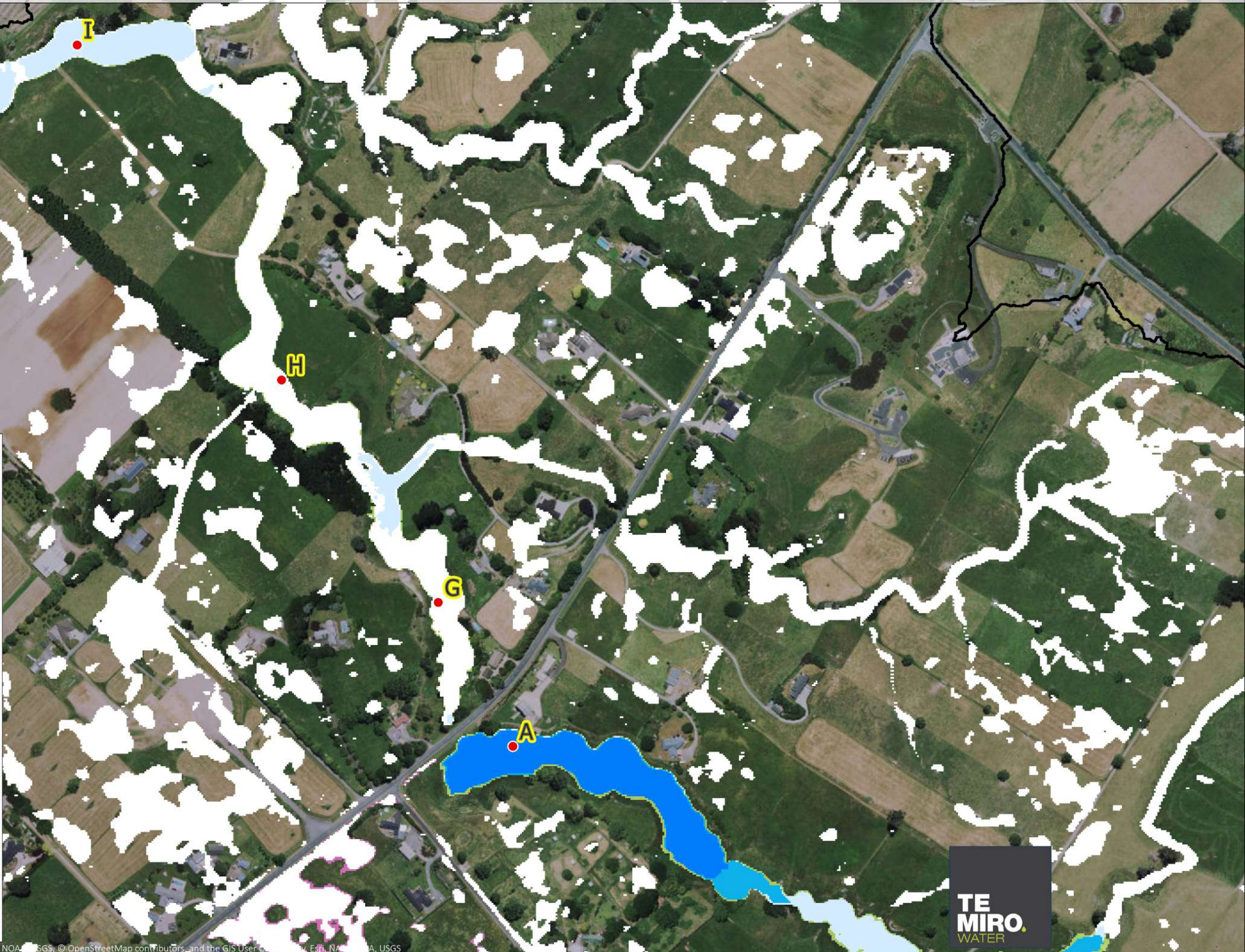
0.10 - 0.20

0.20 - 0.50

>0.5

New Wet Area

Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 10YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

Site Boundary

Flood Level Difference (m)

<= -1.00

-1.00- -0.50

-0.50 - -0.20

-0.20 - -0.05

-0.05 - -0.01

-0.01 - 0.01

0.01 - 0.05

0.05 - 0.10

0.10 - 0.20

0.20 - 0.50

>0.5

New Wet Area

Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



0

0.13

0.25 KM

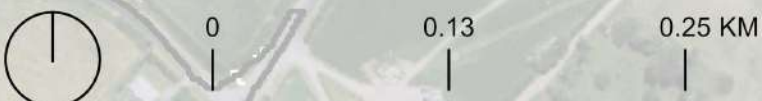


- Model Boundary
- Flood Reporting Points
- Flood Level Difference (m)
- ≤ -1.00
- 1.00 - -0.50
- 0.50 - -0.20
- 0.20 - -0.05
- 0.05 - -0.01
- 0.01 - 0.01
- 0.01 - 0.05
- 0.05 - 0.10
- 0.10 - 0.20
- 0.20 - 0.50
- >0.5
- New Wet Area
- Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

Flood Level Difference (m)

<= -1.00

-1.00 - -0.50

-0.50 - -0.20

-0.20 - -0.05

-0.05 - -0.01

-0.01 - 0.01

0.01 - 0.05

0.05 - 0.10

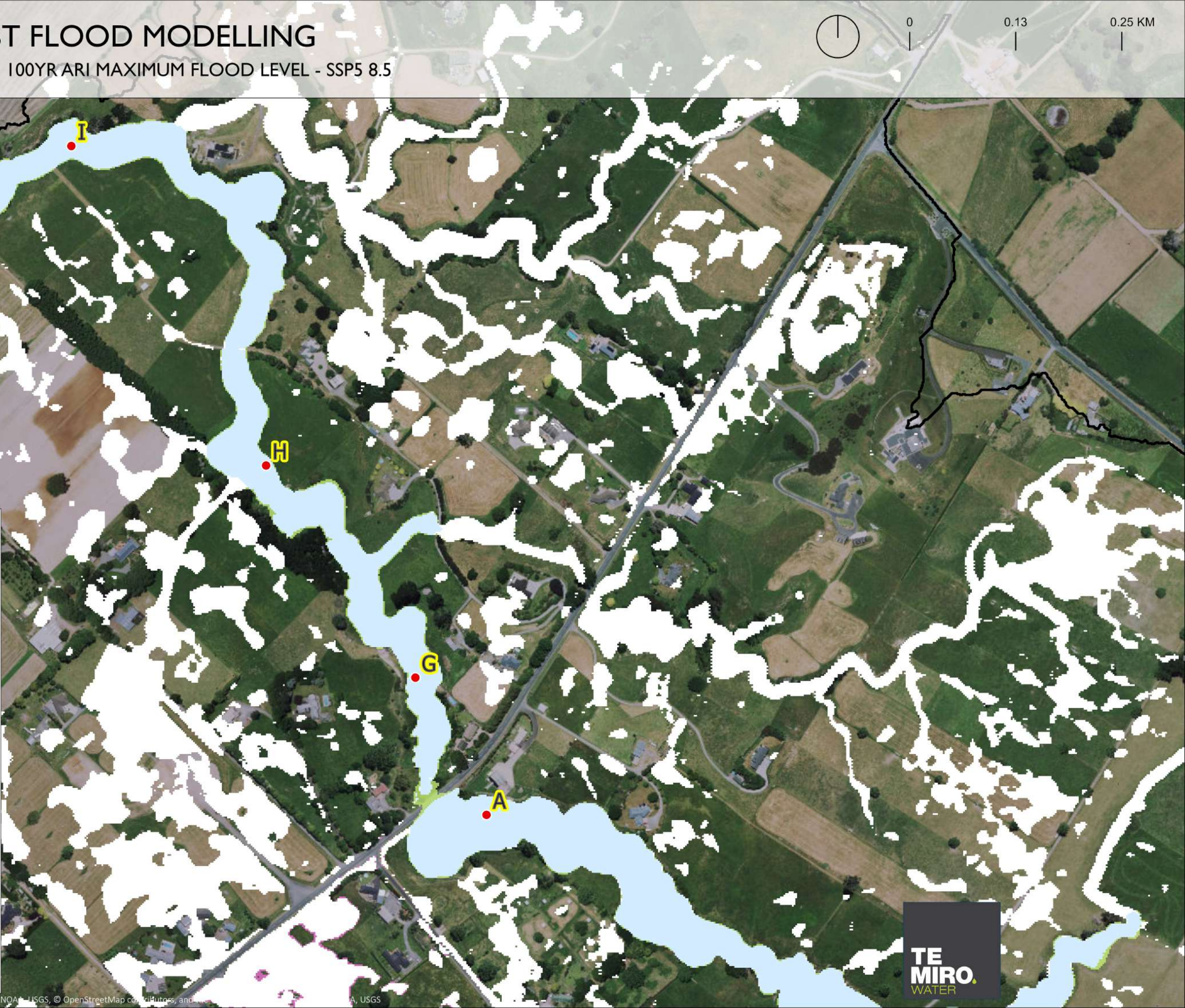
0.10 - 0.20

0.20 - 0.50

>0.5

New Wet Area

Previously Wet Area



TE KOWHAI EAST FLOOD MODELLING

DEVELOPED VS EXISTING - 100YR ARI MAXIMUM FLOOD LEVEL - SSP5 8.5



Model Boundary

Flood Reporting Points

Site Boundary

Flood Level Difference (m)

<= -1.00

-1.00- -0.50

-0.50 - -0.20

-0.20 - -0.05

-0.05 - -0.01

-0.01 - 0.01

0.01 - 0.05

0.05 - 0.10

0.10 - 0.20

0.20 - 0.50

>0.5

New Wet Area

Previously Wet Area



89 Degrees Limited

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89 Degrees Reference: NZ10059-P01-01
August 13, 2026

James Oakley
Principal Three Waters Engineer
Te Miro Water Consultants
Suite 7, 3 Empire Place
Cambridge, 3434

SUBJECT: Te Kowhai East TUFLOW Model – Independent Technical Peer Review

Dear James,

Thank you for engaging 89 Degrees Limited to undertake an independent technical peer review of the TUFLOW model developed for the proposed Te Kowhai East development. The review considered the pre- and post-development model configurations and their application to the 2-, 10- and 100-year ARI design rainfall events.

The detailed findings are provided in the accompanying **Te Kowhai East TUFLOW Model – Peer Review Register**. This letter summarises the overall review outcomes and the principal matters requiring attention.

Overall Review Conclusion

Overall, the model is considered suitable for its intended application, subject to the matters identified in the review being appropriately addressed. No issues have been identified that fundamentally undermine the model methodology or its use for the intended pre- and post-development assessment.

Review of the model results indicates generally coherent hydraulic behaviour and good numerical performance. Flow paths, depths, water levels, velocities and the operation of representative hydraulic structures were reviewed and found to be generally consistent with the expected hydraulic response.

Comparison of the pre- and post-development results for the 100-year ARI event demonstrates attenuation of peak flows, with a smoother and more distributed downstream hydrograph response consistent with the proposed storage and flow-control system. Changes in maximum

depths, water levels and velocities are principally associated with the areas modified by the proposed development, including the proposed wetland and storage areas. No material adverse effects were identified outside the areas modified by the proposed development from the results reviewed.

The review findings are detailed in the accompanying review register and include matters requiring specific assessment or response, together with recommendations for clarification, documentation or improvement. Three key model-implementation matters are highlighted below.

You are requested to address the relevant findings in the 'Modelling Consultant Response' column of the review register. We will then update the review status following consideration of your response and any model amendments undertaken.

Key Review Matters

MIN/GULLY terrain treatment with SGS

MIN/GULLY terrain-modification lines are used at a number of locations in conjunction with TUFLOW Sub-Grid Sampling (SGS). Review of the model inputs and check outputs indicates local saw-tooth effects associated with this treatment.

The affected locations should be assessed to confirm that the terrain representation is appropriate and is not materially influencing the modelled hydraulic response. Alternative terrain-modification approaches should be considered where warranted.

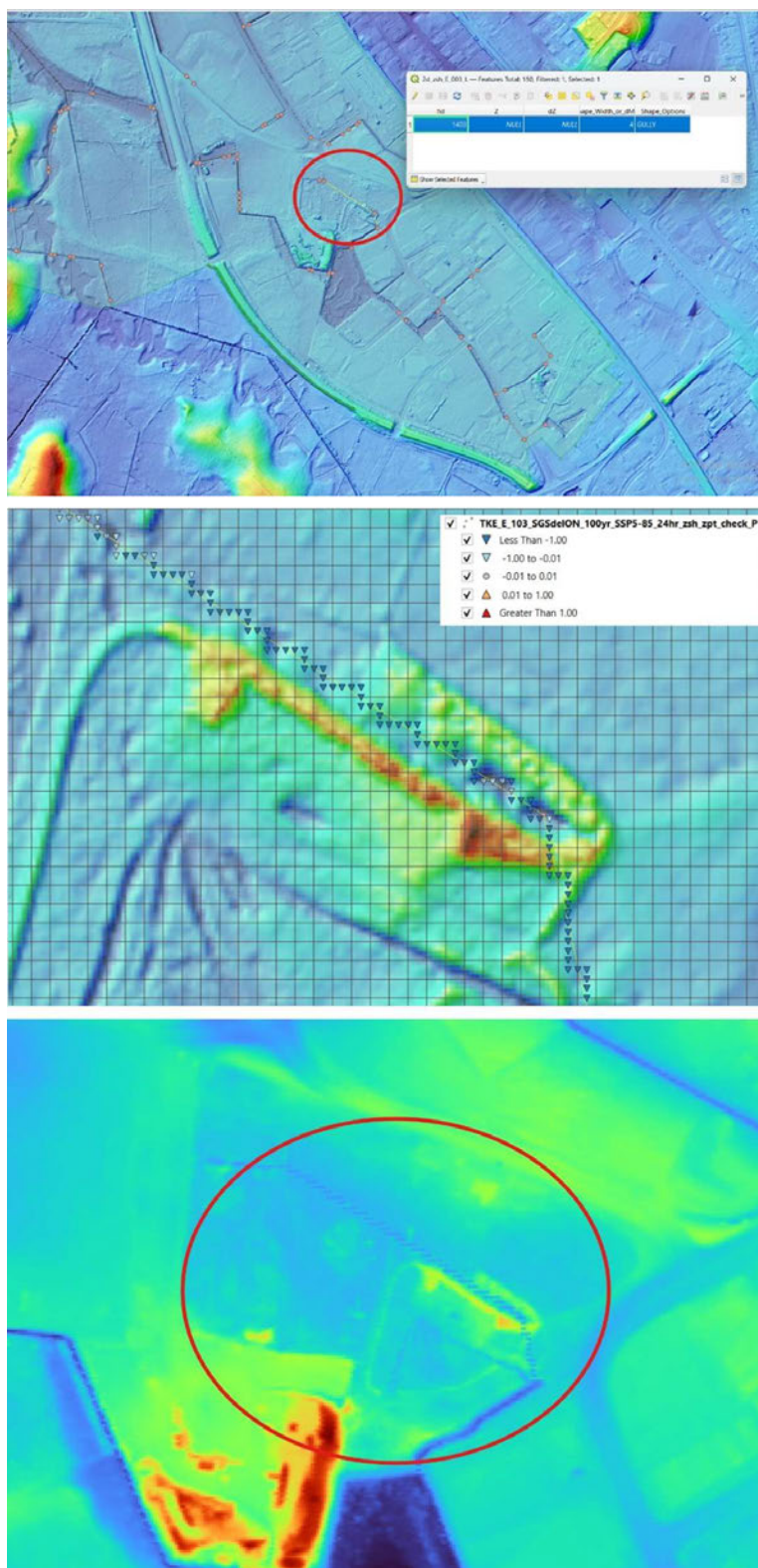


Figure 1. Example of MIN/GULLY terrain-modification treatment and associated zig-zag/saw-tooth representation with SGS (Review Items B4.4).

SX connections and Z-flag adjustments

A number of 1D/2D SX connections use the Z flag, with some relatively large adjustments between the 1D connection levels and underlying 2D terrain identified in the model messages and check outputs.

The affected connections should be assessed to confirm that the adopted connection geometry and elevations appropriately represent the intended hydraulic behaviour. This assessment should also consider the localised oscillatory behaviour identified at some culvert connections and whether refinement of local terrain, connection levels or available 2D storage is warranted.

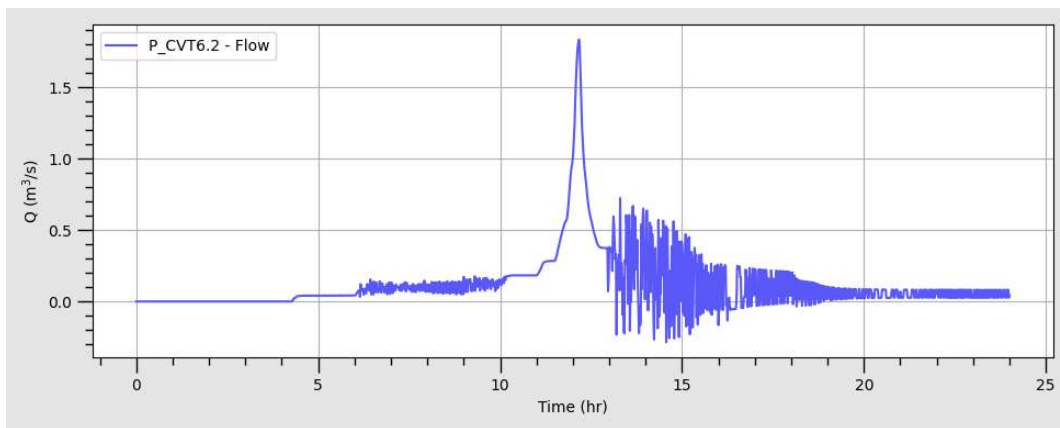


Figure 2. Example of localised oscillatory culvert-flow behaviour at an SX-coupled structure, relevant to review of the SX/Z-flag connection treatment (Review Items B5.3 and C1.5).

Initial water levels

The adopted initial water levels provide the starting hydraulic condition for the model and may influence the early simulation response and available storage within the drainage system. The appropriateness of the adopted initial water levels should therefore be confirmed, including their consistency with the intended antecedent conditions for the design-event assessment.

Review Scope and Limitations

The review has considered the overall model schematisation, detailed model implementation, hydraulic structures and 1D/2D coupling, terrain representation, materials and rainfall-loss representation, boundary conditions and design-event inputs, model check files, simulation messages, numerical performance, hydraulic behaviour and representative pre- and post-development assessment results.

A final modelling report has not been provided as part of this review. Accordingly, the adequacy of the final documentation, presentation and interpretation of the model results and assessment conclusions has not been reviewed.

The review represents an independent technical assessment of the model based on the information made available for review. It has not involved independent redevelopment of the model or independent verification of every input dataset, parameter or modelling assumption.

Ultimate responsibility for the model configuration, assumptions, application, results and interpretation remains with the modelling consultant. The peer review does not transfer or diminish that responsibility.

Figures supporting the findings in the review register are presented on the following pages.

If you have any points you wish to discuss, please do not hesitate to contact me.

Sincerely,

A handwritten signature in black ink that reads "Colin Roberts". The signature is written in a cursive, flowing style.

Colin Roberts

Director



Review Figures

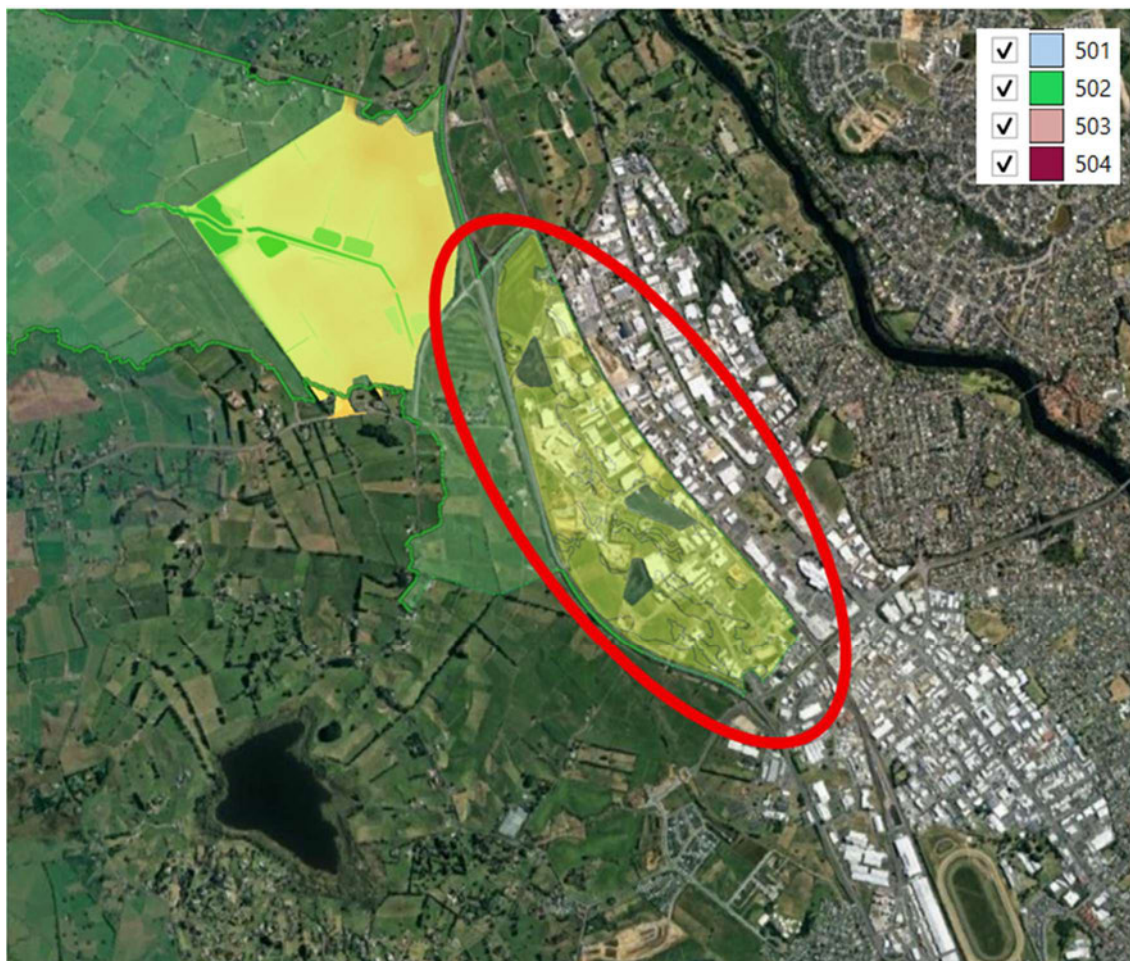


Figure 3. Difference in adopted soil-loss parameters between the pre- and post-development models (Review Item B7.7).

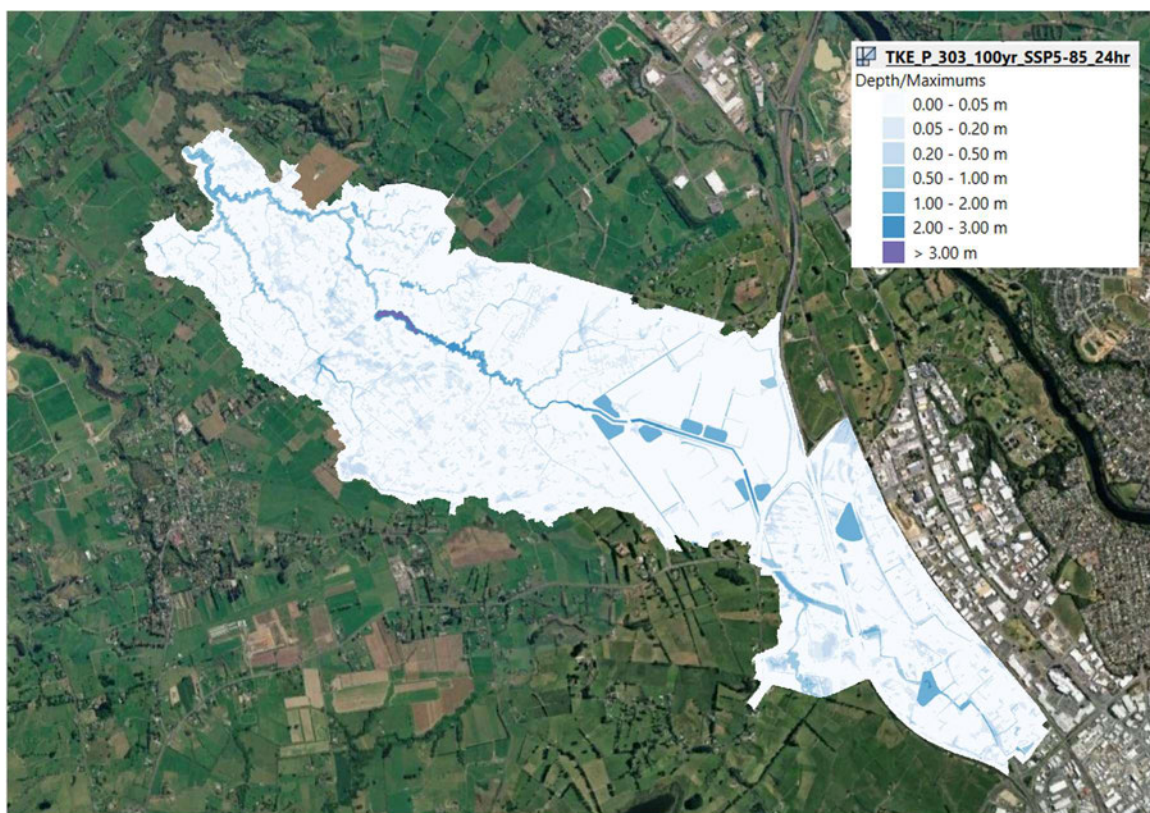


Figure 4. Maximum modelled water depth - post-development 100-year ARI event (Review Item C2.2).

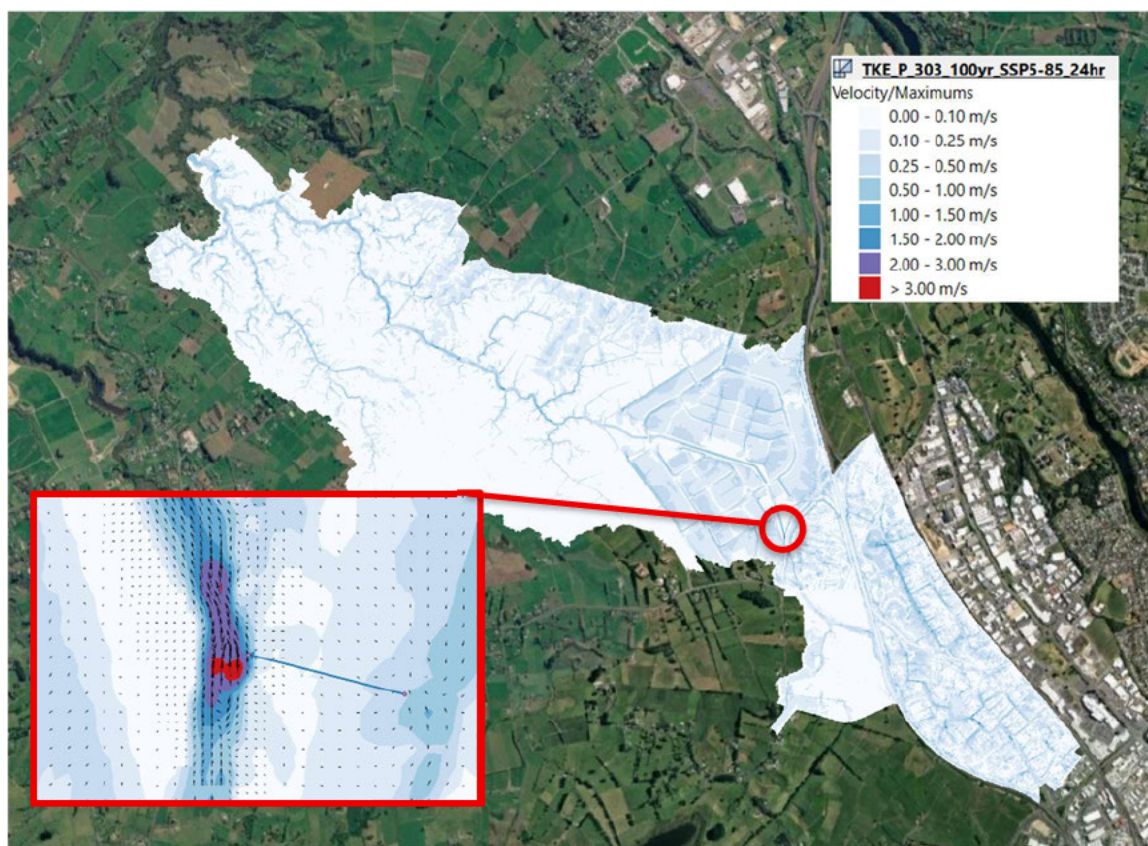


Figure 5. Maximum modelled velocity - post-development 100-year ARI event, including a representative localised high-velocity area (Review Item C2.3).

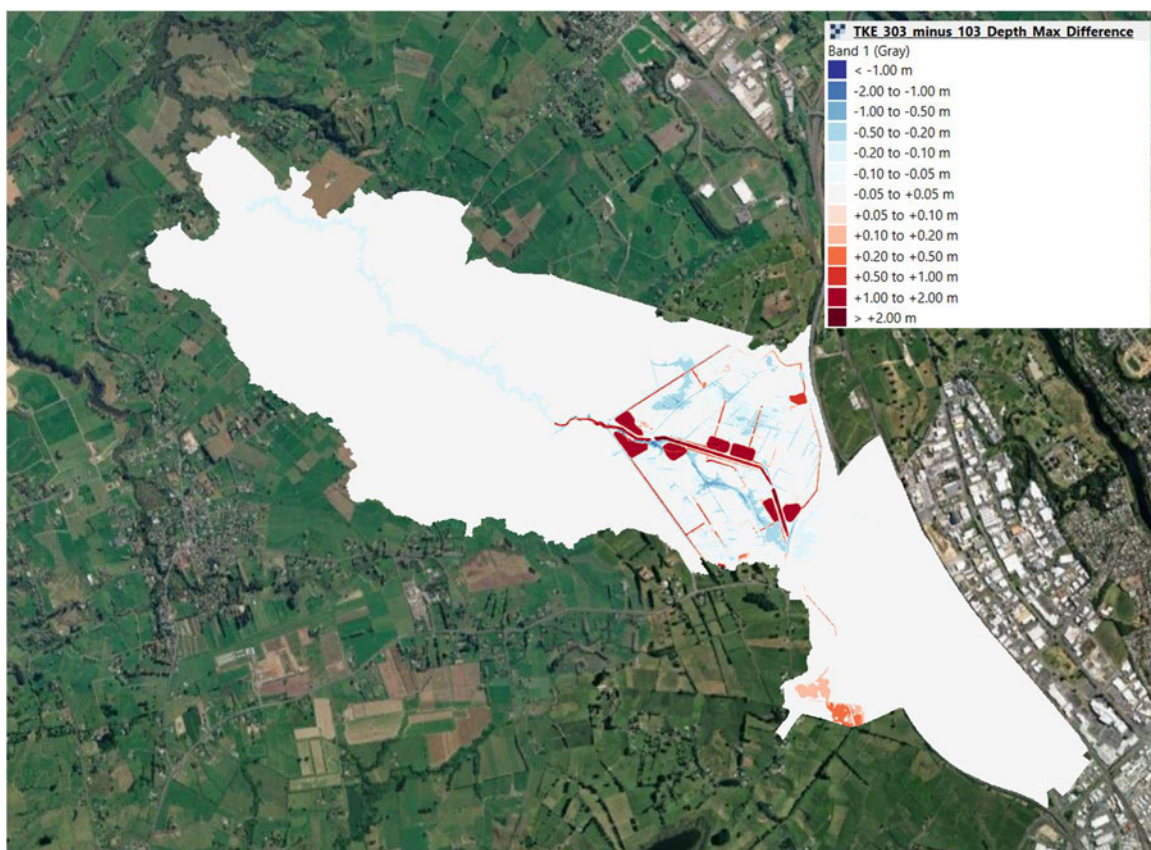


Figure 6. Difference in maximum water depth (post-development minus pre-development) - 100-year ARI event (Review Item C4.2).

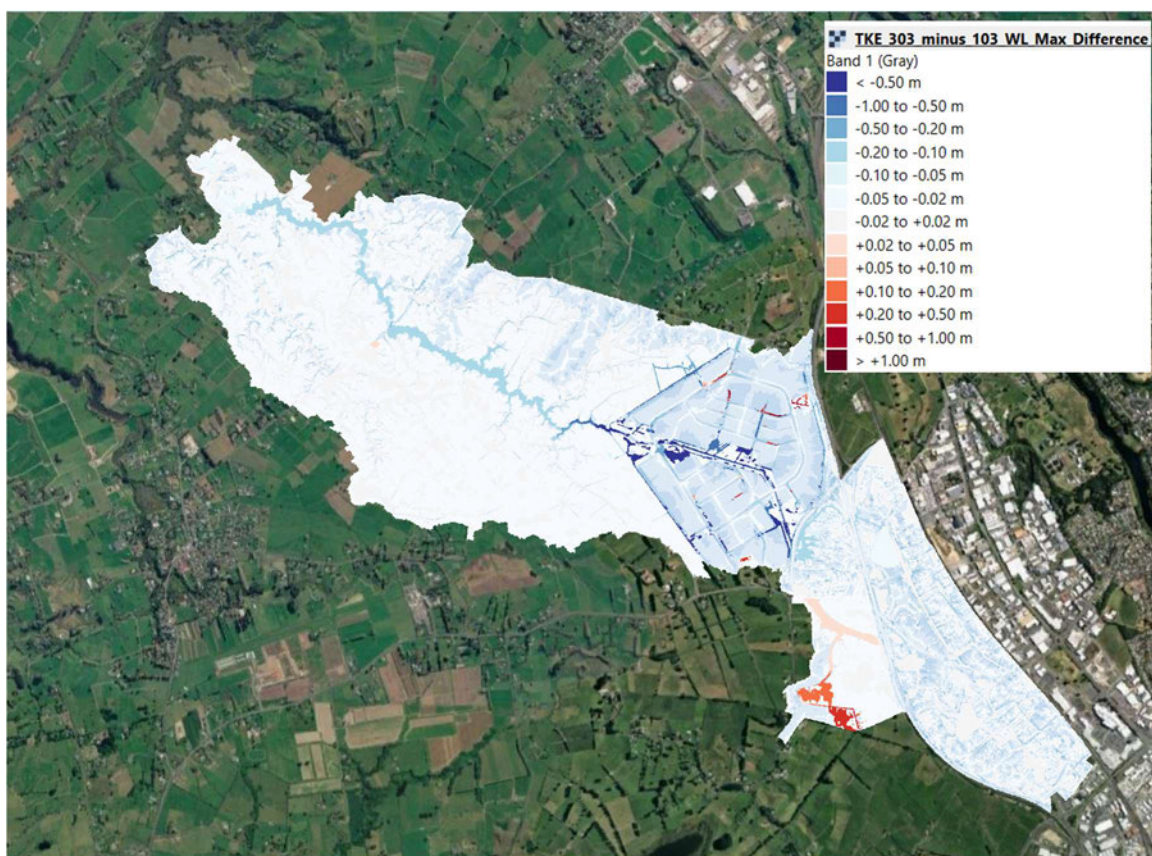


Figure 7. Difference in maximum water level (post-development minus pre-development) - 100-year ARI event (Review Item C4.3).

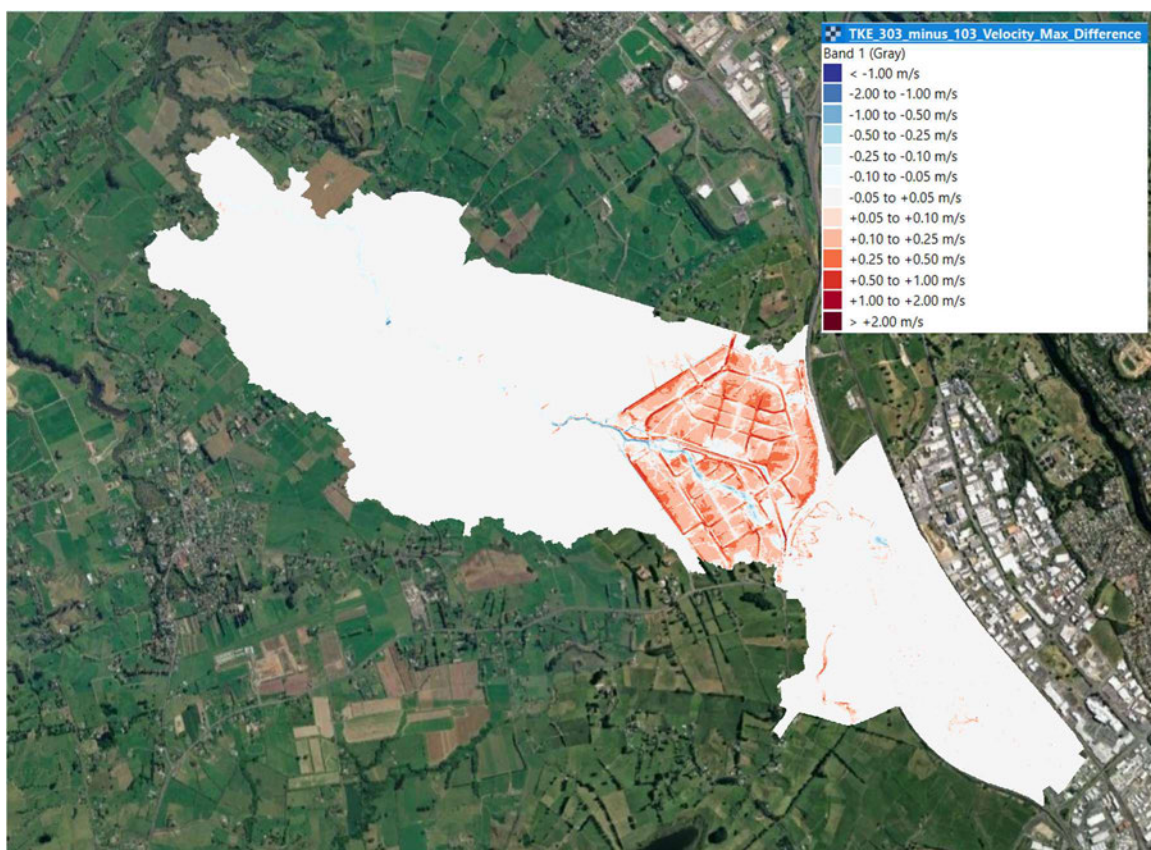


Figure 8. Difference in maximum velocity (post-development minus pre-development) - 100-year ARI event (Review Item C4.4).

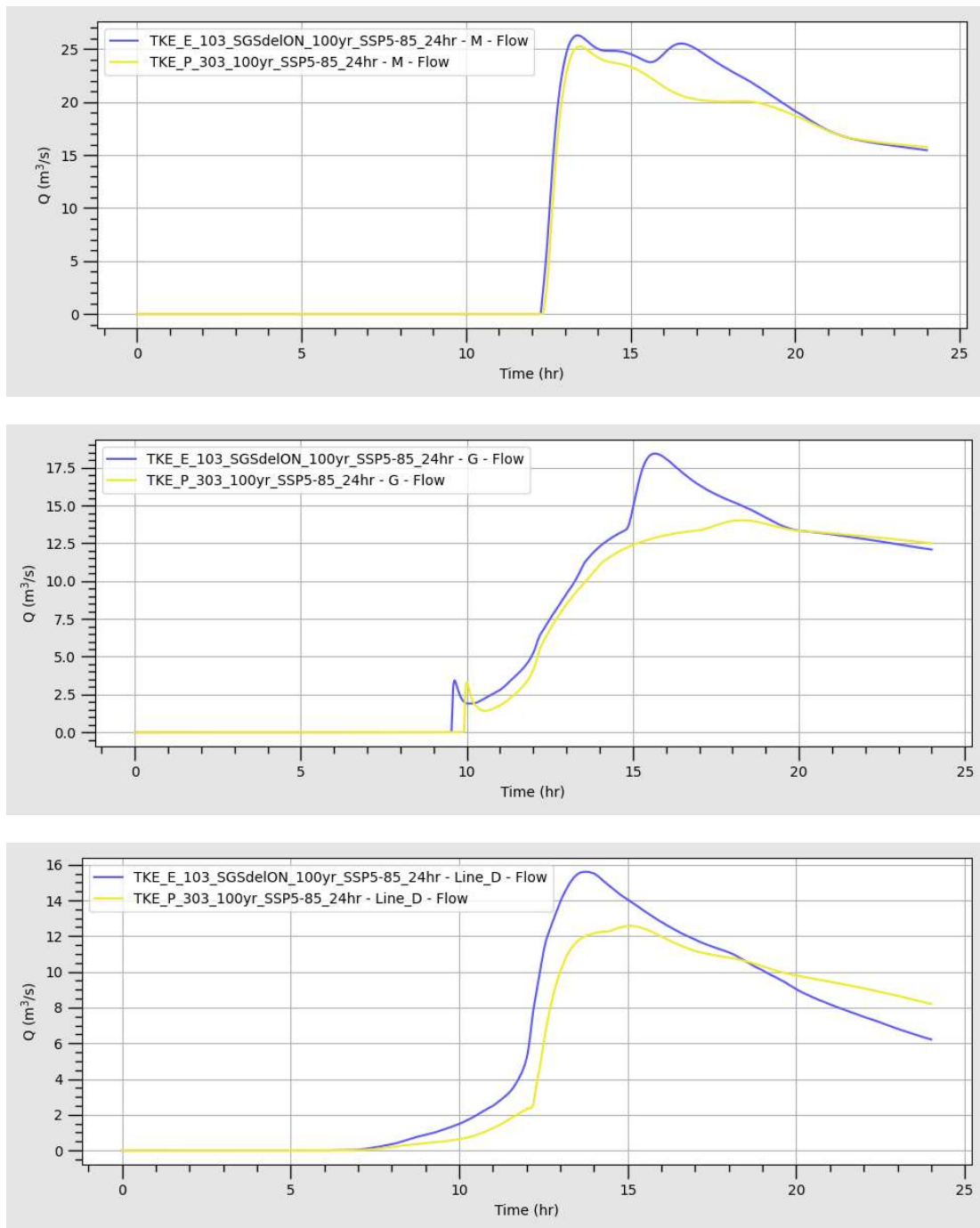


Figure 9. Pre- and post-development flow hydrographs: (a) near the downstream water-level boundary; (b) downstream of Crawford Road; and (c) downstream of the pond system - 100-year ARI event (Review Item C4.1).

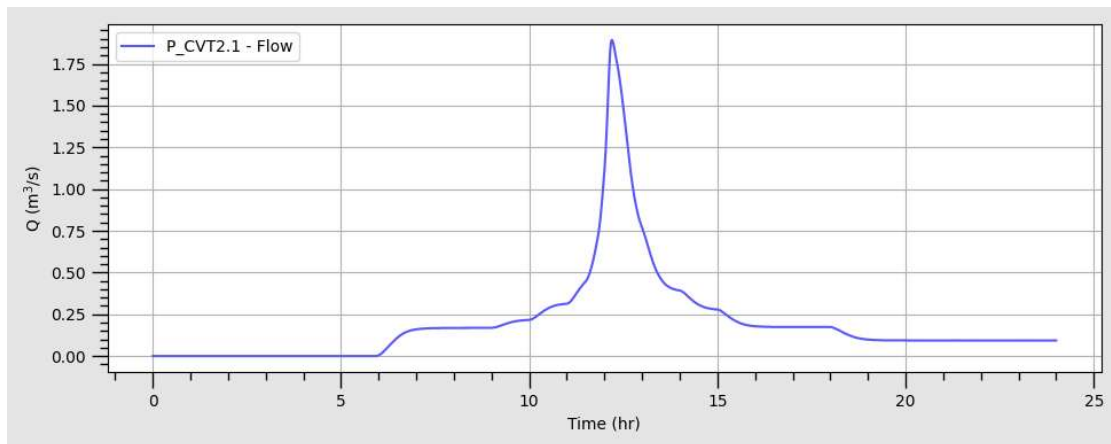


Figure 10. Example of non-oscillatory flow through a representative culvert (Review Item C2.4).



Figure 11. Example of localised SGS cell connectivity identified using the Breakline Detection Delta check (Review Item B2.4).

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89 Degrees Reference: NZ10059-P01-02
28th August 2026

James Oakley
Principal Three Waters Engineer
Te Miro Water Consultants
Suite 7, 3 Empire Place
Cambridge, 3434

SUBJECT: Te Kowhai East TUFLOW Model – Independent Technical Peer Review

Dear James,

Thank you for providing Te Miro Water's responses to the peer review items. I have reviewed each response and, on the basis of the comments provided, have closed out the review items.

For the key matters identified in the review, Te Miro Water has confirmed that these have been considered and, where appropriate, further assessed through sensitivity testing. Te Miro Water is satisfied that these matters do not materially affect the model results or conclusions and that no further model amendments are required.

I have not independently reviewed the additional sensitivity testing undertaken as part of this response. Closure of these items therefore reflects acceptance of Te Miro Water's assessment and professional judgement on these matters.

The updated peer review register, with all items now closed, is provided with this letter.

If you have any points you wish to discuss, please do not hesitate to contact me.

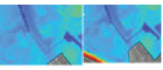
Sincerely,



Colin Roberts

Director



84.6		Post-development topographic modifiers include the existing 3d_rsk_383_LP and 3d_rsk_383_LP features described above, together with proposed development modifiers.	Requires response or amendment		Response accepted. Modeler advises that sensitivity testing has been undertaken and that the matter does not materially affect the model results or conclusions.	Good
84.5	Minerals: Minerals source data	Review source data used to define model materials and confirm its suitability, currency and spatial coverage.	Generally acceptable			Good
84.6	Minerals: Minerals file definition and values	Refer to detailed Minerals file review in Section 85.	Not required			Not applicable
84.7	Minerals: Minerals modifications	Review the spatial implementation of mineral modifications within the TIC, including any local overrides or changes associated with development.	Acceptable – recommendations for improvement	Noted. The goal was to use constraint layers between the existing and the post-development models to the extent and general alignment with local features rather than detailed, so that any changes to one scenario is reflected onto the other consistently with the complete scenario valid. Details of different mappings used in the model is included in the Model Build report.	Response accepted. Modeler has assessed the matter and is satisfied that no further model assessment is required.	Good
84.8	Soils: Soil source data	Review source data used to define model soil definitions characteristics and confirm its suitability and spatial coverage.	Not required			Not applicable
84.9	Soils: Soil file definition and values	Refer to detailed Soils file review in Section 85.	Not required			Not applicable
84.10	Soils: Soil modifications	Review the spatial implementation of soil modification modifications within the TIC, including the changes associated with development.	Not required			Not applicable
84.11	3D Hydraulic Structures	Review the implementation of 3D hydraulic structures and controls defined through the TIC, including their location, geometry and associated with the surrounding terrain.	Generally acceptable			Good
84.12	Project consistency and changes	Review differences between the pre- and post-development TIC configurations and confirm that changes are intentional and consistent with the development representations.	Acceptable – recommendations for improvement	Noted. The small difference in orientation of the modelling surface was considered unlikely to affect results. In the model is intended as a broad model effects assessment not a detailed model.	Response accepted. Modeler has assessed the matter and is satisfied that no further model assessment is required.	Good
85. DETAILED MODEL REVIEW – 3D RISK						
85.1	3D network configuration and structure	Review the overall 3D network configuration and representation hydraulic structures, including structure types, locations, dimensions, flow levels, roughness, loss coefficients and other relevant parameters.	Acceptable – recommendations for improvement	Noted.	Response accepted. Modeler has assessed the matter and is satisfied that no further model assessment is required.	Good
85.2	3D Flow Control	Review the implementation of 3D flow control settings for consistency with the 3D model and operational capability requirements.	Generally acceptable			Good
85.3	3D/2D connections	Review the implementation of 3D/2D connections and mapping methodologies, including connectivity and elevation or representative locations.	Requires response or amendment	The 3D connections have been revised from the initial runs, with additional 2D cells and increased average bottom, particularly along the main channels. This has significantly reduced the modification and gradual structure hydraulic. The locations identified in the review will be checked and revised where appropriate, primarily to improve model results rather than change the overall flow components. The larger 2 adjustments will be checked against the corrected structure levels and local levels to confirm they are appropriate. 10 polygons were generally added where they could create a more realistic large depression around certain water bodies. However, these will be considered where they provide a clearer connection without adversely affecting the local results.	Model updates undertaken. Response accepted. Modeler advises that model updates have been 7 with no substantive and that improvements observed.	Good
85.4	Project consistency and changes	Review differences in the 3D network between pre- and post-development configurations and confirm that changes are intentional and consistent with the development representations.	Generally acceptable			Good
86. DETAILED MODEL REVIEW – 3D/2D/1D/2D						
86.1	Source location data	Review the source location data used to define hydraulic structures.	Not required			Not applicable
86.2	Minerals classification	Review the methodology used to classify source data into model material types.	Generally acceptable			Good
86.3	Minerals mapping	Review mapping between source location data and 3D/2D/1D/2D model files.	Generally acceptable			Good
86.4	Minerals mapping: model and values	Review the adopted Manning's n model and values and their appropriateness for the represented surfaces and expected flow conditions.	Acceptable – recommendations for improvement	Noted. Being pre-post comparing model, emphasis was given to the mapping detail on site, and a straight approach was followed for the one, and large consistent between scenarios.	Response accepted.	Good
86.5	Other materials parameters	Review other relevant materials fit parameters, including percentage impermeability and any loss parameters when applied.	Generally acceptable			Good
86.6	Spatial distribution	Identify representative checks of the spatial distribution of model materials and roughness.	Generally acceptable			Good
86.7	Project consistency and changes	Review changes to materials and roughness between pre- and post-development configurations and confirm these are intentional and appropriate.	Acceptable – recommendations for improvement	Consistent corrections have been applied to the post-development model to ensure any small errors are also properly reflected.	Response accepted. Modeler has assessed the matter and is satisfied that no further model assessment is required.	Good
87. DETAILED MODEL REVIEW – 3D/2D/1D/2D						
87.1	Source well data	Review source well location information used to define surface features.	Acceptable – recommendations for improvement	Refer model build report for further information about adopted CHs.	Response accepted.	Good
87.2	Soil classification	Review classification of source well location into model soil/structure types.	Acceptable – recommendations for improvement	As above.	Response accepted.	Good
87.3	Spatial mapping	Review spatial mapping of soil/structure types within the model.	Generally acceptable			Good
87.4	Hydrology: hydrology, model and parameters	Review the adopted hydrology/flow methodology and associated parameters. Review adopted parameters at a high level for reasonableness and identify whether their basis/assumptions have been documented.	Acceptable – recommendations for improvement	Details of 3D method and adopted values are given in the model build report.	Response accepted.	Good
87.5	Schedules: soil layer flow model and associated parameters	Review the subsurface soil layer flow representation and adopted parameters, where stated.	Generally acceptable			Good
87.6	Hydrology: hydrology, model and parameters	Review the subsurface soil layer flow representation and adopted parameters, where stated.	Generally acceptable			Good

87.7	Pre-joint consistency and changes	Review changes to hydro-physical parameters and input scenarios between pre- and post-development conditions and confirm these are intentional and appropriate.	The pre- and post-development models use different classification layers within the development area. Comparison of the spatial parameters confirms that a number of areas are unaffected by 100-year rainfall and these in the pre-development model. In addition, against aerial imagery indicates that these changes occur within areas that are partly developed under existing conditions and may represent further development under the post-development scenario. The changes therefore appear locally plausible; however, the basis for the adopted pre-joint coupling should be documented as clear changes associated with the proposed development are clearly identifiable.	Acceptable – recommendations for improvement	As per above and detailed in the model build report	Response accepted.	Closed
88. DETAILED MODEL REVIEW – TMC FILE							
88.1	Boundary configuration	Review the overall configuration, location and implementation of model boundary conditions.	Model is applied across the model domain. A downstream water level boundary is applied at the downstream end of the main stream, with additional outlet water level boundaries where cross-channel culverts cross flow out of the domain. Internal 10/20/30 connections are also defined. The post-development model additionally includes an internal 24-hr inflow boundary representing flow from Redbank. The overall boundary configuration is considered appropriate.	Generally acceptable			Closed
88.2	Rainfall boundaries	Review the spatial implementation and assignment of rainfall boundary conditions.	Rainfall is applied as a spatial pattern 24-hr boundary. The design rainfall time series use a model 30-hour temporal pattern with the storm peak centred at approximately 12 hours. Rainfall values are specified in millimetres as mean-day accumulated depth. The implementation is considered accurate.	Generally acceptable			Closed
88.3	Internal boundaries	Review the location, extent and implementation of internal hydraulic boundaries.	A 10/20 normal-depth boundary is applied at the downstream end of the main stream using a slope of 0.005 (0.004). The boundary is located sufficiently far downstream that it is not expected to materially influence the principal area of interest. Additional 10-culvert boundaries are constant water levels. The adopted boundary treatment is considered reasonable.	Generally acceptable			Closed
88.4	Internal 10/20 boundaries	Review relevant boundary connections associated with the coupled 10/20 system.	10/20 coupling is defined using 10 pairs and the connection, with 2 days used to reconcile local 10 connection details with the associated 10 structure elevations. Review of the model messages identifies a number of locations where the 2-point adjustment was used. The slope adjustments should be checked against the design rainfall time series. The 2-point adjustment should be checked against the 10 connection locations and/or structure invert levels are correct. Where the adjustments are incorrect, their basis should be documented.	Acceptable – recommendations for improvement	10 hr time series in the model is imposed (hardly) to the point where the flow in the major culvert is affected without doing much attention to the base flow. Some details for this are presented in the report.	Response accepted. Modeler advises that model output has been undertaken and improvements observed.	Closed
88.5	Internal source/sink boundaries	Review any internal source/sink boundaries, including their purpose, location and implementation.	The post-development model includes an internal Redbank inflow boundary. The applied hydrograph peaks at approximately 2 m/s, around the centre of the reach, and a value of 1.2 is applied in the IC database. The inflow does not appear excessive, however, the source and basis of both the hydrograph and the 1.2 multiplier should be documented in the model report and probably noted in the TMC/IC database configuration.	Acceptable – recommendations for improvement	Redbank inflow is increased by 20% to account for its different climate change scenario in the two models. Redbank uses ICN2.1 whereas TMC uses ICN5.4. An arbitrary 20% increase to the ICN5.4 flow was included to approximate ICN4.5 flow from this catchment. Refer model build report for details.	Response accepted.	Closed
88.6	Pre-joint consistency and changes	Review differences in boundary configuration between pre- and post-development models and confirm changes are intentional and appropriate.	Boundary condition inputs are generally consistent between the pre- and post-development models. Differences are associated with the proposed development configuration, principally the additional Redbank inflow boundary and upper internal.	Generally acceptable			Closed
89. DETAILED MODEL REVIEW – IC DATABASE FILE							
89.1	IC database configuration	Review the IC database structure and confirm boundary values, types and references are consistent with the TMC and associated model inputs.	The IC database is clearly structured and provides the required links between model boundary values and the associated source data. No issue identified.	Generally acceptable			Closed
89.2	Source data linkage	Review linkage of model boundaries to the intended source/time/area data, including relevant units and scaling where applicable.	Boundary condition source file and the associated threshold volume reference, units and scaling were reviewed and are generally consistent with the supplied data. The post-development Redbank inflow applies a value of 1.2, the basis for this multiplier should be documented.	Acceptable – recommendations for improvement	As above	Response accepted.	Closed
89.3	Pre-joint consistency and changes	Review IC database differences between pre- and post-development configurations and confirm changes are intentional and appropriate.	The pre- and post-development IC database configurations are generally consistent. Additional boundary inputs in the post-development model relate to the proposed development configuration and upper internal. Consideration should be given to using a single common IC database for both model configurations, with some specific boundary inputs selected as required. This would reduce duplication and improve consistency and maintainability between the pre- and post-development models.	Generally acceptable			Closed
910. DETAILED MODEL REVIEW – TIME SERIES							
910.1	Design rainfall	Review the source and definition of design rainfall inputs, including assigned event magnitudes.	Design rainfall time series have been provided for the 2-, 10- and 100-year ARI events under the ICN5.4 climate scenario. The corresponding 24-hour rainfall depths are approximately 85.1 mm, 127.2 mm and 158.8 mm respectively. The adopted rainfall depths appear reasonable for the assessment. The source and definition of the design rainfall depths should be documented in the model report.	Generally acceptable			Closed
910.2	Temporal patterns	Review assigned rainfall temporal patterns and their appropriateness for the assessment.	The 30-hour rainfall events use a 5-minute temporal discretisation and a related temporal pattern centred approximately midday across the storm. The pattern provides progressively shorter duration rainfall bursts around the storm peak and is consistently applied across the 2-, 10- and 100-year ARI events. The adopted temporal pattern is considered suitable. The associated rainfall time series should be documented in the model report.	Generally acceptable			Closed
910.3	Storm duration	Review assigned storm durations and whether they adequately capture the critical model response.	A 30-hour storm duration is adopted for the 2-, 10- and 100-year ARI events. The duration is considered reasonable for the assessment and proposed development; however, the basis for selection of the 30-hour duration should be documented, including confirmation that it is appropriate for assessing the critical peak flow response and performance of the proposed development system. The 24-hour model duration period provides additional time to assess retention and drain-down following creation of rainfall.	Generally acceptable			Closed
910.4	Downstream boundary time series	Review downstream boundary time series, including source, levels, datum and temporal coverage.	Downstream boundaries comprise constant water level boundaries at the nominated outfall (24.4 m, 26.0 m and 28.35 m rd) and an HGL normal-depth boundary adopting a slope of 0.005 (0.006). The adopted boundary treatment is considered reasonable. The downstream boundary is located sufficiently far from the principal area of interest that it is not expected to materially influence the assessment results.	Generally acceptable			Closed
910.5	Other time series	Review other time-varying boundary, structure or control inputs where applicable.	The post-development model additionally includes a Redbank inflow time series. The applied hydrograph peaks at approximately 2 m/s, and the IC database applies a value of 1.2. The source and basis of the inflow hydrograph and the applied multiplier should be documented.	Acceptable – recommendations for improvement	As above	Response accepted.	Closed
910.6	Time-series consistency and coverage	Confirm time-series inputs, timing and duration coverage are consistent with model requirements.	Time-series inputs, including rainfall increments and temporal coverage were reviewed and are consistent with the model configuration and data periods. The applied boundary time series provide sufficient temporal coverage for the required simulations. No issue identified.	Generally acceptable			Closed
910.7	Pre-joint consistency and changes	Confirm timing is consistent between pre- and post-development simulation models where time-series related.	ICN5.4 rainfall timing is applied consistently between the pre- and post-development simulations, providing an appropriate basis for direct comparison. Other boundary conditions are generally consistent, with differences associated with the proposed development configuration. These differences appear intentional.	Generally acceptable			Closed
91. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – NUMERICAL PERFORMANCE							
91.1	Model comparison	Confirm representative simulations compare acceptably.	Representative pre- and post-development 100-year ARI simulations were run as part of the review. Both simulations compared consistently to 24 hours. The 30-hour simulation period corresponds to the intended duration defined in the TMC, however, it is currently specified for the 72-hour limit. The modelled flow is 100 m model coordinate. The following messages were reviewed and are considered routine, minor/localised, hardware-related, or otherwise consistent in the modelled model results.	Generally acceptable			Closed
91.2	Simulation log and messages	Review routine CHECK/MINORS messages and confirm they do not indicate material model issues.	• CHECK 350 – 60 occurrence. MINOR/LOCAL These options used within the 900 model. Addressed under the terrain/hydrographic modification review. • WARNING 344 – 18 occurrence. Input file geometry may be empty. Appears to relate to minor 3D geometry flopping and does not materially affect the simulation. Should be checked. • WARNING 1202 – 7 occurrence. structure cross/height below adjoining channel bed, with relatively small offset of approximately 0.02-0.08 m. • CHECK 207/207A – 4 occurrence each. changing c/s/level of 10/20 lines. Local geometry checks with no apparent material hydraulic consequence. • WARNING 2683 – 7 occurrence. Manning's n exceeds the Ws formulation limit of 0.100 and is limited to 0.030 for the relevant hydraulic model calculation. • WARNING 2684 – 8 occurrence. Quadratic cells with no active faces. Localised/localised occurrences with no apparent material effect on model performance. • CHECK 2252 – 5 occurrence. top of first flow Constitution layer below ground level. Associated with the layered IC representation reviewed separately. • WARNING 2384 – 1 occurrence. two GPUs required with only one available. Hardware-related only and of no hydraulic significance. No material model performance issue is identified from these messages.	Generally acceptable			Closed
91.3	Simulation log and messages	Review CHECK/MINORS messages that warrant confirmation or improvement of the model configuration.	The following messages warrant further consideration, although they are not presently considered indicative of a material numerical stability issue.	Acceptable – recommendations for improvement	The current 3D setup provides stable peak flow and is considered appropriate for the intended existing versus post-development comparison. Further changes are therefore not considered necessary at this stage. Description of 3D assumptions are included within the report.	Response accepted. Modeler has assessed the matter and is satisfied that no further model amendment is required.	Closed
91.4	Model balance / volume checks	Review reported mass balance and volume checks and identify any material conservation issues.	• CHECK/MINORS 2118 – 114 occurrence. SE IC elevation lowered to associated 10 node bed levels. Adjustments range from approximately 0.149 m, with 42 occurring 0.1 m and 18 exceeding 1.0 m. The larger adjustments should be checked against local terrain, structure level levels and 3D connection configuration. • CHECK 1452 – 8 occurrence. multiple culverts connected at nodes where a markid has not been created. The affected locations should be checked to confirm that the time-step connectivity and hydraulic representation are appropriate. • WARNING 2684 – 2 occurrence. 10 node/30 IC connection associated with locations that are not active Quadratic cells. The affected locations should be checked. • CHECK 2086 – 3 occurrence. repeat application of an 3D boundary to a 2D cell has been ignored. These locations should be reviewed together with the 3D connection geometry. These messages primarily relate to local 10/20 coupling, structure configuration and requires treatment rather than overall numerical stability. The affected locations/settings should be reviewed and confirmed.	Generally acceptable			Closed
91.5	Numerical performance	Review time-series behaviour, control numbers, solution times occurrence, repeated iterations and other indicators of numerical stability/performance.	The reviewed post-development 100-year ARI simulation has a final cumulative mass error of 0.006, with a reported volume error of 18 m ³ (0.006 of total volume in m ³). No material mass conservation issue is evident.	Acceptable – recommendations for improvement	The variations identified at some culverts have been reviewed and are considered minor and localised, with no overall effect on the peak flow or overall model response. The 3D configuration is therefore considered adequate provide stable hydraulic behaviour at the key structures while maintaining the existing 10/20 treatment as far as practicable.	Response accepted. Modeler has assessed the matter and is satisfied that no further model amendment is required.	Closed
91.6	Model check file	Review relevant TUFLOW check files to identify apparent errors, inconsistencies or unexpected aspects of the model operation; record material issues rather than cataloguing all check outputs.	Adaptive time-stepping is enabled and the computational time-step responds appropriately through the simulation, reducing during the more hydraulically active period and increasing as conditions permit. The time-step remains within the configured allowable range, with no evidence of persistent time-step violation or numerical instability. No significant control numbers or traps, reduced time-step execution, and no negative 10 depths were reported.	Acceptable – recommendations for improvement	Some localised oscillatory flow behaviour is evident at selected 10-culverts, predominantly during the initial and recession portions of the simulation. This is not considered to materially affect the principal model response. However, consideration should be given to refining the local 3D coupling at affected structures, including elevation smoothing, upstream/downstream connection treatment and provision of sufficient local 10 storage.	Response accepted. Modeler has assessed the matter and is satisfied that no further model amendment is required.	Closed
91.7	Model check file	Review relevant TUFLOW check files to identify apparent errors, inconsistencies or unexpected aspects of the model operation; record material issues rather than cataloguing all check outputs.	The principal model check file was reviewed for the post-development 100-year ARI simulation. The check file generally confirms that the intended model inputs, materials, units, boundary conditions, 10 treatment, flow connections and terrain modification have been applied as expected. No additional specific issues were identified beyond those already documented in the detailed model review.	Acceptable – recommendations for improvement	Noted. The identified 10-20 level differences acknowledged in the current model setup. As the current configuration provides the required hydraulic connectivity and stable model response, no further changes are considered necessary at this stage.	Response accepted.	Closed
91.8	Model check file	Review the 10/20 check output references the observations made regarding 10 coupling. A number of locations show appreciable differences between the 10 node bed level and the connected 10 node elevation, including several proposed substructure connections. These should be considered together with the 10/2 adjustment messages and the recommendations made previously regarding local 3D connection geometry and elevation treatment.	Review of the 10/20 check output references the observations made regarding 10 coupling. A number of locations show appreciable differences between the 10 node bed level and the connected 10 node elevation, including several proposed substructure connections. These should be considered together with the 10/2 adjustment messages and the recommendations made previously regarding local 3D connection geometry and elevation treatment.	Acceptable – recommendations for improvement			Closed
92. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – HYDRAULIC BEHAVIOUR							
92.1	Flow patterns	Review whether principal modelled flow patterns are hydraulically sensible.	The spatial distribution and direction of flow were reviewed using the modelled velocity vectors. Flow paths are hydraulically coherent and consistent with the model terrain and expected drainage behaviour. Local eddies and circulation patterns occur where reasonably expected from the local geometry, with no evidence of implausible flow patterns identified. Flow approaching the downstream boundaries is well behaved and generally perpendicular to the boundary alignment, including appropriate boundary placement and hydraulic behaviour.	Generally acceptable			Closed
92.2	Depth and water levels	Review representative depths and water levels for plausibility.	Maximum depth results were reviewed spatially across the model domain. The distribution of modelled depths is hydraulically coherent, with greater depths generally associated with defined channels, drains, storage areas and local depression. No anomalous areas of unrealistic depth were identified from the model.	Generally acceptable			Closed
92.3	Velocities	Review representative velocities and spatial patterns for plausibility.	Maximum velocity results were reviewed spatially across the model domain. The velocity distribution is generally hydraulically coherent, with higher velocities concentrated along defined channels, drainage paths and local hydraulic controls. The highest velocities are localised and appear consistent with concentrated convergence through the represented drainage system. No widespread anomalous velocity behaviour was identified.	Generally acceptable			Closed
92.4	Hydraulic structures	Review modelled behaviour of key culverts, bridges and other hydraulic controls/obstacles.	Localised high velocity regions, including maximum velocities of approximately 4 m/s, should nevertheless be checked and confirmed as representative of the intended local hydraulic and terrain/hydrographic representation.	Generally acceptable			Closed
92.5	Hydrograph response and overall hydraulic behaviour	Review representative hydrographs and overall temporal hydraulic response for consistency with the expected catchment and model behaviour.	Flow through representative culverts and other hydraulic structures was reviewed using the modelled structure flow time series. The magnitude, timing and overall form of the hydrographs are considered reasonable and consistent with the expected hydraulic response. No anomalous structure flow behaviour was identified that is considered to materially affect the model results.	Generally acceptable			Closed
92.6	Hydrograph response and overall hydraulic behaviour	Review representative hydrographs and overall temporal hydraulic response for consistency with the expected catchment and model behaviour.	Some localised oscillatory behaviour is evident at selected culverts, this is addressed separately under the numerical performance review and is not repeated here.	Generally acceptable			Closed
93. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – PRE-JOINT COMPARISON							
93.1	Model consistency	Confirm common model components and settings are consistent between pre- and post-development simulations.	The pre- and post-development models are currently maintained as separate model configurations. The review indicates that the models are generally consistent, with the principal differences associated with the proposed development. However, maintaining separate configurations results in duplication of control files and associated inputs, including between where the flow has the same name but different content between the pre- and post-development model packages. This increases the potential for unintended differences and makes auditing of the development changes more difficult.	Acceptable – recommendations for improvement	Noted with thanks.	Response accepted.	Closed

C3.2	Development changes	Confirm differences between model configurations appropriately represent the proposed development and mitigation works.	Consideration should be given to applying proposed development changes as near-to-specific overlays to the existing condition model whenever practicable, rather than replicating existing terrain or modifying existing inputs. This would provide a clearer audit trail of the changes introduced by the development and reduce the potential for unintended modification of existing condition inputs.	Acceptable – recommendations for improvement	Noted with thanks	Response accepted.	Closed
C3.8	Comparability of results	Confirm reported pre/post differences can reasonably be attributed to development changes rather than unrelated model differences.	No unrelated systematic differences have been identified that would prevent meaningful comparison of the pre- and post-development results. Consideration should nevertheless be given to using a common 10% and associated control structure, with pre- and post-development configurations selected using TUFLOW scenarios. This would reduce duplication, improve consistency between model configurations and provide a more transparent and rationalised model structure for future application and review.	Acceptable – recommendations for improvement	Noted with thanks	Response accepted.	Closed
C4. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – ASSESSMENT RESULTS							
C4.1	Peak flow and hydropower response	Review representative pre- and post-development flow hydrographs at key assessment locations to assess changes in peak flow, timing and overall hydropower response.	Representative pre- and post-development flow hydrographs were compared for the 100-year ARI event at key locations through and downstream of the development. The post-development hydrographs demonstrate attenuation of the pre-development peak flows, with a smoother and more distributed response consistent with the proposed storage and flow control system. The timing and form of the hydrographs are hydraulically coherent, and no adverse increase in downstream peak flow was identified at the locations reviewed. The post-development response includes redistribution of flow into the recession period, consistent with attenuation and delayed release from the proposed storage system.	Generally acceptable			Closed
C4.2	Maximum depth differences	Review spatial differences in maximum depth between the pre- and post-development simulations, with particular consideration of the location and magnitude of changes and whether material adverse increases occur outside areas modified by the proposed development.	Maximum depth differences between the pre- and post-development 100-year ARI simulations were reviewed spatially. The principal changes are associated within the proposed development and wetland/storage areas and are consistent with the associated changes in terrain, drainage and storage representation. Outside the areas modified by the development, changes are generally minor and no material adverse increase in maximum depth was identified.	Generally acceptable			Closed
C4.3	Maximum water level differences	Review spatial differences in maximum water level between the pre- and post-development simulations, with particular consideration of whether material adverse increases occur outside the proposed development and associated wetland/storage areas.	Maximum water level differences between the pre- and post-development 100-year ARI simulations were reviewed spatially. Material changes are principally associated with the proposed development and wetland/storage areas. Outside these areas, differences are generally negligible and no material adverse increase in maximum water level was identified.	Generally acceptable			Closed
C4.4	Maximum velocity differences	Review spatial differences in maximum velocity between the pre- and post-development simulations, including the location and magnitude of increases and whether material adverse changes occur outside the proposed development or at downstream hydraulic controls.	Maximum velocity differences between the pre- and post-development 100-year ARI simulations were reviewed spatially. Changes are principally concentrated within the proposed development and associated drainage and wetland/storage areas and are consistent with the adverse flow paths and hydraulic controls. Outside these areas, changes are generally minor and no widespread or unexpected adverse increase in maximum velocity was identified. Localised higher-velocity areas were reviewed and appear hydraulically reasonable.	Generally acceptable			Closed
C5. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – ROBUSTNESS AND UNCERTAINTY							
C5.1	Model uncertainty and sensitivity	Consider the significance of model uncertainty and whether sensitivity testing is appropriate for key model descriptions and parameters that may influence the assessment conclusions.	The assessment is principally comparative, with the effects of the proposed development assessed relative to the pre-development model using broadly common model descriptions and forcing. This reduces the influence of non-absolute model uncertainty when assessing relative change. Nevertheless, uncertainty remains associated with model schematisation and numerical scheme parameters and component parameter choices, and targeted sensitivity testing should be considered where assessment conclusions depend on relatively small predicted differences or sensitive hydraulic controls.	Acceptable – recommendations for improvement	Noted. Updates to cell configuration and model initial conditions have been tested and resulted in no change to the model outputs, particularly at the downstream end of the development site.	Response accepted. Modeller agrees that sensitivity testing has been undertaken and that the model does not materially affect the model results or conclusions.	Closed
C5.2	Proposed development controls	Consider uncertainty associated with the representation of proposed attenuation and flow control structures, and whether sensitivity to key structure parameters could materially affect the assessment conclusions.	Particular consideration should be given to uncertainty associated with the proposed attenuation and flow control structures, as their representation may directly influence predicted peak flows, water levels and drain-down behaviour. Sensitivity to key structure dimensions, levels and discharge characteristics should be considered where flow parameters are material to the assessment conclusions.	Acceptable – recommendations for improvement	Flow control inputs reviewed from Mosen and applied in the model as is. However, outputs are checked for the effective outlet control operation.	Response accepted. Modeller agrees that sensitivity testing has been undertaken and that the model does not materially affect the model results or conclusions.	Closed
C6. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – DOCUMENTATION							
C6.1	Model documentation	Review whether the modelling methodology is adequately and accurately described.	A final modelling report was not provided as part of this review and the adequacy of the model documentation has therefore not been assessed. The final documentation should clearly describe the model development and methodology, including the principal assumptions and the basis for key modelling decisions, and should address matters identified in this review where clarification or additional documentation has been recommended. A final modelling report was not provided as part of this review and the presentation and interpretation of the model results and assessment conclusions have therefore not been reviewed. The final report should clearly present the relevant pre- and post-development results and demonstrate how these support the assessment conclusions.	Not reviewed			Not applicable
C6.2	Results and conclusions	Consider whether the model results are presented and interpreted sufficiently to demonstrate the effects of the proposed development and support the assessment conclusions.		Not reviewed			Not applicable
C7. MODEL PERFORMANCE, APPLICATION AND FITNESS FOR PURPOSE – OVERALL ASSESSMENT							
C7.1	Fitness for purpose	Taking account of model schematisation, detailed representation, numerical performance, uncertainty and results, assess whether the model is fit for its stated purpose.	Overall, the model is considered suitable for its intended application, subject to the matters identified in this review being appropriately addressed. No issue has been identified that fundamentally undermines the model methodology or its use for the intended pre- and post-development assessment. Review of the model results indicates generally coherent hydraulic behaviour and good numerical performance. The pre- and post-development comparisons reviewed for the 100-year ARI event demonstrate attenuation of peak flows and hydraulically reasonable changes in depths, water levels and velocity, with no material adverse effects identified outside the areas modified by the proposed development. Three model matters are highlighted for specific response: the adopted initial condition water levels, the use of 100/1000 L/s rainfall modification flow within the full model, and the extent and magnitude of 3-day adjustments associated with some 10/100 L/s connections. The initial condition water levels should be reviewed to confirm that they provide an appropriate representation of antecedent conditions and do not materially influence the assessment results. The 100/1000 L/s 3-day adjustments should similarly be further assessed to confirm that the adopted model representation is appropriate and is not materially influencing the modelled hydraulic response. Other review findings are principally recommendations for clarification, documentation or improvement.	Acceptable – recommendations for improvement	Thanks for the review. The identified items have been considered and sensitivity checks have been undertaken as necessary, with the conclusion that the identified areas do not materially affect the assessment outcomes. No further model changes are proposed.	Response accepted. Modeller agrees that sensitivity testing has been undertaken and that the model does not materially affect the model results or conclusions.	Closed