Stokes Valley

PREPARED FOR WELLINGTON WATER | March 2023

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Stantec

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Executive Summary

Stantec (then MWH), were engaged by Wellington Water Ltd (WWL) in 2016 to develop a comprehensive model of the stormwater network for the Stokes Valley catchment in Lower Hutt. This work is part of the collaborative Stormwater Modelling Panel formed by WWL in 2016 to build and maintain stormwater models across the Wellington region.

The model predominantly follows the standard approach set out by WWL in the WWL Regional Stormwater Modelling Specifications v5 (Wellington Water Ltd, 2017). However, there are some key deviations from the specification as the methodology was under development concurrently with the development of the model. This report describes in detail the initial model build, the model validation, the model sensitivity analysis and freeboard assessment. This report is a compilation of previous reports submitted to WWL including: Stokes Valley Hydraulic Model Build (MWH (Now Stantec), 2016), Stokes Valley Model Validation of November 15 2016 event (Stantec, 2017), Stormwater Modelling Panel - Stokes Valley Model Update and Validation (Stantec, 2018), and Stormwater Modelling Panel – Stokes Valley Model Update – Time of Concentration Values. It also incorporates some of the more recent works including sensitivity and freeboard analysis and further model updates following external peer review.

The results for the 100-year ARI design simulation shows extensive flooding across Stokes Valley. There is significant flooding through, and upstream of, the town centre. In the town centre flood depths are upwards of 1m. Downstream of the town centre flooding is restricted mainly to the Stokes Valley stream and its corridor through the valley, except significant overland flow originating from the eastern branch of the valley containing Glen Rd. The 10-year ARI design simulation shows moderate flooding across Stokes Valley, particularly through and upstream of the town centre. There is also extensive overland flow in the eastern branch of the valley.

Model validation was undertaken using recoded rainfall during the December 2019 and November 2016 storm events. The model validates well against the available data for observed flooding during the December 2019 storm. Generally, flood extents are consistent between observed and modelled flooding, however there are some areas where minor overestimation of flooding has likely occurred in the model results. The 15 November 2016 model simulation appears to also compare well against the observed flooding. Photos taken following the rainfall event in the lower reaches of the Stokes Valley Stream channel show that flood depths within the channel are similar to those indicated in the photos. Though, at some locations in the middle reaches of Stokes Valley, modelled depths are greater than observed flooding.

Model sensitivity analysis was undertaken in 2019 with the intent of understanding the sensitivity of the stormwater network to a range of different model parameters. These were used to select values for freeboard allowance to develop a baseline flood extent for assigning minimum building floor levels for Stokes Valley. Freeboard values were assigned across the catchment and the final freeboard simulation was run in May 2021.

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Abbreviations

WWL	Wellington Water Limited
HCC	Hutt City Council
GWRC	Greater Wellington Regional Council
LINZ	Land Information New Zealand
NIWA	National Institute of Water and Atmospheric Research
ICM	Integrated Catchment Modeller
UMM	Urban Model Manager
DTM	Digital Terrain Model
ASL	Above Sea Level
ARI	Annual Recurrence Interval
RFHA	Rapid Flood Hazard Assessment
HIRDS	High Intensity Rainfall Design System
SCS	Soil Conservation Service
IA	Initial Abstraction
CN	Curve Number
ToC	Time of Concentration
LiDAR	Light Detection and Ranging
GIS	Geographical Information System
SQL	Structured Query Language

1. INTRODUCTION

1.1 BACKGROUND

Stantec (MWH) were engaged by Wellington Water Ltd (WWL) in 2016 to develop a comprehensive model of the stormwater network for the Stokes Valley catchment in Stokes Valley, in Hutt City. This work is part of the collaborative Stormwater Modelling Panel formed by WWL in 2016 to build and maintain stormwater models across the Wellington region.

The model predominantly follows the standard approach set out by WWL in the WWL Regional Stormwater Modelling Specifications v5 (Wellington Water Ltd, 2017). However, there were some key deviations from the specification as the methodology was under development concurrently with the construction of the model. This report will describe in detail the initial model build, the model validation, the model sensitivity analysis and freeboard assessment. This report is a compilation of previous reports submitted to WWL including: Stokes Valley Hydraulic Model Build (MWH (Now Stantec), 2016), Stokes Valley Model Validation of November 15 2016 event (Stantec, 2017), Stormwater Modelling Panel - Stokes Valley Model Update and Validation (Stantec, 2018), and Stormwater Modelling Panel – Stokes Valley Model Update – Time of Concentration Values. It also incorporates some of the more recent works including sensitivity and freeboard analysis and further model updates following external peer review.

1.2 CATCHMENT OVERVIEW

The catchment is in Stokes Valley, a major suburb of the city of Lower Hutt (Figure 1-1). The township of Stokes Valley lies within a relatively flat valley ranging from elevations of ~30 to 90m above sea level (asl), while the surrounding slopes rise to elevations of ~200 to 400m asl. There are several small natural channels that contribute to the main Stokes Valley Stream, many of these flow through private property and have private structures such as driveways and footbridges crossing the channels.

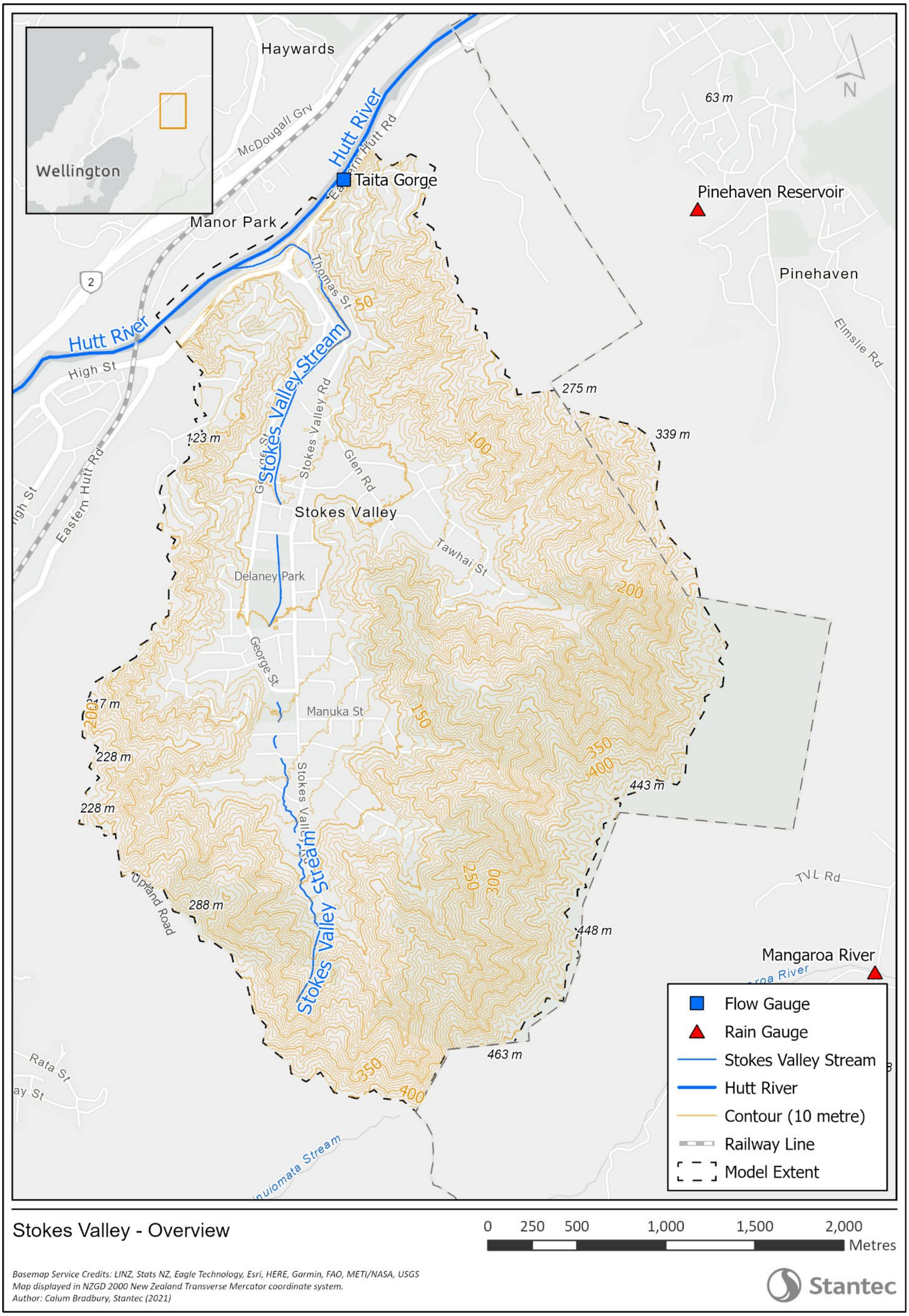


Figure 1-1: Overview of the Stokes Valley catchment.

1.3 ACTIVITIES AND SCOPE

The objective of this work is to develop a comprehensive one-dimensional/two-dimensional (1D-2D) model of the Stokes Valley stormwater network for use by Wellington Water and Hutt City Council (HCC). Primary uses of the model will include the development of minimum building floor levels for future development in Stokes Valley, as well as optioneering network upgrades to mitigate existing stormwater flooding. The following activities are discussed in detail in this report:

1. Develop a fully integrated 1D/2D hydrological and hydraulic model following the Regional Stormwater Hydraulic Modelling Specifications V5 (Wellington Water Ltd, 2017). All public stormwater assets including the 1D pipe network, 1D channel network, and the 2D ground surface are to be incorporated into the model. Design events including the 10yr ARI with existing climate, and the 100yr ARI with existing and future climate would be simulated with the base model.
2. Validate the model using reported flooding incidences and photographs for two storm events. It was agreed with WWL that the 15 November 2016 and the 8 December 2019 events would be simulated and analysed.
3. Undertake sensitivity analysis on the model to understand the impact of different model parameters on flooding in Stokes Valley. These sensitivity scenarios would be used to inform the selection of freeboard values to be applied to the network before the publishing of flood maps.
4. Undertake the freeboard assigning process following the Dynamic Freeboard Analysis (Jacobs, 2017) memorandum and simulate the freeboard run.
5. Develop a report outlining the model build process including all relevant model details, the validation process and findings and the sensitivity and freeboard process and findings.

2. AVAILABLE INFORMATION

2.1 PREVIOUS WORK

In May 2015, GHD was commissioned by WWL to carry out a scoping study for the development of a hydraulic model for the Stokes Valley catchment. GHD completed a Rapid Flood Hazard Assessment using MIKE URBAN, MIKE11, and MIKE21 and provided a review of the available GIS data for the stormwater network in Stokes Valley. Key findings and recommendations from this study were as follows:

- No flow data were available for the Stokes Valley stream
- Data from four rain gauges in the vicinity of Stokes Valley are available
- Installation of a flow gauge in the Stokes Valley Stream was recommended
- Identified 869 manholes either missing lid or invert levels: 18 missing both
- Identified 6 x stormwater pipes with missing diameters (out of 2176)
- Identified 61 pipes with no Type assigned
- Identified 699 sump nodes missing from upstream of pipes categorised as sump type
- Recommended lid levels of sumps estimated from DTM
- Identified 180 inlets/inlet structures with missing invert levels
- Identified 73 outlet/outlet structures with missing invert levels
- Identified no readily available data on bridge structures in Stokes Valley
- Recommended survey to determine bridge dimensions and levels
- Identified multiple streams in addition to Stokes Valley Stream
- Recommended surveying number of cross sections throughout the catchment
- Identified Evans and Bowers Street as susceptible flooding areas
- Other problem areas included Rawhiti Street, Glen Road, Horoeka Street, Maru Street, Tawhai Street, Hawthorn Crescent, Stokes Valley Road, Richard Grove, Hanson Drive, George Street, Rintoul Grove, Poppy Watts Grove, Chittick Street, and Raukawa Street

A comparison between the results of the RFHA and results of the Stokes Valley ICM model will be made in the calibration and validation phase of the project.

2.2 DRAINAGE NETWORK DATA

2.2.1 Asset Data

The following GIS layers for stormwater assets are available from WWL:

- Stormwater_nodes
- Stormwater_pipes
- Stormwater_sumps
- Stormwater_lateral_nodes
- Stormwater_pipes_private

This asset data was provided by WWL in 2016 in ESRI shapefile format in New Zealand Transverse Mercator 2000 geographic projection. Table 2-1 and

Table 2-2 provide a breakdown of assets and some of the missing information. The information was taken from the Stokes Valley Scoping Study undertaken by GHD in 2015 (GHD, 2015)

Table 2-1: Manhole Types and Missing Data

Category	Number	Missing Lid Level	Missing Invert Level	Missing Lid and Invert Levels
Manhole	1149	64	13	362
Sump	125	4	-	121
Inlet	177	-	167	-
Inlet Structure	24	-	13	-
Outlet	71	-	63	-
Outlet Structure	15	-	10	-
Inspection Point	3	-	-	3
Chamber	10	-	1	9
Node	1	-	-	1
Dead End Cap	4	-	-	4
Total	**1579**		**267**	**500**

Table 2-2: Number of Each Pipe Type

Type	Number
Sump	699
Scour	1
Overflow	3
Gravity	1416
No Type Assigned	61
Total	**2180**

2.2.2 Topographical Data

Topographical data was sourced from HCC by WWL for use in the development of hydraulic models for the Panel. The topographical data was supplied as LiDAR points from which a Digital Terrain Model (DTM) with 1m x 1m cells was generated for use in the model. The LiDAR was collected in 2013 with the Wellington 1953 vertical datum. The model build process was undertaken prior to the 2016 LiDAR becoming available. It has not been agreed to update to the newer data at this stage. For further information on the available LiDAR refer to Ground Model Assessment Summary Report (MWH (Now Stantec), 2016) submitted in April 2016 to Wellington Water.

2.2.3 As-Built Data

As-built plans have been reviewed to obtain additional asset information. Any data obtained from as-built plans are flagged as such. There were ~25 as-built plans with information that was used in the model build. These were particularly useful for the stormwater network surrounding Tawhai Street, Horoeka Street, and parts of Stokes Valley Road. As-builts were predominantly used for invert levels of pipes, manholes and to confirm the connectivity of missing sump pipes.

2.2.4 Survey Data

Survey of stormwater structures and cross sections was undertaken to provide additional information for the model build. All structures in the main Stokes Valley Stream channel were surveyed along with cross sections in accessible tributaries of the main channel. The locations of cross sections and structures included in the survey are included in Appendix A.

2.2.5 Reported Flooding Issues

Stokes Valley has been prone to flooding in the past due to the steep terrain that results in short critical duration for the peak flows in the catchment. In 2004 there were reports of flooding on Manuka Street. There was also a large flood on 20th December 1976 that caused widespread flooding and slips. Significant flood events in 2016 and in 2019 are used in the validation of the model, discussed in Section 4. Notable areas for flooding in Stokes Valley include Manuka Street, Tawhai Street, George Street, Glen Road, Horoeka Street, and Rawhiti Street. The following figures show some of the flooding that a has occurred in the Stokes Valley catchment.

Figure 2-1: 113 Manuka Street 16/02/04.

Figure 2-2: Manuka Street 16/02/04.

Figure 2-3: 108 Manuka Street 16/02/04.

Figure 2-4: 108 Manuka Street 16/02/04.

Figure 2-5: Driveway of 27 Manuka St ca 20/12/1976.

Figure 2-6: Driveway of 27 Manuka St ca 20/12/1976.

Figure 2-7: Looking down Manuka St towards the fire station in the distance ca 20/12/1976

Figure 2-8: In front of 18 Hanson Grove ca 13/03/1990 (*source National Library website)*

2.3 HYDROLOGIC/HYDROMETRIC DATA

2.3.1 Rainfall Data

Rainfall data is available as both point source in the form of rain gauge records, and spatially averaged from rainfall radar data. Both sources of rainfall data are discussed further in Section 4 following their analysis in the model validation.

2.3.1.1 Gauge Data

The closest rainfall gauge to the Stokes Valley catchment is the Mangaroa River at Tasman Vaccine gauge, which has been operated by Greater Wellington Regional Council since 1980. It is located at the top of the Mangaroa catchment approximately 2km east of the top end the Stokes Valley catchment. A gauge is also operated by Greater Wellington Regional Council at Pinehaven Reservoir, with data recorded from 2010. There are two additional rain gauges located along the periphery of the Stokes Valley catchment including the Taita and Rata Street gauges. Gauge locations are shown in Figure 2-9.

2.3.1.2 Rainfall Radar

Regional rainfall temporal profiles with a 5-min time step from 2008 onwards were available for download from a Mott Macdonald interface. Although this rainfall radar database covers the Wellington region and records cloud moisture content at a 500m x 500m resolution, only averaged rainfall across large watersheds was available for input. However, as shown in Figure 2-9, the Stokes Valley rainfall radar catchment corresponds well to the model extent. For details on how rainfall was applied to the model, see Section 4. The Rainfall Gauge polygons shown are split by the contour line at the elevation of the Mangaroa River rainfall gauge. The Mangaroa River rainfall gauge is used for anything above 240m and the Pinehaven rainfall gauge is used for anything below 240m.

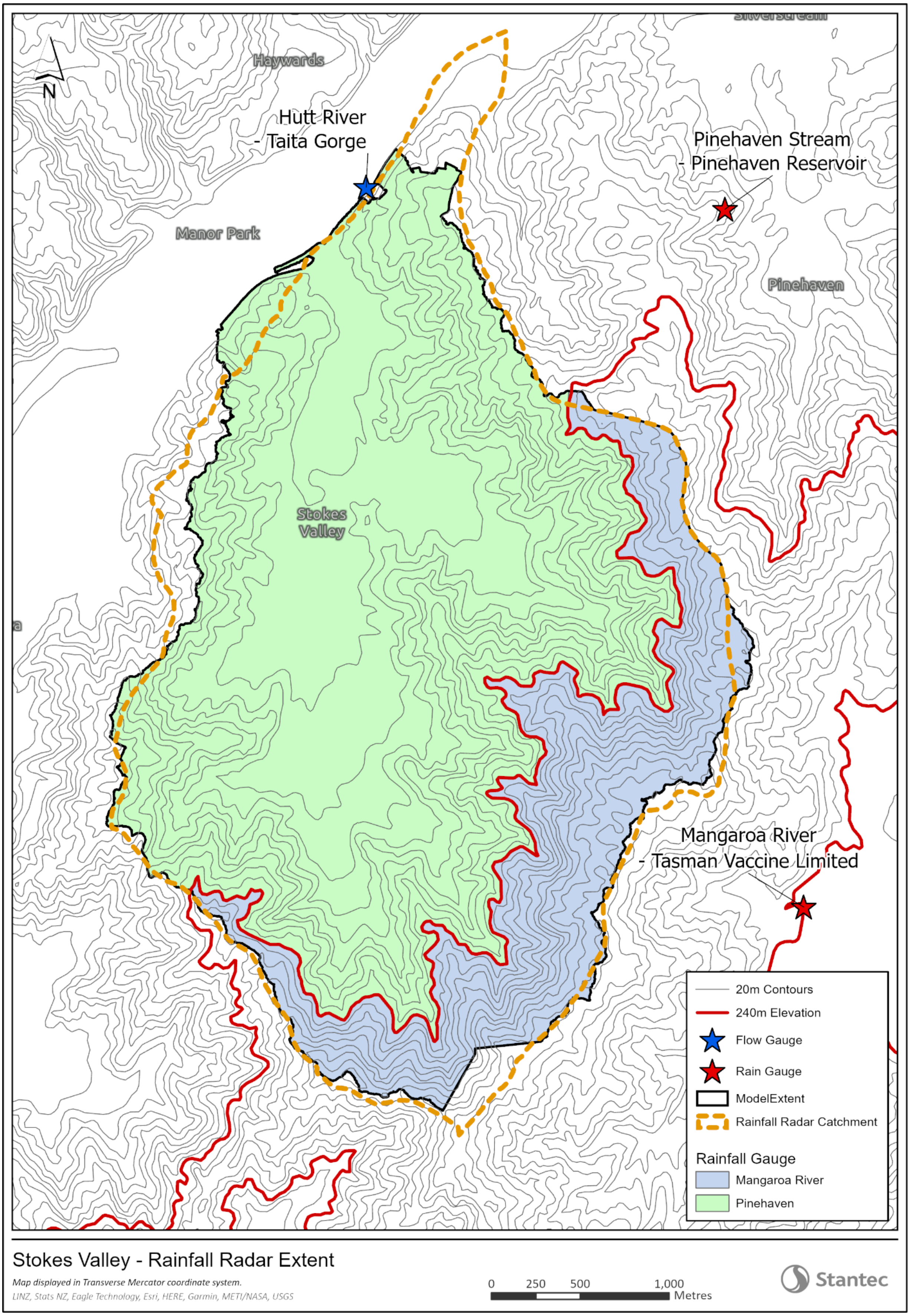


Figure 2-9: Rainfall radar catchment extent, flow and rain gauge locations and model extent.

2.3.2 Flow and Water Level Data

There are no recorded flow or level data for the Stokes Valley Stream. There is a flow and level recorder on the Hutt River which the Stokes Valley Stream discharges to. The Hutt River at Taita Gorge gauge is located approximately 900m upstream of the Stokes Valley Stream/Hutt River confluence and has been operating since 1980. This information is used in the model validation to represent Hutt River flow events.

2.3.3 Tidal Data

The Stokes Valley catchment is not expected to be influenced by tidal levels as it is ~30m above sea level and shares no boundary with the sea.

2.3.4 Regional Hydrology Layers

Regional hydrology layers for the Wellington region consisting of curve number, initial abstraction (Appendix B:), percentage impervious (%IMP, and Manning's N roughness (Section 3.3.4.3) are available for use. The layers were developed for WWL using GIS software in 2016 by MWH (now Stantec), based on the hydrology guidelines provided in the Quick Reference Guide for Design Storm Hydrology (Cardno, 2016). The layers were formed using land use, soil classifications, and topography and their application to the model is discussed in Section 3.2.2.

3. MODEL BUILD

3.1 MODEL BUILD OVERVIEW

A fully integrated 1D/2D model has been developed for Stokes Valley. The 1-D/2-D coupled model includes both a hydrological model for the conversion of rainfall into runoff, and a hydraulic model for conveyance of this runoff across the surface and through the reticulated stormwater network. The software used in the initial development of the Stokes Valley Stormwater model was Urban Model Manager (UMM), developed by Awa Environmental. The UMM model was then exported and edited further within InfoWorks ICM Version 6.5, developed by Innovyze. The model has more recently been updated to v9.5 and all recent works have been undertaken in this version, including validation and sensitivity. InfoWorks ICM (Integrated Catchment Modelling) is an integrated modelling platform that incorporates both urban and river catchments.

The model build section described below has been broken into the three major model components: the hydrological model build, the hydraulic model build, and the modelled boundary conditions.

The following projection and vertical datum have been adopted:

- **Projection** – NZGD 2000 New Zealand Transverse Mercator
- **Vertical Datum** – Wellington Vertical Datum 1953

3.2 HYDROLOGICAL MODEL

As per the Wellington Water Regional Stormwater Modelling Specifications v5 (Wellington Water Ltd, 2017) and the Quick Reference Guide for Design Storm Hydrology (Cardno 2016), the Soil Conservation Service (SCS) method is used as the hydrological model for catchment runoff estimation. This method requires catchment area, curve number (CN), initial abstraction (IA), and time of concentration (ToC). Each parameter and its application is described in the following sections.

3.2.1 Sub-Catchment Delineation

The sub-catchments used in the hydrological model to calculate inflow to the hydraulic model have been generated within UMM using a combination of custom-built tools and Arc Hydro tools available in ArcGIS. 2020 sub-catchments have been delineated from the DEM, draining to all sump and inlet locations as shown in Figure 3-1.

1,020 of these are the boundaries around buildings that have been included as individual sub-catchments that drain directly to the nearest manhole. These were selected based on a GIS process to determine where private assets were likely connected to the main stormwater network, as opposed to discharging to the roadside kerb.

Some sub-catchments on the eastern boundary of Stokes Valley are slightly misrepresented due to the boundary edge of the Hutt City Council 2013 LiDAR. It was agreed that due to the minimal difference it would have made to the curve number and initial abstraction values (discussed in the following sections) generated in UMM, the sub-catchments were delineated and parameterised using only the HCC 1m DEM, 2013 instead of clipping it to the GWRC 1m DEM, 2013. These catchments were then digitised manually outside of UMM, and the correct total contributing areas (m2) were input into the ICM model. Figure 3-1 below shows the final modified sub-catchment delineations.

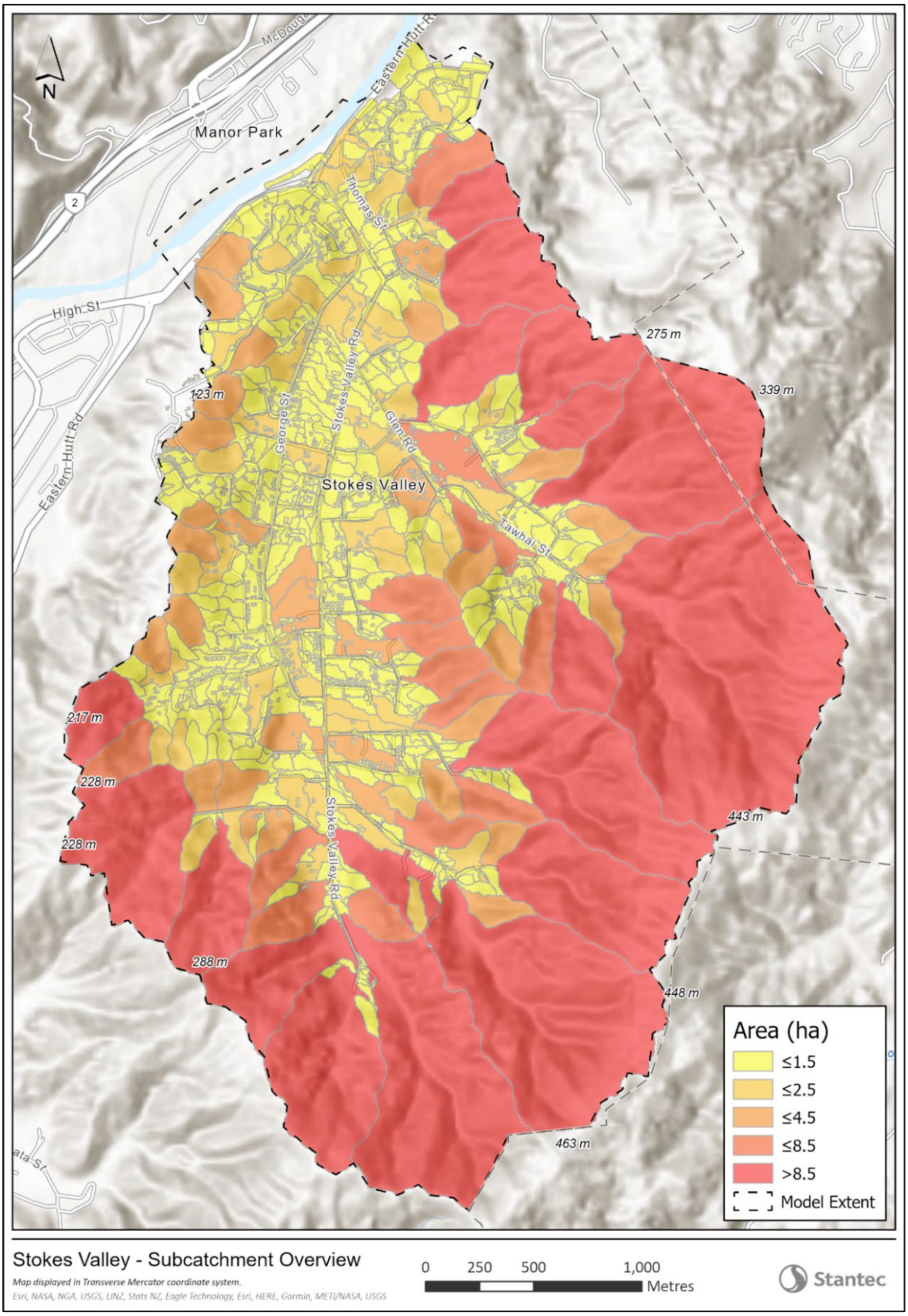


Figure 3-1: UMM Generated sub-catchment boundaries.

3.2.2 Sub-Catchment Hydrology

As per the Wellington Water Regional Stormwater Modelling Specifications v5 (Wellington Water Ltd, 2017) and the Quick Reference Guide for Design Storm Hydrology (Cardno, 2016), the Soil Conservation Service (SCS) method is used as the hydrological model for catchment runoff estimation. This method requires catchment area, curve number (CN), initial abstraction (IA), and time of concentration (ToC). Each parameter and its application is described in the following sections.

As per other Panel models, the Regional Hydrology Layers were analysed for any discrepancies/errors where the incorrect CN, IA, or roughness values were applied. Following analysis, no necessary updates were made to the Regional Hydrology Layers in the Stokes Valley catchment.

3.2.2.1 Curve Number

Curve numbers were assigned for each sub-catchment based on the Regional Curve Number Layer (discussed in section 2.3.4). A single composite value was determined for each sub-catchment using zonal statistics in ArcGIS with the sub-catchment polygons and the Regional Curve Number layer (Figure 3-2).

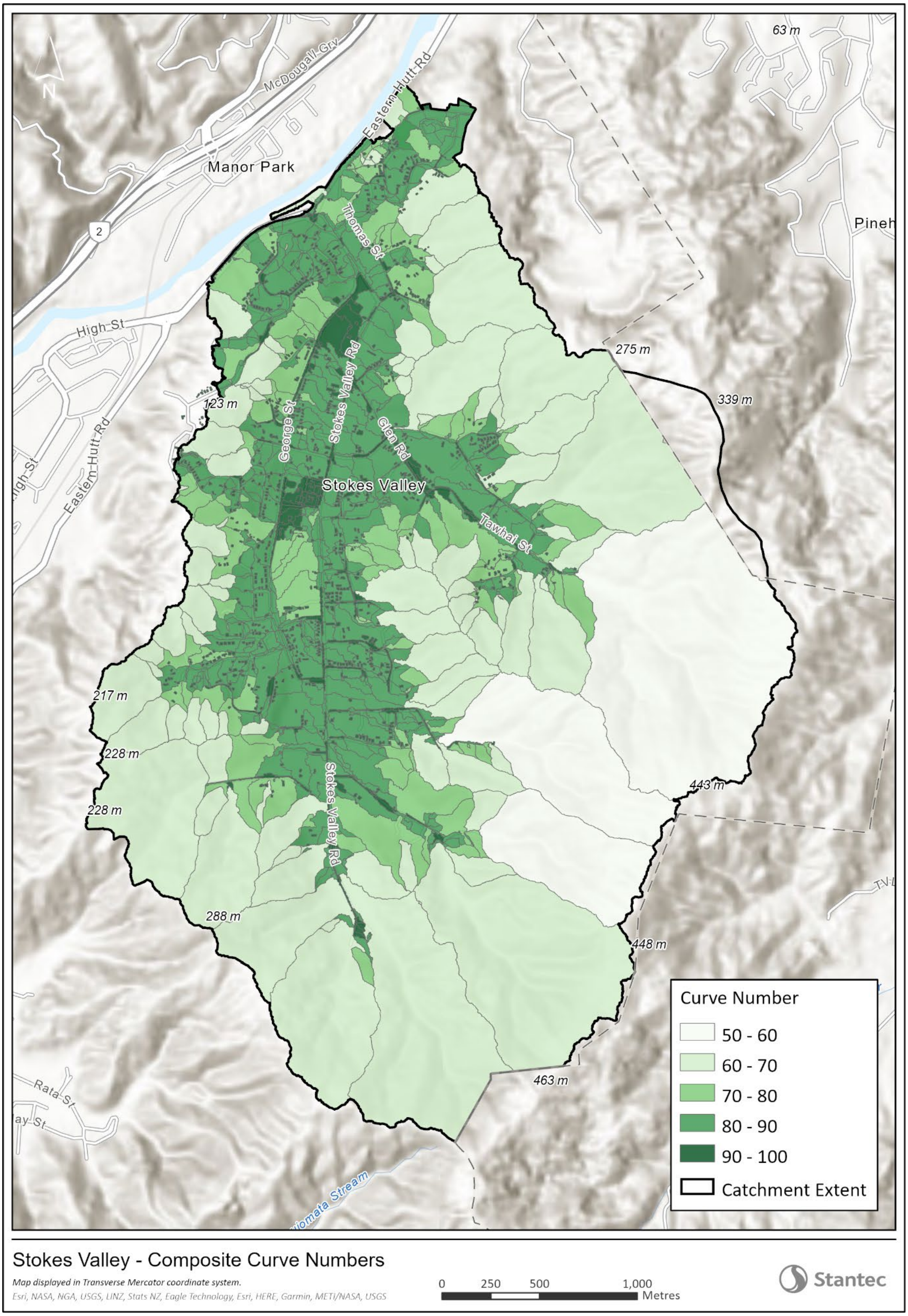


Figure 3-2: Composite Curve Numbers applied to each sub-catchment.

3.2.2.2 Initial Abstraction

As per the curve numbers above, a single IA value was determined for each subcatchment using zonal statistics in ArcGIS with the sub-catchment polygons and the Regional Initial Abstraction layer (Figure 3-3).

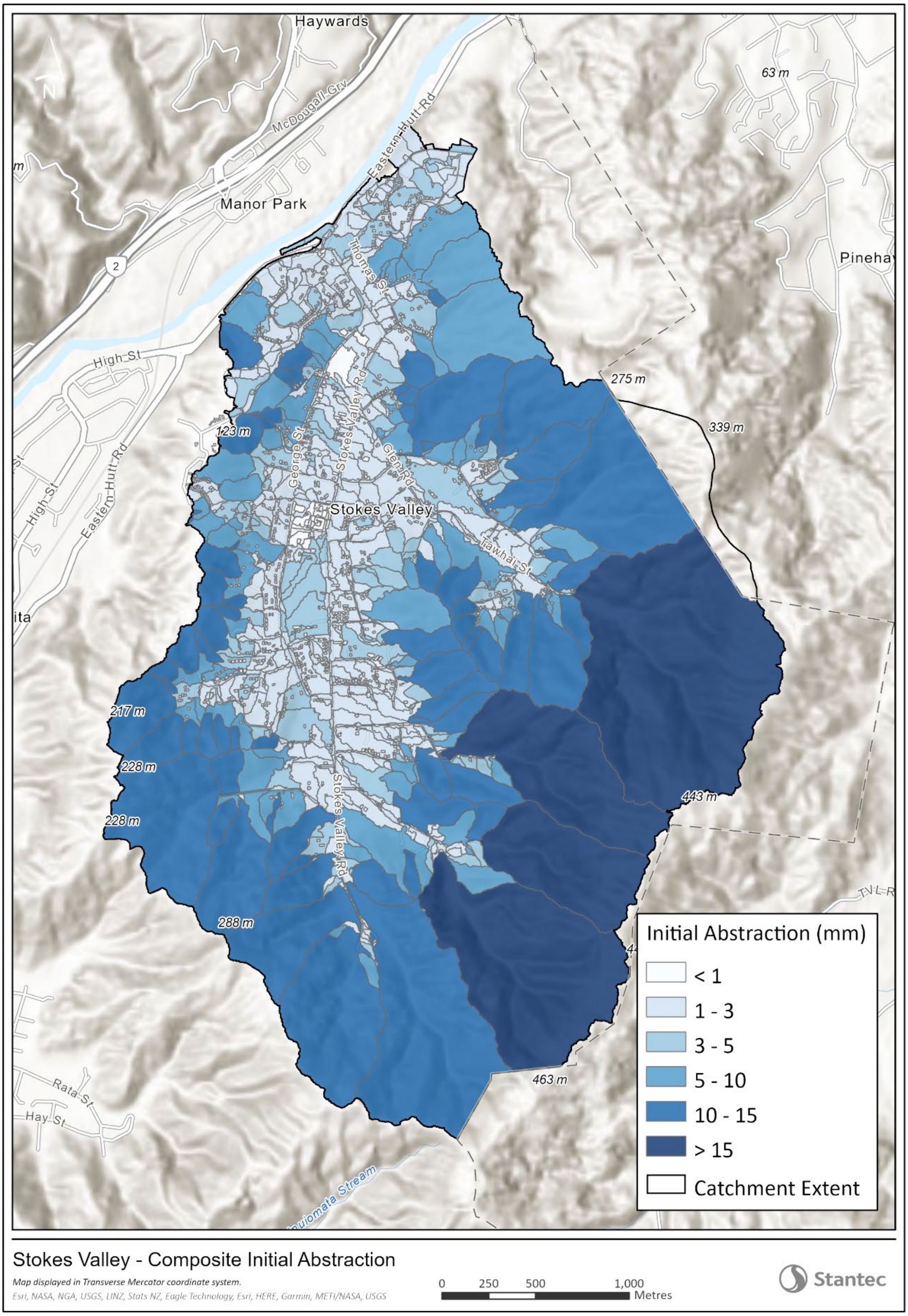


Figure 3-3: Composite Initial Abstraction values applied to each sub-catchment.

3.2.2.3 Time of Concentration (ToC)

The WWL Hydrology Quick Reference Guide (Cardno, 2016) outlines the required components of runoff and associated time of concentration (ToC) calculations required for estimating a time of concentration for a whole model sub-catchment. A summary of a practical approach to applying these different components in the context of a model is provided in the WWL Modelling Specifications V5 (Wellington Water Ltd, 2017). This approach can be summarised as follows:

- Sheet flow up to 50m
- Shallow overland flow up to 150m
- Gutter flow
- Pipe flow

Prior to the calculation of the various components of ToC the longest flow path (LFP) for each catchment is required. These were generated for each model catchment using the ArcHydro tools extension in ESRI ArcMap. Once created, the slope and average surface roughness for the LFPs were calculated using GIS processes in ArcMap. The WWL Regional Roughness Layer (see section 2.3.4) developed for the Panel models was used to calculate the roughness values for each LFP. The LFP lines were then intersected with the road polygons and extracted from the Regional Roughness Layer to help establish at what distance along each line the gutter flow ToC calculation was to be initiated.

For the sheet flow ToC calculations, the first 50m (or less if the LFP was shorter than 50m) of each LFP was extracted and separate slope and roughness values were calculated. These values were then used to define whether the length of sheet flow for each sub-catchment was 20m or 50m. If the slope of a line was 10% or greater and the average roughness along the line was greater than 0.045 then a length of 20m was adopted for sheet flow in a sub-catchment (unless the LFP was less than 20m). If the above condition was not met, then a length of 50m was adopted for sheet flow (unless the LFP was less than 50m). The exception to the rule set out above was if a sub-catchment LFP began within a road polygon, in which case a sheet flow length of 6m was adopted (Awa, 2017).

The length of LFP adopted for the shallow overland Flow ToC equation was calculated by subtracting the remainder of LFP length minus sheet flow length from the remainder of LFP length minus the length to gutter flow. However, if the result of this was greater than 150m then a length of 150m was adopted for the Shallow Overland Flow calculation (Awa, 2017).

For the gutter flow ToC calculation, the length adopted depended on how far along an LFP (if at all) a road polygon was intersected. If an LFP began in a road polygon, then the length was the length of the LFP minus 6m. If an LFP began outside of a road polygon, then the length adopted for gutter flow was either the length of the LFP within the road polygon it intersected, or the remainder (if any) of LFP length minus sheet flow and 150m (maximum allowable shallow overland flow).

Figure 3-4 below provides an example of the various components applied in the ToC calculations. The Stokes Valley model has been built based on the WWL modelling specifications so sub-catchments were drawn to sumps and inlets. As a result, no pipe flow has been included in the ToC concentrations.

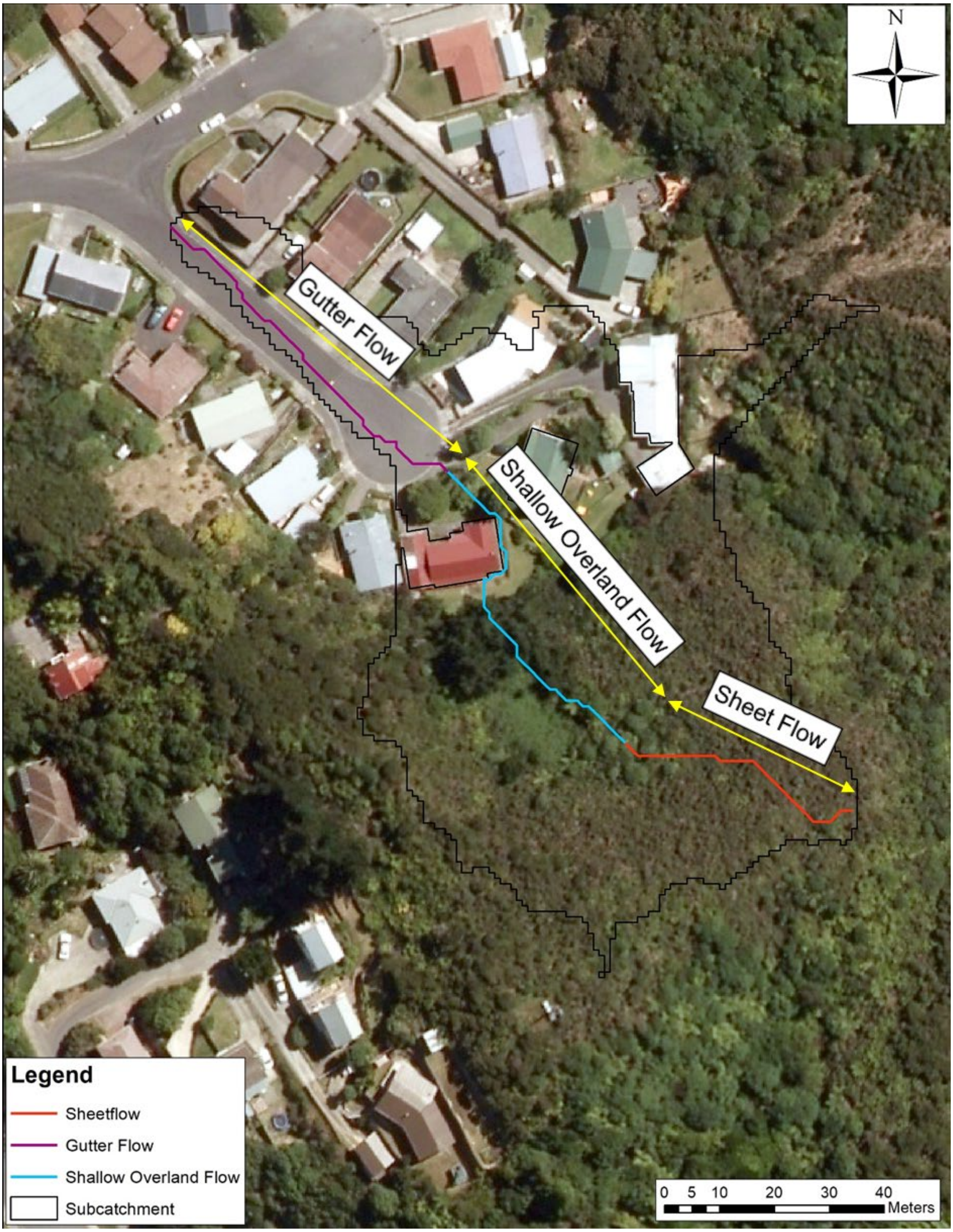


Figure 3-4 Time of concentration components of a sub-catchment longest flow path.

3.3 HYDRAULIC MODEL

3.3.1 Hydraulic Model Overview

The hydraulic model is used to simulate hydraulic processes within the stormwater network. It covers conveyance of sub-catchment generated runoff from the hydrological model across the surface and through the reticulated stormwater network.

The hydraulic model is made up of the following:

- 1-D components including the reticulated pipe network and open channels (Sections 3.3.2 and 3.3.3).
- 2-D components as defined by the flexible 2-D mesh area that is generated using the ground model for surface flow hydraulic calculations (Section 3.3.4)
- 1-D/2-D inter-connections including sumps, manholes, bank lines, and inline banks (Section 0).

Figure 3-5 provides an overview to the extent of the hydraulic model components.

As the asset data received had many missing attributes, an interpolation process was conducted in UMM to assume asset information based on set rules outlined in the Wellington Water Stormwater Modelling Specifications (Wellington Water Ltd, 2017). As-built drawings were used where available, particularly for complex structures such as bridges, channel crossing, and inlet structures. Where data was unavailable, engineering judgement was used for many of the edits. For further details on the rules and assumptions applied in UMM, refer to the updated WWL Stormwater Hydraulic Modelling Specifications (Wellington Water Ltd, 2017).

The Stokes Valley Stream channel and majority of its tributary channels have been modelled using 1-D River Reaches based on cross sections from a mixture of surveyed data and the model ground model. A small number of local channels have been represented in the 2-D Zone using Mesh Level Zones to adjust the TIN mesh in the model. Mesh level zones were used in some locations where there was no available cross section information to better represent the capacity of the channel. This was due to poor representation of small channels in LiDAR data. The level of uncertainty associated with model results along with model limitations are described in sections 3.5, 3.5.2 and 3.5.2.

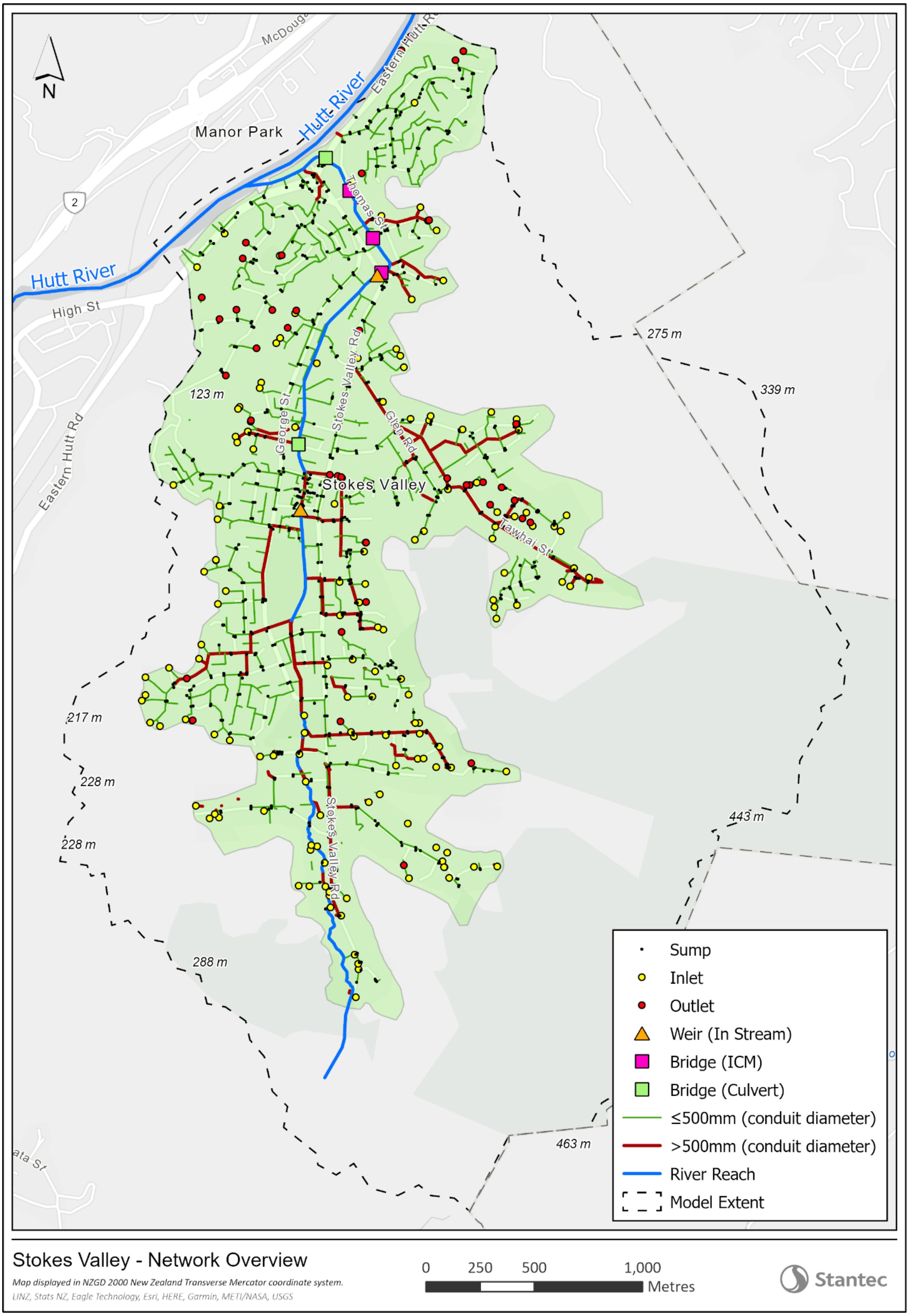


Figure 3-5: Overview of the Hydraulic Model Components.

3.3.2 1-D Reticulated Network

The 1D pipe network model is used to simulate the hydraulic processes in the stormwater network. The pipe network model includes all stormwater drainage pits, pipes and manholes in the catchment that could be identified from the GIS layers supplied, with the exception of small diameter private pipe assets.

The model includes some assets (discussed in section 2.2.3) that were identified from the as-built drawings, ortho-photos and site surveys undertaken by Stantec staff. A number of pipes were added to the model to connect drainage sumps to the network.

The following GIS layers for stormwater assets were supplied by WWL:

- Stormwater_nodes
- Stormwater_pipes
- Stormwater_sumps
- Stormwater_lateral_nodes
- Stormwater_pipes_private

While the data contains some errors and omissions the majority of these were able to be resolved with a reasonable degree of confidence by applying engineering judgement. Site survey of key infrastructure and cross sections has been undertaken where missing data could not be estimated with a sufficient degree of confidence. The degree of uncertainty associated with the model is related to the accuracy of input data and this needs to be considered when using model results in infrastructure design.

3.3.2.1 Node Ground Levels

Where node (Manholes, drainage sumps) ground levels are available in the GIS data supplied by WWL, these have been used in the model. The remaining node ground levels were extracted from the DEM.

3.3.2.2 Sumps

The invert levels for sumps have been based on the depth in the modelling specifications provided by WWL and the ground levels for the sumps updated from the DEM. All sumps have a minimum depth of 1.2m. The diameters for sumps have been calculated where:

- L = sump length
- W = sump width

Sump lengths and widths were taken from the modelling specification and in most cases are equal to 0.675m and 0.53m, respectively.

In instances where sump lead invert levels were not available an assumed upstream invert level 0.45m above the sump invert level and an assumed downstream invert level based on an assumed slope of 5% has been used. Where this would result in the downstream invert being below the invert of the downstream node or close to the ground level of the downstream node, the gradient has been adjusted to 1% or 10%. If the adjusted levels are still too low or too high the downstream invert has been set to the manhole invert or based on the ground level, pipe diameter and a minimum pipe cover of 100mm. Sump lead invert levels calculated using this methodology can be identified in the metadata using the "sump" code.

Sump inlet capacities and model properties are shown in Table 2-1 below. The parameters have been adopted from work carried out by DHI for Tauranga City Council where the coupling parameters have been calculated for each combination.

Table 3-1: Sump Inlet Capacities

Sump Characterisation			ICM Coupling Parameters		
Grate Type	**Kerb Opening**	**Sump Chamber**	**Inlet Area (m^2)**	**Discharge Coefficient**	**QdH Factor**
Standard	Open	Single	0.122	0.68	0.25
Standard	Inline	Single	0.122	0.67	0.3
Standard	Inline	Double	0.243	0.67	0.3
Standard	Recessed	Single	0.122	0.67	0.3
Standard	Back Entry	Single	0.063	0.67	0.3
Standard	Combination Inline	Single	0.1845	0.55	0.35
Standard	Combination Recessed	Single	0.1845	0.55	0.35
Standard	Combination Inline	Double	0.369	0.55	0.35
Super Pit	Inline	Single	0.353	0.62	0.4
Mega Pit	Inline or Recessed	Single	0.729	0.55	0.05

3.3.2.3 Pipe Connectivity

The model includes some assets (discussed in section 2.2.3) that were identified from the as-built drawings, ortho-photos and site survey undertaken by Stantec staff.

- The connectivity of the network was checked for issues; for example, pipes digitised in the wrong direction, pipes not connected to an outlet structure or bifurcations in the network.
- Errors that could be reasonably corrected without survey have been resolved using engineering judgement and or LiDAR information. This often involves adjusting link connectivity or splitting links and creating nodes.
- In the case of missing junctions, the split links have been named with the Asset ID of the original link suffixed with "_n", where n is an integer. The created node has been labelled with the Asset ID of the original link suffixed with "_a", or "_b" in the case where the link needs to be split multiple times.
- Similarly, where links do not terminate at a node, a node has been created and labelled with the links' Asset ID suffixed with "_a" or "_b"
- Where it was necessary to add a link to connect a drainage sump to the network, the link has been labelled with the Asset ID of the sump suffixed with "_1".

3.3.2.4 Pipe Invert Levels

- Where the WWL's GIS data for pipes contains upstream or downstream invert levels, these have been applied to the connected nodes.
- Where WWL's GIS node data contains depths and invert level data could not be extracted from the connected pipes, an invert level has been calculated from the depth and the ground level.
- If no other invert level data is available, the invert has been estimated based on the connected pipe diameters and an assumed pipe cover depth of 450mm.
- Where the invert levels used have resulted in negative slopes, the inverts have been adjusted manually.

- In a number of model locations it was found that pipes are surcharging and generating flooding from the network due to changes in pipe capacity. These changes in capacity have arisen due to changes in pipe gradient rather than changes in pipe diameter. A significant portion of the Stokes Valley network has been inferred from ground levels and/or inlet levels. In locations where flooding was being caused by this mechanism the source of the pipe invert levels along the network line was checked. If the invert levels were inferred they were adjusted to provide a more continuous/consistent grade along the length of network (Figure 3-6).

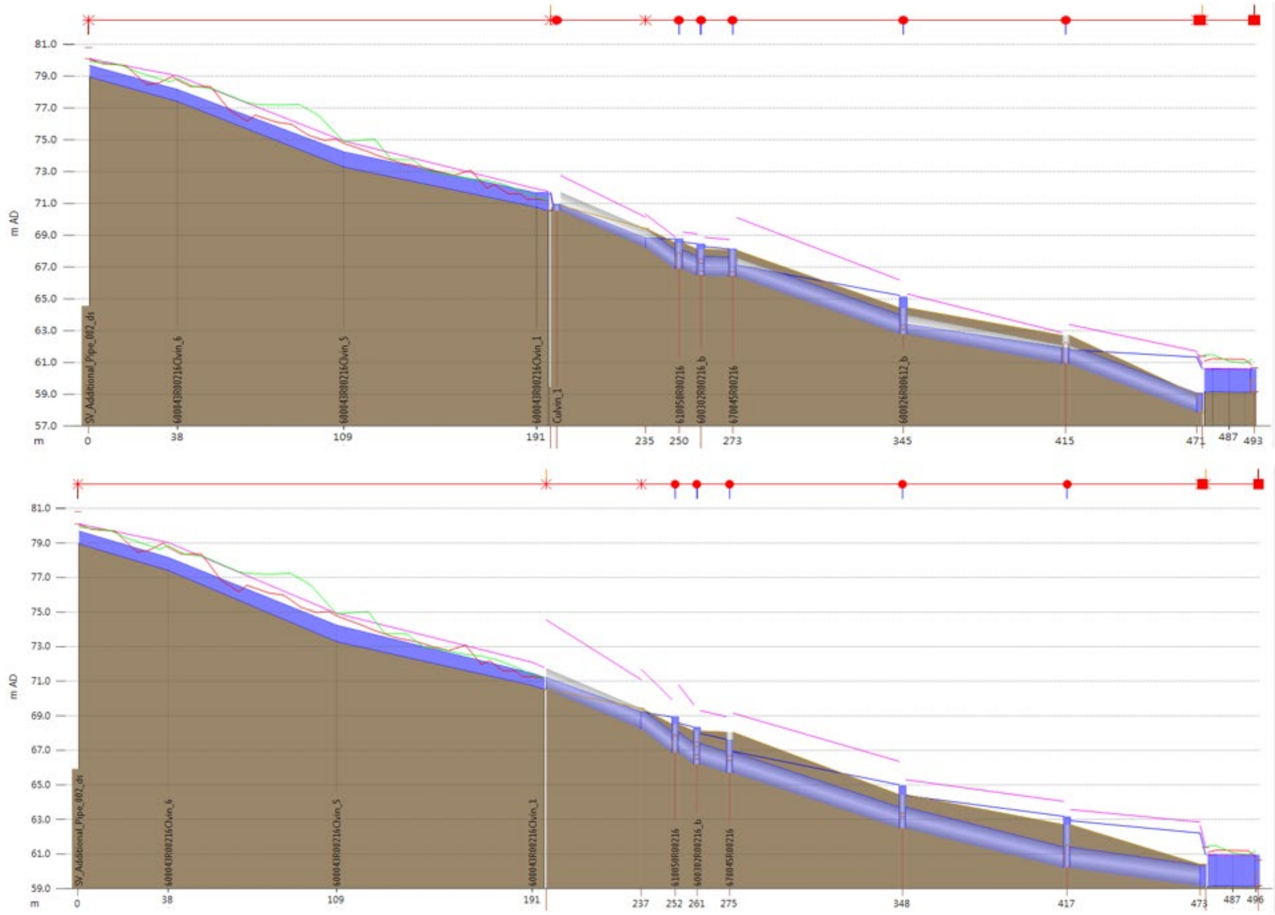


Figure 3-6: Initial and updated network from 373 Stokes Valley Road to Kamahi Park.

3.3.2.5 Pipe Losses

Energy losses in the 1-D pipe network due to surface friction have been accounted for using the pipe materials information in the WWL's GIS data for pipes where available. WWL's materials attributes were mapped to the ICM roughness values shown in Table 3-2.

Table 3-2: Pipe Roughness Values

Materials Type	Manning's n
Cement Mortar	77
Ceramics	70
Concrete (Normal)	75
Concrete (Rough)	68
Concrete (Smooth)	85
Iron (cast)	70

Materials Type	Manning's n
Iron (wrought)	65
Plastic	80
Stone	80
Corrugated Metal	50
Asbestos Cement	77
Timbre	68
Storm Boss	70
Nexus	70
Nova Flow	65

Pipe headlosses have been inferred using the in-built tool within ICM. As per Innovyze advice, headlosses are removed from pipes with a gradient greater than 10%.

3.3.3 1-D Channel Network

3.3.3.1 1D River Reaches

There are numerous open channels within the Stokes Valley catchment. Figure 3-7 shows those included in the model. The majority of these have been modelled using 1D River Reaches, however a small number (~4) have been modelled by adjusting the 2D mesh using Mesh Level Zone polygons, as discussed in section 3.3.1. Small channels are not well represented in the available LiDAR often due to the extensive vegetation cover in Stokes Valley. For the Stokes Valley Stream most of the cross-sections applied in the model have been developed from surveyed sections collected for this project. However, for most the tributaries modelled, most cross-sections have been developed from the ground model and site photographs.

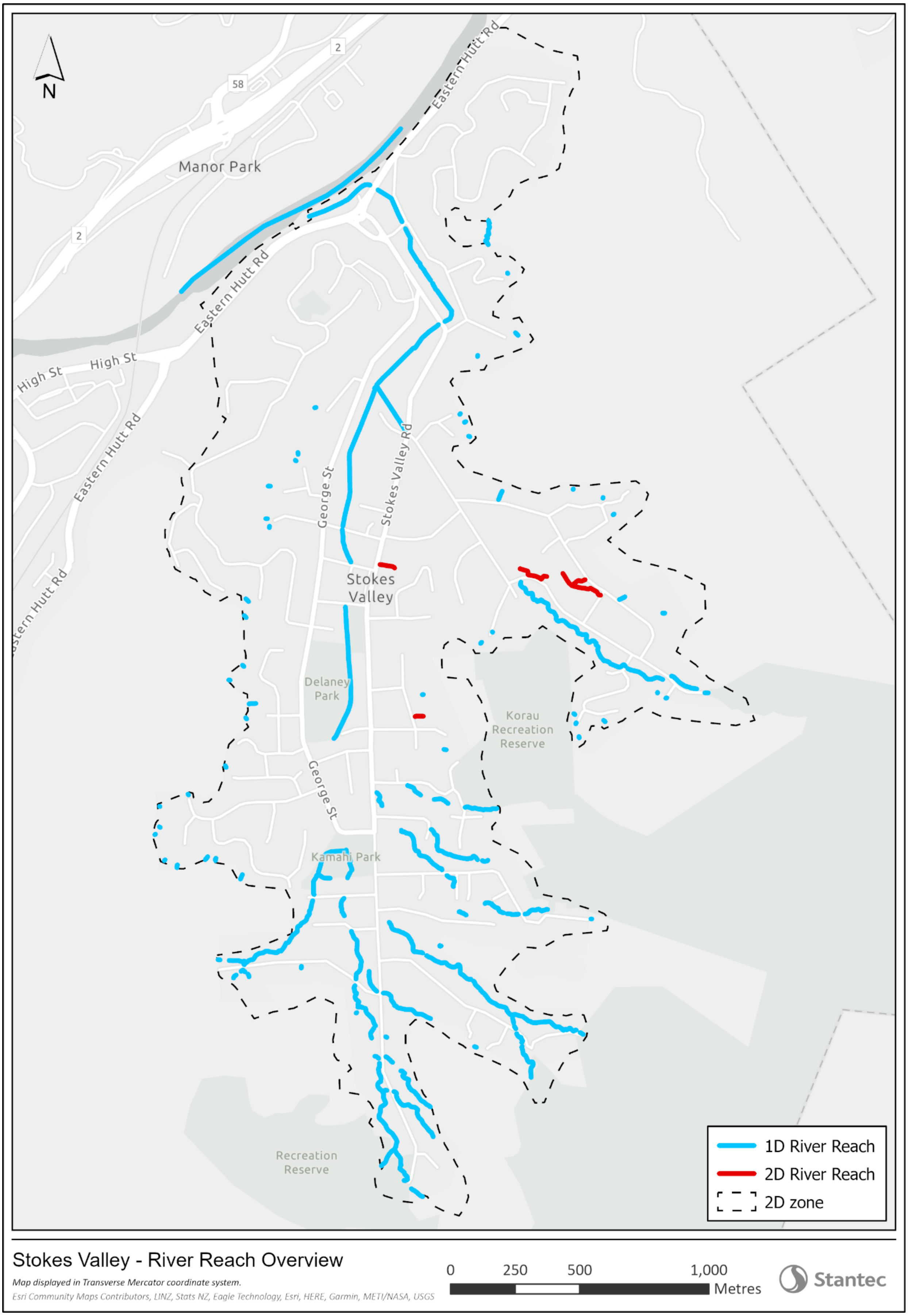


Figure 3-7: Extent of Modelled 1D River Reaches.

Along with the major channels shown in Figure 3-7, over 70 river reaches have been included at inlet locations for the piped network. These river reaches have been applied so runoff will have velocity when entering the piped network and as result the model will provide a better representation of energy losses at the inlets.

In total 191 river reaches have been developed with 865 cross sections. The length of each river reach has been defined by the location of structures on, or pipe outlets to, the channels. Break nodes have been placed at the upstream and downstream sections of the river reaches to allow for the connection of weirs from the pipe network outlets, lateral inflows from sub-catchments not included in the pipe network, or to mark the start or end of a structure. At most of the junctions, inlets, and outlets, storage nodes with associated storage have been used.

Each river reach uses two bank lines to connect to the adjacent 2-D zone. The bank lines have been generated from the ground model, unless cross section survey data was available. The alignment of the bank lines has been digitised to represent an appropriate top-of-bank location, pick up existing stop banks and any relevant features identified during the Stantec staff site visits. The discharge co-efficient values for the bank lines have been set at 0.8, representing the lower end of the range suggested for grass embankments. The modular limits for the bank lines have been set at 0.9, the ICM default.

3.3.3.2 1D Channel Structures

On Stokes Valley Stream downstream of Bowers St, 5 major stream crossings were identified and included in the model, three as bridge elements and two as culverts (Figure 3-8). The two crossings represented as culverts were the Rawhiti St crossing, which comprises twin box culverts and the Eastern Hutt Road crossing. Initial modelling included the Eastern Hutt Road crossing as a bridge element. It was not possible to achieve model stability this way and the nature of the crossing (Figure 3-9) meant the bridge could be represented using a culvert element with a user specified shape.

Two in-stream weirs were identified along the Stokes Valley Stream (Figure 3-10), the first immediately upstream of the Stokes Valley Rd Bridge, and the second approximately 100m upstream of the inlet at Evans St. Both weirs were surveyed and have been included in the model as in-stream weir structures.

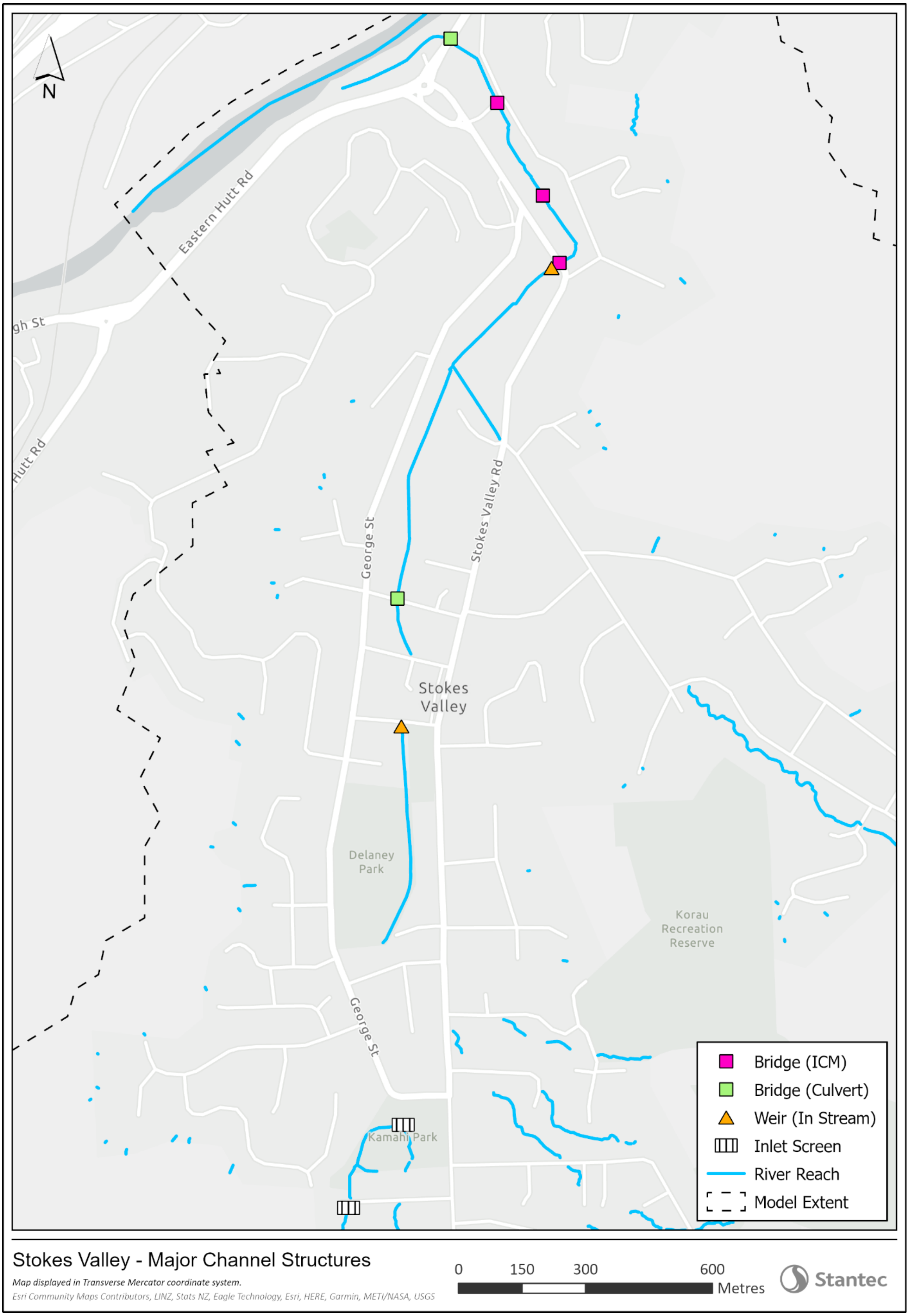


Figure 3-8: Overview of the major channel structures modelled.

Figure 3-9: The Eastern Hutt Road bridge, represented in the model as a culvert.

(i) **(ii)**

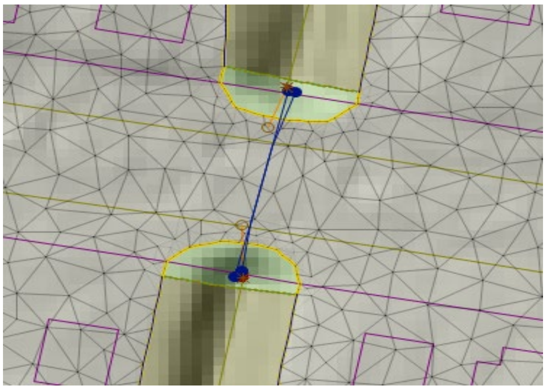
Figure 3-10: i) Weir immediately upstream of Stokes Valley Rd bridge; ii) Weir 100m upstream of Evans St inlet.

Two major inlet screens were also identified by Stantec staff and surveyed for inclusion in the model and are shown in Figure 3-11. The first screen is located at the confluence of two channels at an inlet on the downstream side of Kamahi Park. The second screen is a series of circular poles in front of an inlet for a culvert conveying a stream beneath Korau Grove.

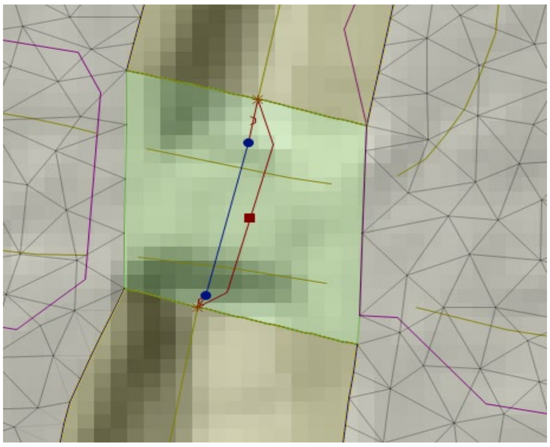
(i) (ii)

Figure 3-11: i) Kamahi Park Inlet screen; ii) Korau Gr Inlet screen.

At network inlet locations and road crossings, the overtopping of the inlet or crossing has been modelled using Inline Bank elements in the majority of cases and inline weirs in a small number of cases. In particular, for the Eastern Hutt Rd Bridge where the culvert element has been used to model the bridge structure, a weir has been used to model any overtopping flow. Figure 3-12 below, shows the model set up for both these scenarios.



(i)



(ii)

Figure 3-12: i) Inline banks used to model overtopping flow; ii) Weir used to model overtopping flow.

There were ~3 locations where surveys had provided levels and dimensions of fences that surrounded inlet structures. These were modelled using a porous wall with a porosity of 0.8.

3.3.3.3 1D Channel Losses

Energy losses in the 1-D channel network due to surface friction have been accounted for using the Manning's n Roughness Values specified in Table 3-3, taken from the Regional Specification, below.

Table 3-3: Typical 1-D Manning's n Roughness Values [1]

Classification	Manning's n Values
Open Channel	
Straight uniform channel in earth and gravel in good condition	0.0225
Unlined channel in earth and gravel with some bends and in fair condition	0.025
Channel with rough stony bed or with weeds on earth bank and natural streams with clean straight banks	0.03
Winding natural streams with generally clean bed but with some pools and shoals	0.035
Winding natural streams with irregular cross section and some obstruction with vegetation and debris	0.045
Irregular natural stream with some obstruction with vegetation and debris	0.06
Very irregular winding stream obstructed with significant overgrown vegetation and debris	0.1

All the channels in the network have been assigned a Manning's n value of 0.03 as this best agrees with the values in Table 3-3. An investigation of separate manning's n values for each river reach was not undertaken for the construction of the model. Channel, culvert, and bridge roughness values may be varied during the model validation phase as the model is altered to replicate as best possible recorded events.

3.3.3.4 1D Channel Structure Losses

For inlet and outlet losses at culvert inlets in the channel network the guidance provided in the ICM help section has been used and default values have been applied depending on the type of inlet or outlet (based on assessment of site photographs). Table 3-4 outlines the culvert inlet parameters used in the model.

Table 3-4 Culvert inlet parameters (CIRIA ref)

Description	Equation	K	M	c	Y	Inlet Headloss Coefficient (Ki)
Circular conduit/concrete/headwall/ square edge	A	0.0098	2.000	0.0398	0.67	0.50
Circular conduit/concrete/headwall /socket end of pipe	A	0.0078	2.000	0.0292	0.74	0.30
Rectangular conduit/concrete/ headwall/wingwall (30° – 75°)/square edge	A	0.0260	1.000	0.0385	0.81	0.30
Rectangular conduit/concrete/ headwall/wingwall (0°)/square edge	A	0.0610	0.750	0.0423	0.82	0.70

[1] Source: Table 5.8, pg 35 of Wellington Water Regional Stormwater Hydraulic Modelling Specifications

For the bridges, expansion and contraction losses have been calculated by ICM during the construction of each bridge element. Default values were considered appropriate for this level of assessment. The losses are calculated by multiplying a user defined loss coefficient by the change in velocity head between sections.

Initial Model results showed that there was a significant amount of flooding originating from some inlet locations (where open channels entered the reticulated network). Long sections of these transitions showed there was significant headloss across the culvert inlet elements at these locations causing excessive flooding. At the locations where significant headloss was identified (approximately 1.5m in Figure 3-13 the culvert inlet elements applied in the model were removed to allow less restricted flow into the pipe network or culverts.

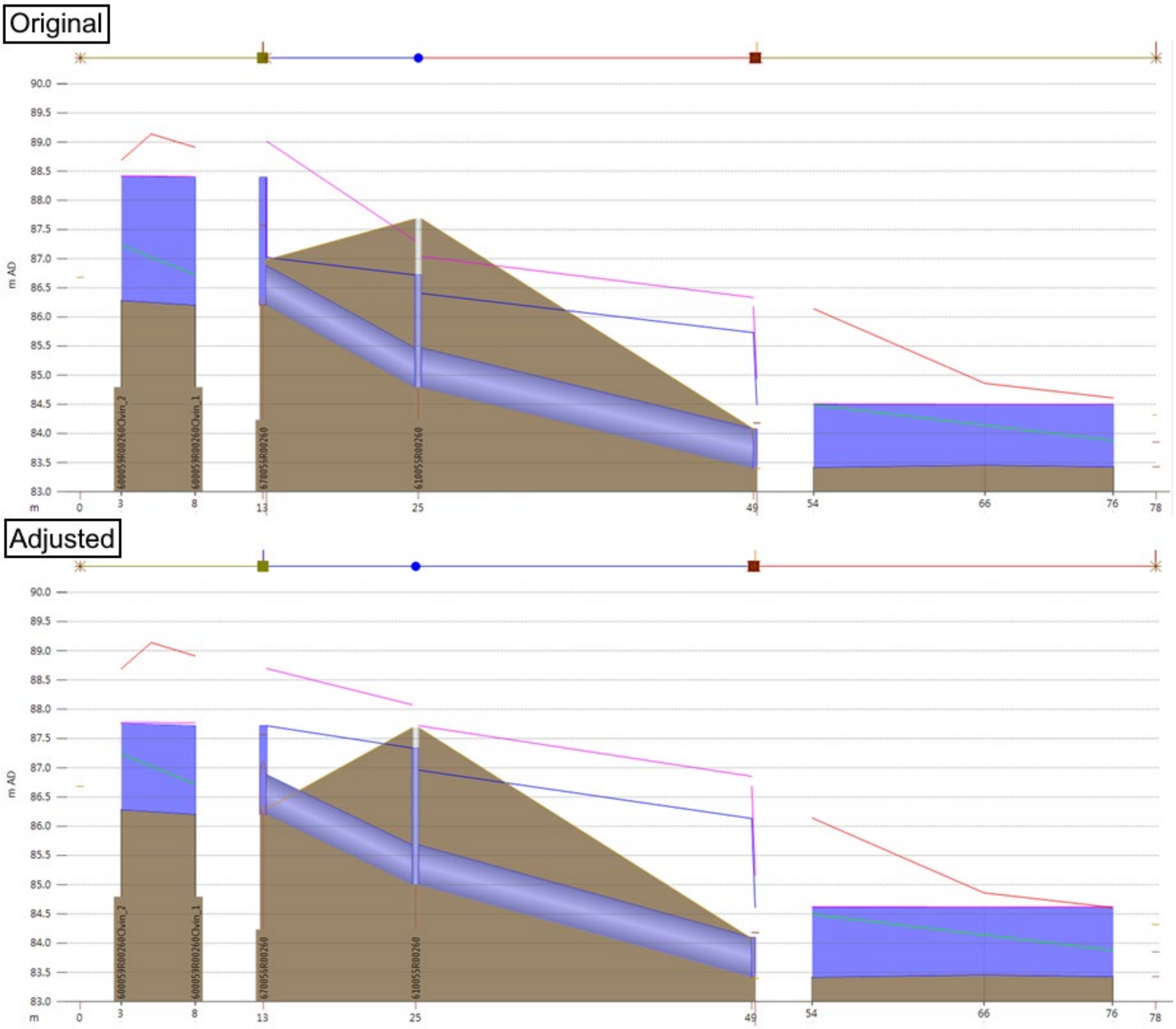


Figure 3-13: Initial and updated culvert inlet at 94 Tawhai Street.

3.3.4 2-D Network

3.3.4.1 2D Mesh

The extent of the 2-D network is shown in Figure 3-7. The 2-D zone developed for the model is approximately 4.09km^2 and covers most of the floor of Stokes Valley. The modelled 2-D Mesh Zone has been developed from the LiDAR generated Digital Terrain Model. It covers the valley floor up to the maximum extent of the residential area or piped network, whichever is further. The mesh triangles applied in the 2-D Zone have a minimum area of 2m^2, a maximum area of 4m^2, except in areas such as channels where greater detail has been applied, using maximum and minimum areas of 2m^2 and 1m^2, respectively. A minimum angle between vertices of 25º has been adopted. All buildings larger than 500m^2 were voided out of the mesh using simplified building footprints.

3.3.4.2 Mesh Zones and Mesh Level Zones

The 2D mesh has been manipulated using mesh zones and mesh levels zones to decrease the maximum triangle size (increase detail) and lower mesh elements to better represent overland flow paths in specific areas. Mesh zones have also been used to set ground level of the mesh elements containing a 2-D outfall node (where the 1-D pipe network discharges to the 2-D surface) to the level specified for the 2-D outfall node.

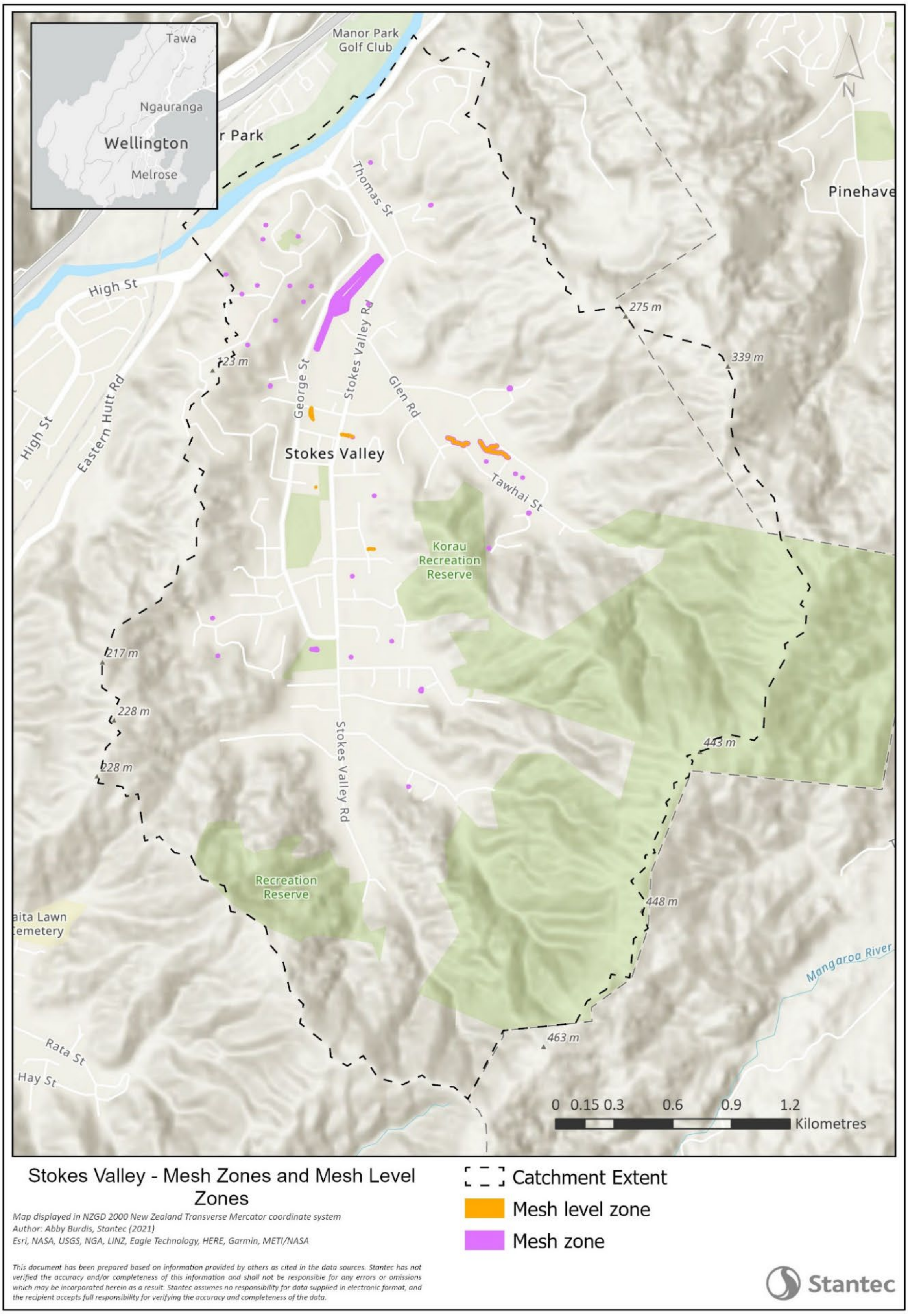


Figure 3-14: Locations of mesh zones and mesh levels zones applied to manipulate the 2D mesh.

3.3.4.3 2D Surface Roughness

Energy losses in the 2-D network due to surface friction have been accounted for using roughness zones to set Manning's n roughness values across the 2D Zone. The roughness zones have been adopted from the Regional Ground Roughness Coefficients work carried out by MWH, now part of Stantec, on behalf of the Panel. Table 3-5 and Figure 3-15 below show the roughness values applied to the different landuses identified across the Wellington Region.

Table 3-5: Manning's N Roughness Coefficients

Ground cover	Manning roughness coefficient n	Comment
Roads and footpaths	0.02	Upper limit from Regional Specification (value indicated by Wellington Water)
River	0.05	"Winding with some weeds and large stones" in Ven Te Chow table (value indicated by Wellington Water)
Vegetation: alpine	0.05	Maximum value of "rock cut channel, jagged and irregular" in Ven Te Chow table
Vegetation: bare	0.04	"Rock cut channel, jagged and irregular" in Ven Te Chow table (value indicated by Wellington Water)
Vegetation: forest	0.1	"Medium to dense brush in summer" in Ven Te Chow table (value indicated by Wellington Water)
Vegetation: impervious	0.05	"Impervious" here means water-logged (value indicated by Wellington Water)
Vegetation: pasture	0.05	Maximum value of "high grass" in Ven Te Chow table (value indicated by Wellington Water)
Vegetation: scrub/flax	0.08	Maximum value of "light brush and trees in summer" in Ven Te Chow table (value indicated by Wellington Water)
Vegetation: urban open space	0.05	"Urban open space" can be any of forest/open field/landscaped garden/pavement
Recreational area, playing field	0.05	Upper limit from Regional Specification
Non-residential properties: pavement	0.02	Upper limit from Regional Specification as assumed to be poorly maintained or have plantings and dividing barriers
Non-residential properties: building	0.5	Upper limit from Regional Specification
Residential properties: pavement	0.02	In line with Regional Specification (value indicated by Wellington Water)
Residential properties: grass	0.04	In line with Regional Specification (value indicated by Wellington Water)
Residential properties: trees	0.15	Upper limit from Regional Specification
Residential properties: small fenced backyards	0.1	
Residential properties: building	0.5	Upper limit from Regional Specification

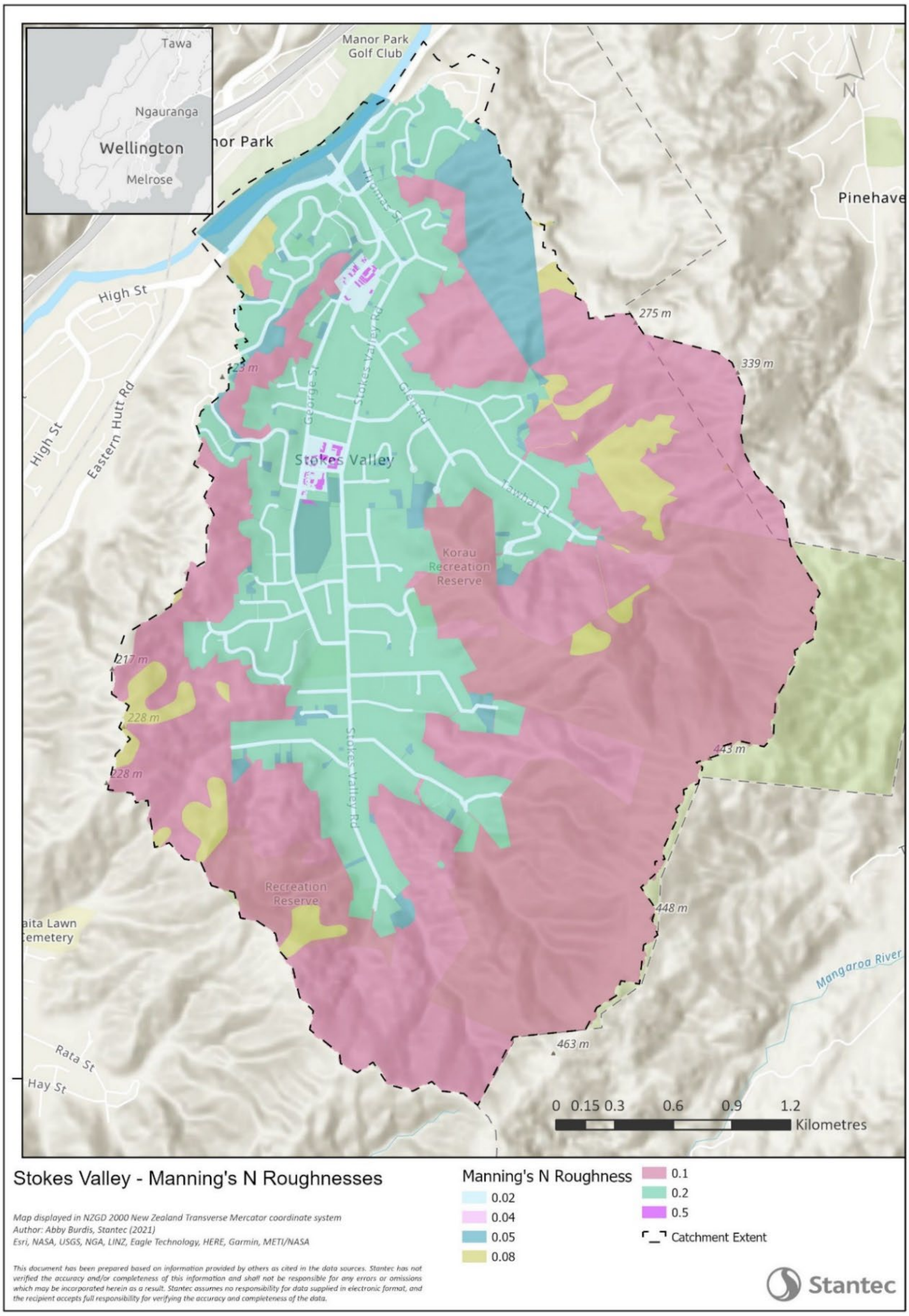
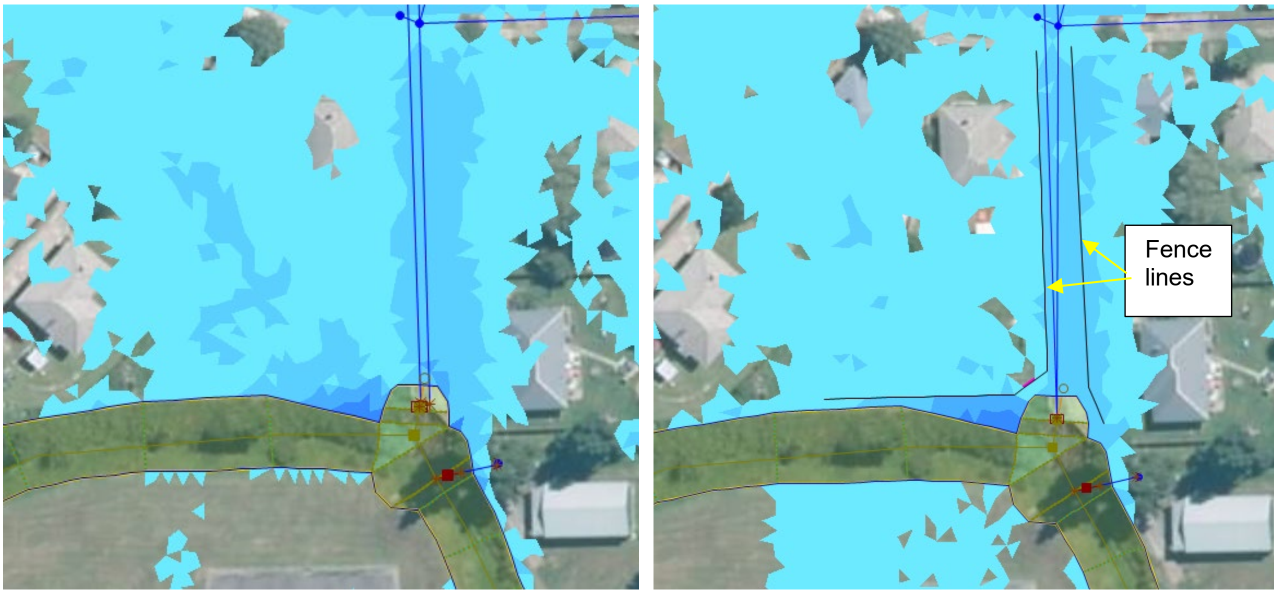


Figure 3-15: Manning's N Roughness values.

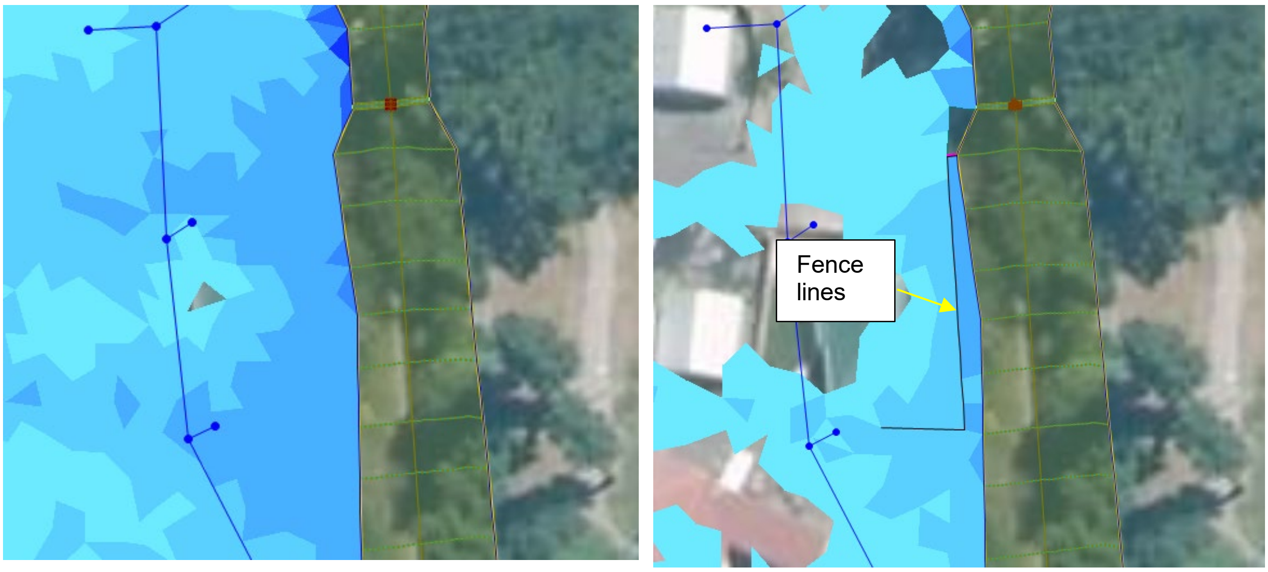
3.3.4.4 Fences

In two locations, fences were identified as potential barriers to overland flow originating from open channels and would likely help contain overland flow near the channels. To represent these fences in the model porous walls with a porosity of 0.1 have been included in the mesh. The first location is immediately downstream of the major network inlet at Kamahi Park (Figure 3-16) while the second is immediately upstream of the weir structure near Delaney Park (Figure 3-17). While the inclusion of the fences at these locations do not stop the overland flooding seen during the validation event, they do reduce it a little.



Initial *Updated*

Figure 3-16: Initial and updated network at the Kamahi Park grated inlet structure.



Initial *Updated*

Figure 3-17: Initial and updated network at the Delaney Park weir.

3.3.5 1D – 2D Connectivity

All modelled 1D and 2D structures are interconnected. In points of transition, such as bank lines and 2D nodes, flow is transferred between each domain in all directions. There are four key links between the 1D pipe network and the 2D surface in the Stokes Valley model including: 2D nodes, 2D Outfalls, Banklines, and Inline Banks.

3.3.5.1 2D Nodes and Sumps

Manholes with a connection to the 2-D surface are modelled using the “2D” flood type and sumps with a Flood Type “Gully 2D”. These nodes can transfer water to the 2D mesh elements they are located in. For “2D” Flood Type nodes, flow between the node and the 2D mesh element is calculated using a weir equation where the circumference of the node (manhole) is the weir width. For Flood Type “Gully 2D” flow is transferred between the 1D and 2D via a custom head discharge curve as per the WWL modelling specifications (Wellington Water Ltd, 2017)

3.3.5.2 2D Outfalls

At a few locations, the pipe network outlet discharges directly onto the 2D surface. At these locations, an Outfall 2D node has been used. Flow between Outfall 2D nodes and its corresponding 2D mesh element is calculated as a vortex control with a nominal head discharge relationship.

3.3.5.3 Bank lines

Each river reach uses two bank lines to connect to the adjacent 2-D zone. The bank lines have been generated from the ground model or surveyed cross section ends. Flow across bank lines is calculated using an irregular weir equation. The discharge co-efficient values for the bank lines have been set at 0.8, and the modular limit set at 0.7 as per the Regional Stormwater Hydraulic Modelling Specifications (Wellington Water Ltd, 2017).

3.3.5.4 Inline Banks

Inline banks are used at connections between river reaches and the reticulated network, or at the majority of stream crossings. Flow across bank lines is calculated using an irregular weir equation. The discharge co-efficient values for the inline banks have been set at 0.8, and the modular limit set at 0.7 as per the Regional Stormwater Hydraulic Modelling Specifications (Wellington Water Ltd, 2017).

3.3.6 Network Addition 2020

Following discussions with WWL on 9th January 2020, it was agreed that additional HCC stormwater network would be added into the model from asset data supplied by WWL as GIS layers. Figure 3-18 provides an overview of the additional network that lies between Stokes Valley and Taita Gorge in relation to the existing 2016 model, and Table 3-6 provides an asset type summary.

Table 3-6: Summary of network additions

Hydraulic Component	Count
Conduits	79
Manholes	55
Sumps	28
Outfalls	4

All additional sumps were modelled as standard grates, inline with single sump as assumed using Google Street View. Ground levels were verified using the HCC 1m LiDAR-derived DEM captured in 2013, which was also used to infer any missing data.

Figure 3-18: Overview of the additional network added to the Stokes Valley model.

3.3.7 Model Updates 2020

Several model updates were also made to the 2016 model based on the discussions with WWL. These changes included:

- Raising inline banks at James Grove inlet
- Adding a missing inline bank at the top end of Manuka Street
- Altering building widths on Oates Street
- Sealing manholes between Evans Street and Bowers Street
- Correcting network at intersection of Glen Road and Hawthorn Crescent

Figure 3-19 provides an overview of the changes made to building outlines along Oates Street.

Figure 3-19. Overview of changes in the vicinity of Evans Street – Bowers Street.

3.4 BOUNDARY CONDITIONS

3.4.1 Rainfall Data

Design rainfall data for the Stokes Valley catchment has been downloaded from the HIRDSv3 website. At the time the model was built, version 3 was the most current data, and it has been agreed with WWL that no update to Version 4 would be undertaken at this stage of works. The design data was extracted for the point with the following NZMG co-ordinates -

X: 2676288

Y: 6001102

The depth-duration values for this location are provide in the table below. A 12-hour nested rainfall profile has been used to temporally distribute the rainfall (Figure 3-21), with the peak occurring at approximately 60% of the storm duration, as per the WWL stormwater panel hydrology recommendations (Cardno, 2016). Rainfall distribution between the upstream hill areas of Stokes Valley and the lower Valley is considered in Section 4 (Validation).

Table 3-7: Design rainfall depths (mm) adopted for the Stokes Valley hydraulic model

ARI (years)	AEP	10m	20m	30m	60m	2h	6h	12h
1.58	0.633	7	9.9	12.2	17.3	24.1	40.7	56.5
2	0.5	7.5	10.7	13.2	18.7	26	43.8	60.9
5	0.2	9.5	13.6	16.7	23.8	33	55.2	76.5
10	0.1	11.2	15.9	19.6	27.9	38.5	64.4	88.9
20	0.05	13	18.5	22.8	32.5	44.8	74.5	102.8
30	0.033	14.2	20.2	24.9	35.5	48.8	81.1	111.7
40	0.025	15.1	21.5	26.5	37.7	51.9	86.1	118.5
50	0.02	15.8	22.5	27.7	39.6	54.4	90.1	124
60	0.017	16.4	23.4	28.8	41.1	56.5	93.6	128.6
80	0.012	17.5	24.9	30.7	43.7	60	99.3	136.3
100	0.01	18.3	26.1	32.1	45.8	62.9	103.9	142.6
100 + 20% CC	-	21.4	30.5	37.5	53.5	73.5	121.4	166.6

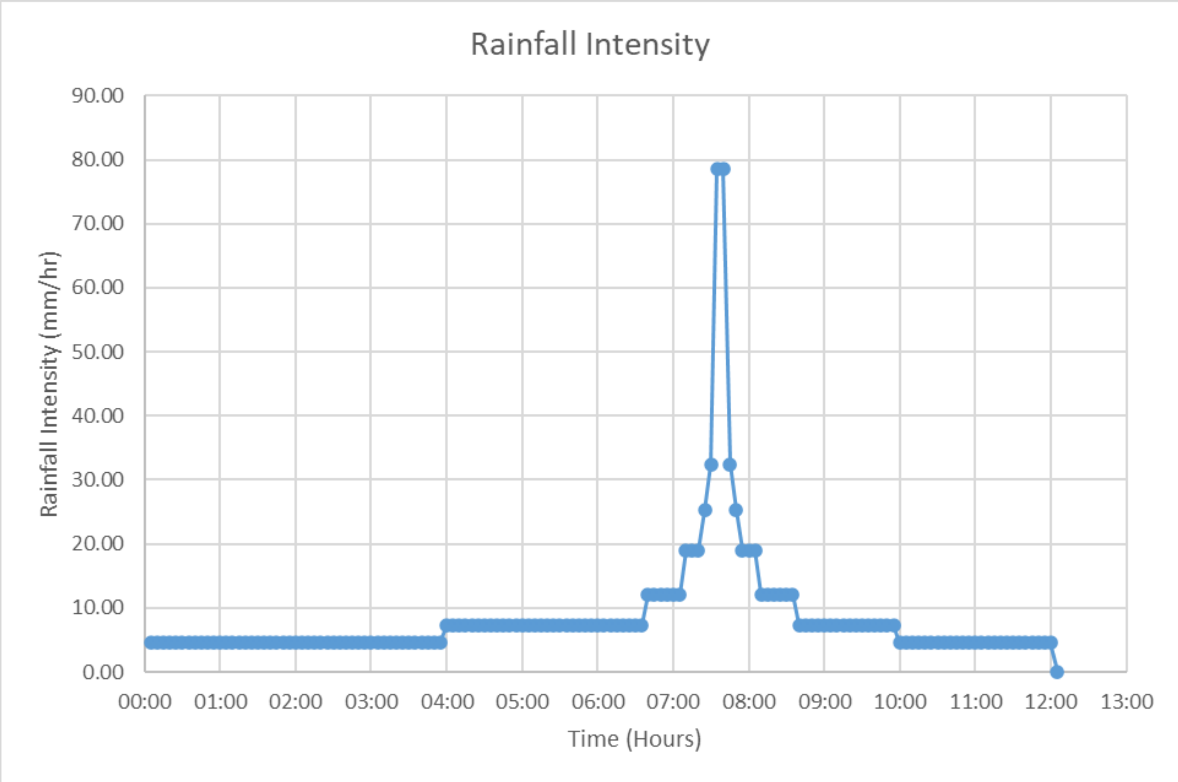


Figure 3-20: Design rainfall intensity profile applied for the 10-year ARI event

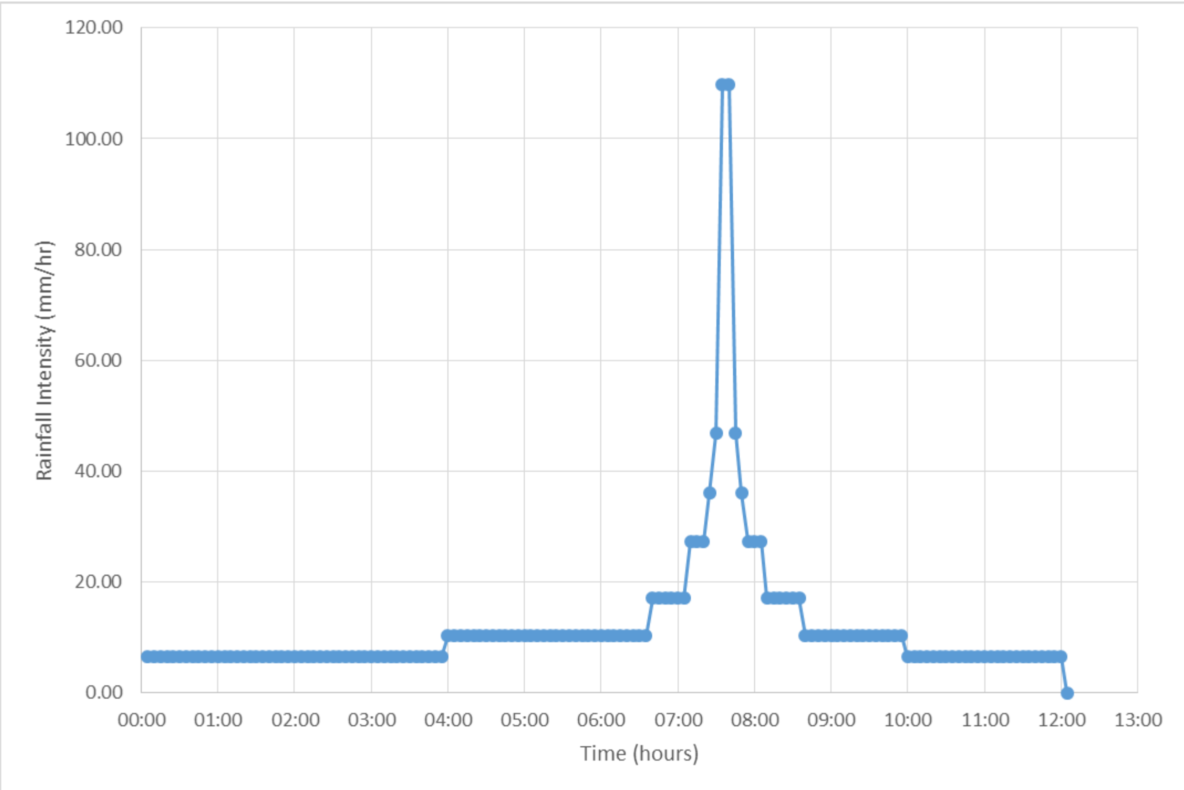


Figure 3-21: Design rainfall intensity profile applied for the 100-year ARI event.

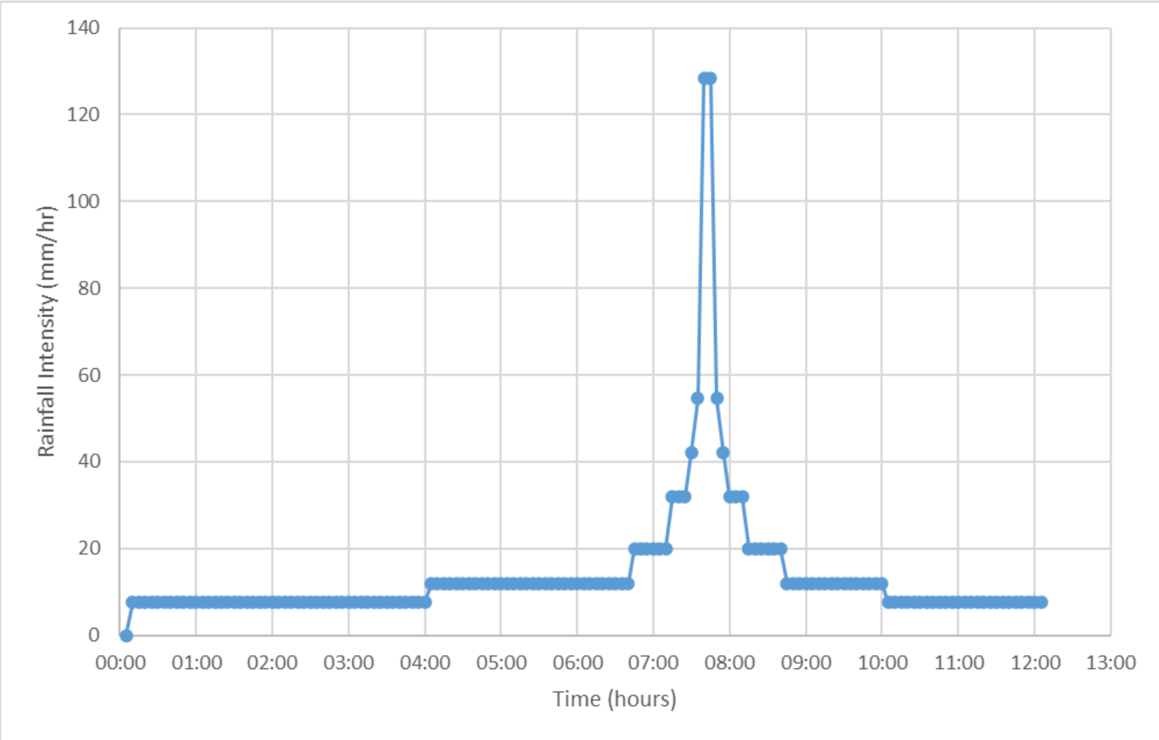


Figure 3-22: Design rainfall intensity profile applied for the 100-year ARI event with 20% rainfall increase for climate change

3.4.2 Hutt River

There is a single outflow boundary for the 1-D channel network at the confluence of Stokes Valley Stream and the Hutt River. At this location, a 1km reach (500m upstream and downstream) of the Hutt River has been modelled using cross sections cut from the ground model.

For all simulations excluding validation events, a hydrograph has been applied to the upstream boundary of this reach with flow increasing from $1m^3s^{-1}$ to $1000m^3s^{-1}$ over a period of 6 hours, after which the flow remains at $1000\ m^3s^{-1}$. This simulated constant 10yr ARI flows in the Hutt River and was agreed with WWL as appropriate for all design simulation. No investigation was undertaken to determine the timing of peak flows in the Hutt River compared with the Stokes Valley Stream. The validation events use the Hutt River flow as recorded at Taita Gorge as inflow into the modelled section of the Hutt River, see Section 4.

3.4.3 2-D Zone

A normal boundary condition has been set for the boundary of the 2-D zone. The 2-D zone extent has been developed so overland flow is unlikely to reach the boundary of the model except towards the outlet.

3.5 MODEL LIMITATIONS AND ASSUMPTIONS

3.5.1 Model Limitations

Computational models are only as accurate as the information input into them, and the data available to verify their accuracy. The primary sources of information for this investigation were available asset data provided by Wellington Water Limited (WWL), LiDAR provided by WWL, recorded stream and rainfall data provided by WWL, scanned as-built plans provided by WWL, and surveyed information collected by MWH, now part of Stantec.

The constraints and limitations of the stormwater flood model are listed below:

- Manhole and pipe levels for a substantial amount of the network have been interpolated from available or surveyed data
- Large parts of the model are based on LiDAR. Where the quality of the LiDAR is suspect or there have been changes made since the collection of LiDAR the model will not represent the real-life structures. Further

information on the quality of LiDAR can be referred to in Ground Model Assessment Summary Report submitted by MWH (now Stantec) in April 2016 to Wellington Water.

- Building floor levels have not been examined and no DEM adjustments have been made to represent appropriate floor levels
- Trash screens have not been included in the model. No trash screens were identified during site visits except Kamahi Park Inlet screen; and the Korau Gr Inlet screen. However, investigation may be required to determine if there are any present and how to therefore represent losses
- Cross sections for several of the modelled open channels have been generated from the LiDAR data
- Where survey data was not available, fences and walls that may constrict flow paths are not represented in the model
- No investigation into channel roughness has been undertaken. It is recommended that if the model is used for detailed design, channel roughnesses are analysed and updated if appropriate

3.5.2 Hydraulic Model Assumptions

The following assumptions have been applied in the development of the hydraulic model:

- The LiDAR generated ground model is an accurate representation of catchment topography
- Manhole lid levels are adequately represented in the ground model
- The interpolation rules applied in UMM are appropriate
- No sediment has been added to the pipes
- Peak outflow from the Stokes Valley catchment coincides with the 10-year constant flow of the Hutt River

3.5.3 Hydrological Model Assumptions

The following assumptions have been applied in the development of the hydrological model:

- The hydrological method specified by Wellington Water Limited for use in the Stokes Valley catchment is appropriate.
- The sub-catchments delineated and parameterised by UMM are appropriate and have been correctly parameterised.

3.6 INITIAL MODEL TESTING

3.6.1 Instability Tests

Initial model testing was carried out using the following events:

- The 100yr ARI nested design storm with a 20% rainfall increase for climate change
- The 100yr ARI nested design storm
- The 10yr ARI nested design storm

At the road crossings time was spent adjusting the 1-D and 2-D components of the couple to provide an accurate and smooth transition between the networks.

3.6.2 Mass Balance Checks

By default, the ICM simulation engine undertakes mass balance checks at every simulation time step. If the cumulative Mass Balance error exceeds 0.01 m^3 at any individual time step the simulation is automatically terminated. This implies that any simulation that is completed is considered to have passed this check. Volume balance information for each simulation is provided in the simulation log file. The volume balances for the two simulations used for initial testing are provided below Table 3-8.

Table 3-8: Volume and Mass Balance Summary for Initial Simulations

Design Storm Event	2D Mass and Volume Balance		
	Total Mass Error (m^3)	Total Volume Balance Error (%)	Total Volume Balance Error (m^3)
100yr ARI + CC	0.55	0.0000	1.46
100yr ARI	0.29	0.0000	1.17
10yr ARI	1.61	0.0003	1.84

3.7 QUALITY ASSURANCE AND QUALITY CHECKS

A full model review has been undertaken at Stantec for the model build and validation (2018 – 2020). An external peer review was undertaken by WSP in 2020.

3.8 RESULTS

The Stokes Valley model has been used to simulate the 100-year ARI with existing and future climate and 10-year ARI with existing climate events. The results are shown in **Error! Reference source not found.** to Figure 3-25 below.

The result for the 100-year ARI simulation shows extensive flooding across Stokes Valley, with overland flow not restricted to roads and non-residential areas. There is significant flooding through, and upstream of, the town centre. In the town centre flood depths are upwards of 1m. Downstream of the town centre flooding is restricted mainly to the Stokes Valley stream and its corridor through the valley, except significant overland flow originating from the eastern branch of the valley containing Glen Rd.

The simulation results for the 10-year ARI event show moderate flooding across Stokes Valley, particularly through and upstream of the town centre. There is also extensive overland flow in the eastern branch of the valley, although it becomes shallower with the smaller event magnitude, as expected.

The flooding originates from both the stormwater network and the open channels, although a preliminary review of the dynamic results indicates that generally the network becomes overwhelmed prior to the open channels spilling, particularly in the eastern and western (containing Delaney Dr) branches. Significant channel overflow occurs on the downstream side of Kamahi Park (flowing to George St) and on the upstream side of the town centre where the channel enters the pipe network at Evans St.

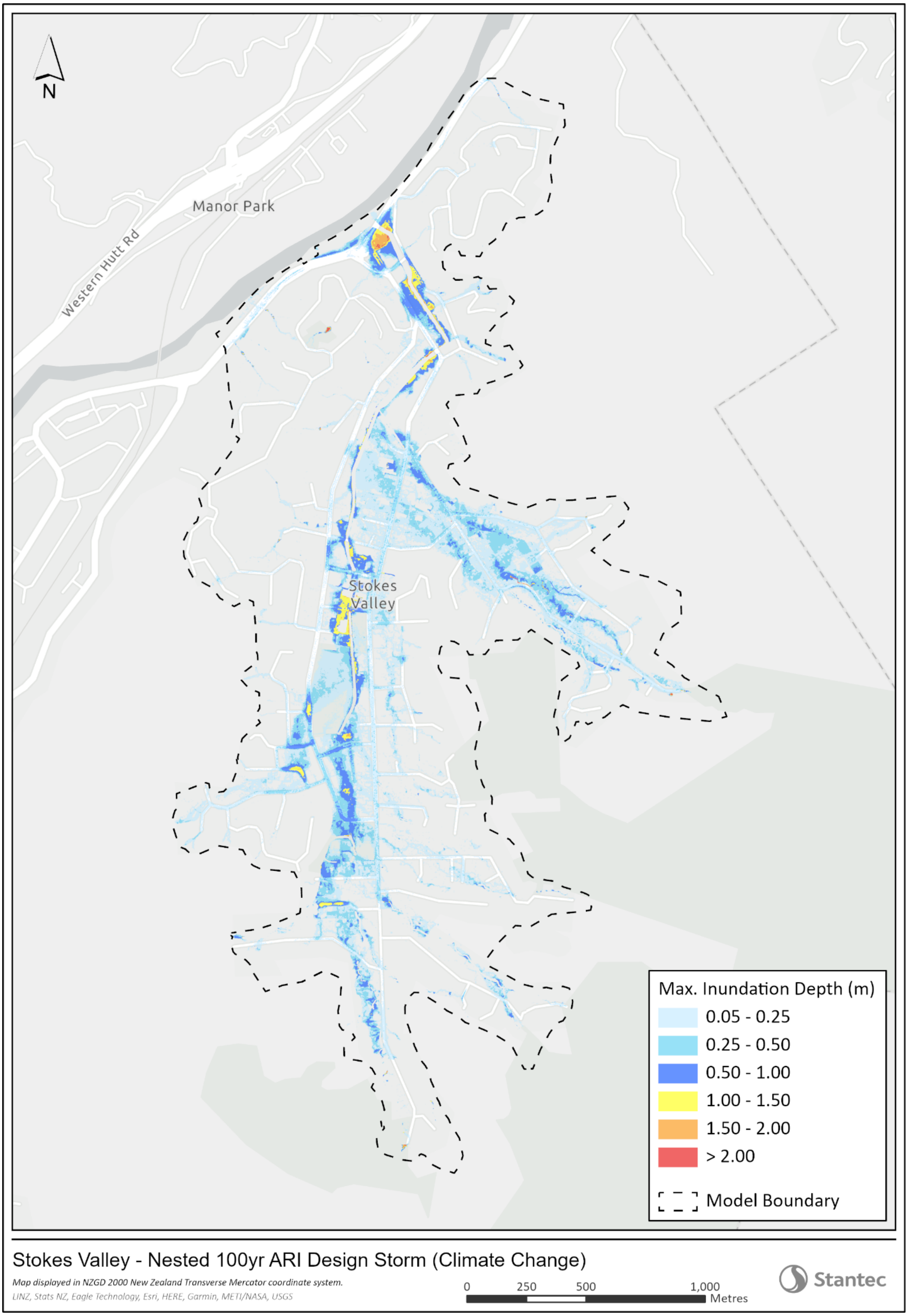


Figure 3-23 100-year ARI (+ climate change) maximum flood depths

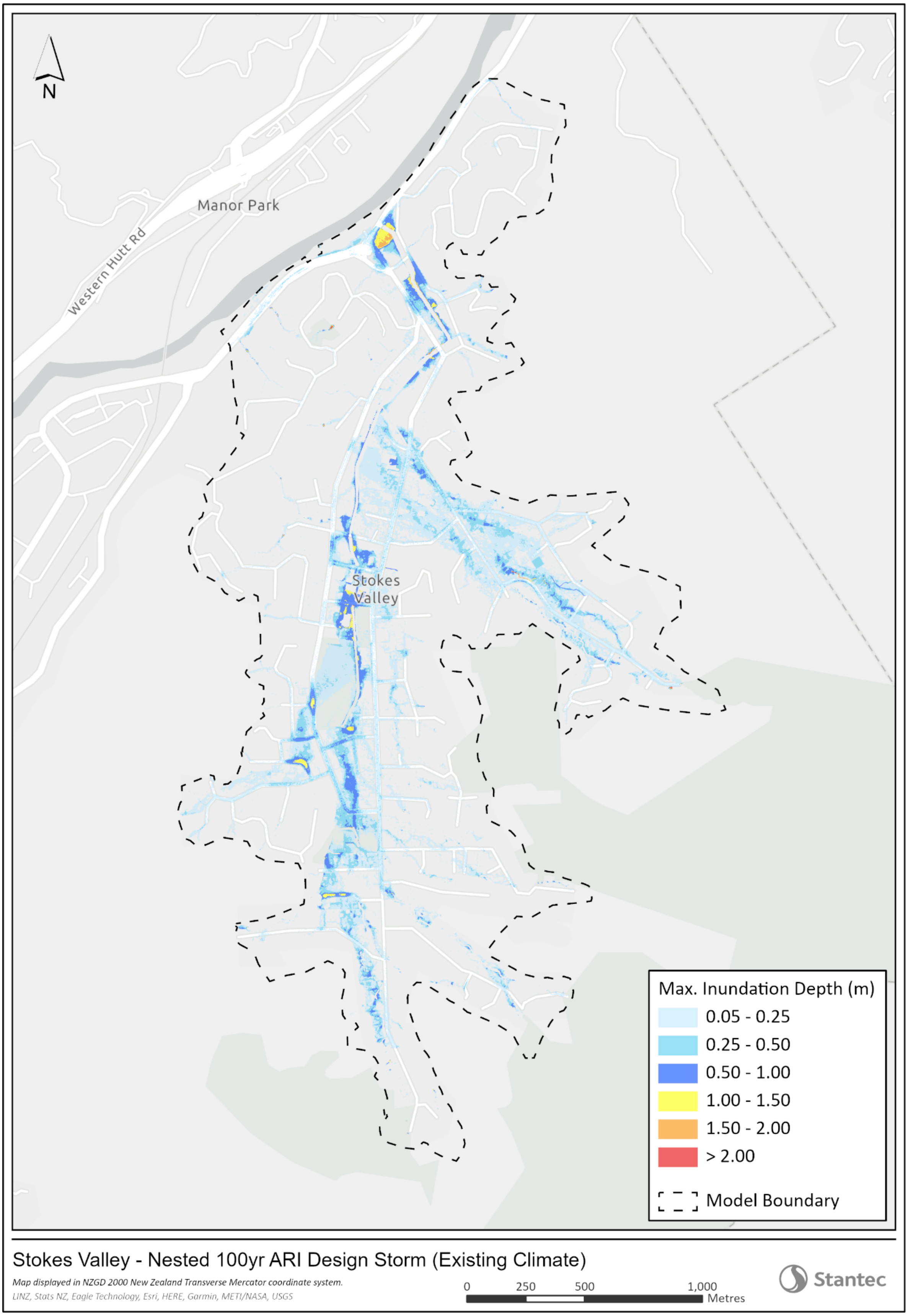


Figure 3-24: 100-year ARI (existing climate) maximum flood depths.

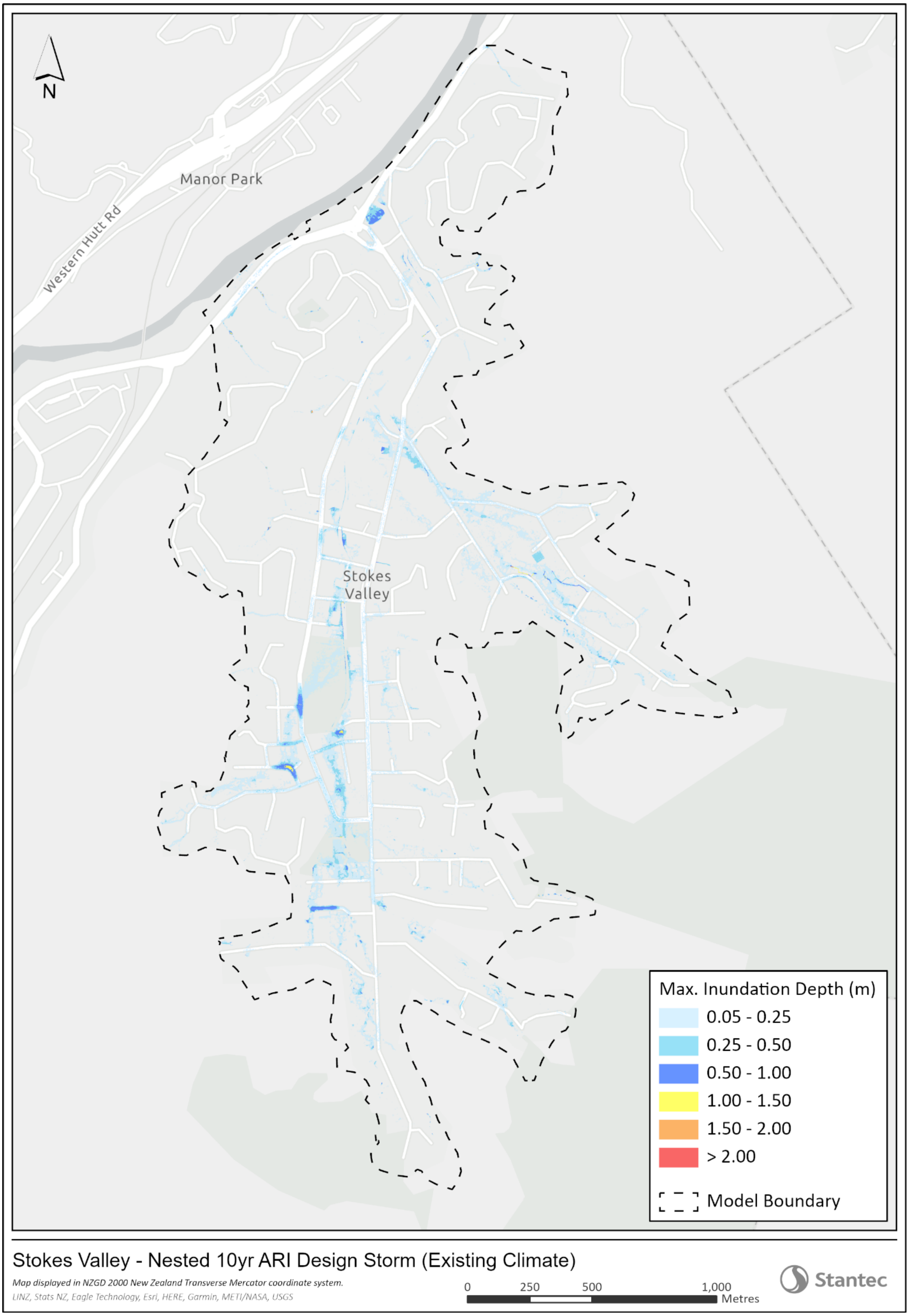


Figure 3-25: 10-year ARI (existing climate) maximum flood depths.

4. MODEL VALIDATION

On 15 November 2016 and on 8th December 2019, heavy rain fell in the Wellington region which caused flooding in parts of the Hutt Valley and high flows within Stokes Valley. After both events (within 1-3 hours of the peak), the site was visited, and photos of flood levels and debris were taken. Recorded rainfall and flow data for the events were input to the Stokes Valley hydraulic model and results compared with the observed indicative maximum flood levels. The purpose of this section is to report on the results of the comparison between observed and modelled flood levels. This was undertaken to provide confidence in the model results.

Both the 2016 and 2019 rainfall events were chosen as they cooccurred close to the construction of the model and were agreed with WWL.

Although the 2004 rainfall event was significant for parts the Lower North island it was not included in the validation, due to data availability, proximity to the model build, and agreement with WWL to focus on the 2016 and 2019 events.

4.1 DECEMBER 2019 EVENT

On 8th December 2019 the Wellington area experienced a significant rainfall event. A 2hr total of 52.5mm of rainfall was recorded between 4.35am and 6.35am, corresponding to an ARI of approximately 50 years. The 24hr total rainfall from 6pm on the 7th of December to 6pm on the 8th of December was 117.3mm which corresponds to an ARI of approximately 5 years. Flooding mostly occurred mid-catchment along George Street, Horoeka Street and around Stokes Valley shops.

4.1.1 Event data

Rainfall radar data was used for the December 2019 event validation to maintain consistency with the November 2016 validation (discussed in Section 4.2), and due to the confidence in total rainfall volume provided by rainfall radar data. Furthermore, as the spatial variability of the storm was unknown, significant assumptions would have to have been made to assign nearby rainfall gauge spatially. We did not have confidence in the appropriateness of this approach. Table 4-1 summarises the rainfall radar total depth and peak rainfall intensity for the Stokes Valley catchment and Figure 4-1 shows the temporal profile of the event including small periods of no rainfall before and after the modelled 6pm to 6pm event.

Hutt River flow, as recorded at the Taita Gorge gauge, was applied as an inflow with a peak discharge of 257m^3s^{-1}

Table 4-1 Summary of rainfall data (24 hours from 3pm December 7th, 2019)

Rainfall Source	Peak Intensity (mm/hour)	Total Depth (mm)
Stokes Valley Rainfall Radar	61.8	117.3

The rainfall radar catchment is shown in Figure 2-9, although as discussed in Section 2.3.1.2 spatial rainfall variation within the catchment is not captured. As validation using rainfall radar data for both November 2016 and December 2019 events is consistent with observed flooding, rain gauge data is not investigated.

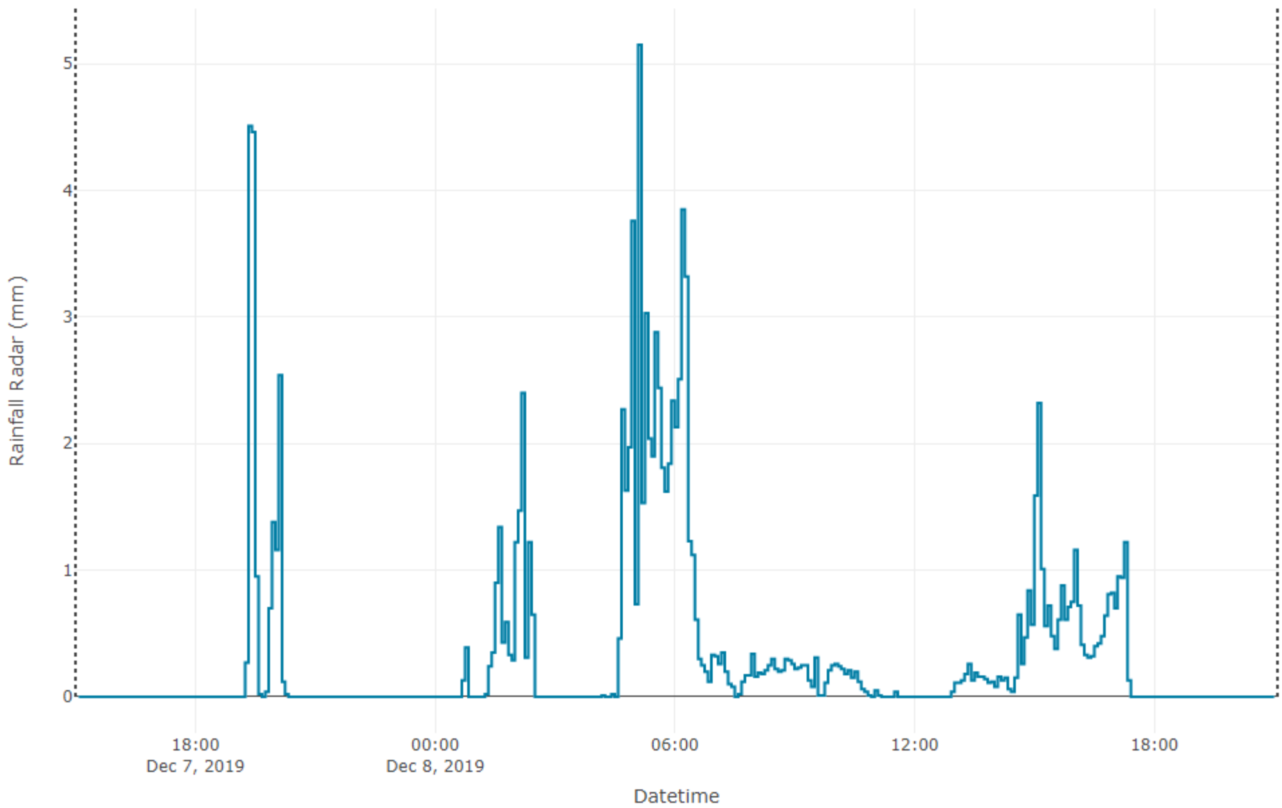


Figure 4-1 Rainfall data for the December 2019 validation event

4.1.2 Results

The model generally correlates well with observed flooding data for the December 2019 event. Flood extents are show to be consistent, although there is some uncertianty around levels due to timing of the photographs (taken a few hours after the peak). There are some minor overestimations of flood extents at some of the Stokes Valley Stream crossings. The following sections discuss areas of interest in Stokes Valley where photos were taken following the 8th December 2019 event to compare observed and modelled flood levels.

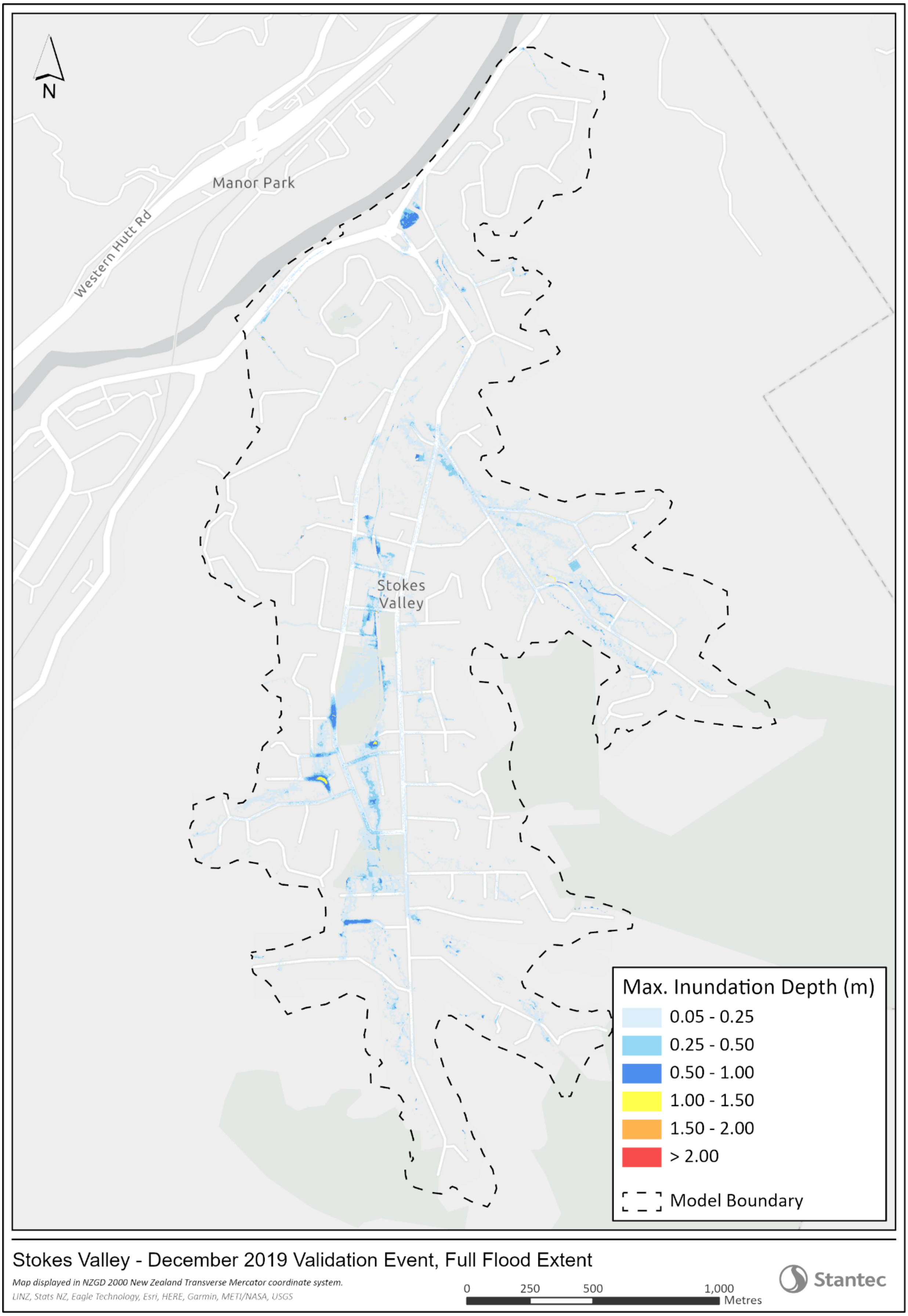


Figure 4-2: Full extent of the December 2019 flood event

4.1.2.1 Stokes Valley Stream – Stokes Valley Road crossing

Although the photo in Figure 4-3 was taken after the peak, debris marks are used to estimate maximum level in the Stokes Valley Stream upstream of the Stokes Valley Road crossing. This indicates that no flooding was observed over the left bank due to its height, but overbank flow may have occurred on the eastern bank. This is consistent with the model simulation results shown in

Figure 4-4.

Figure 4-3: Stokes Valley Stream at Stokes Valley Road Looking Upstream in December 2019.

Figure 4-4: Modelled Flooding of the Stokes Valley Stream at Stokes Valley Road.

4.1.2.2 Glen Road and Richard Grove

Extensive inundation of Glen Road was observed close to the junction with Stokes Valley Road and Richard Grove as shown in Figure 4-5. Properties to the west of Glen Road are indunated in the model simulation, and also appear to be inundated in Figure 4-6. This indicates flooding is well replicated in the model simuilation.

Figure 4-5: Flooding on Glen Road and Richard Grove in December 2019.

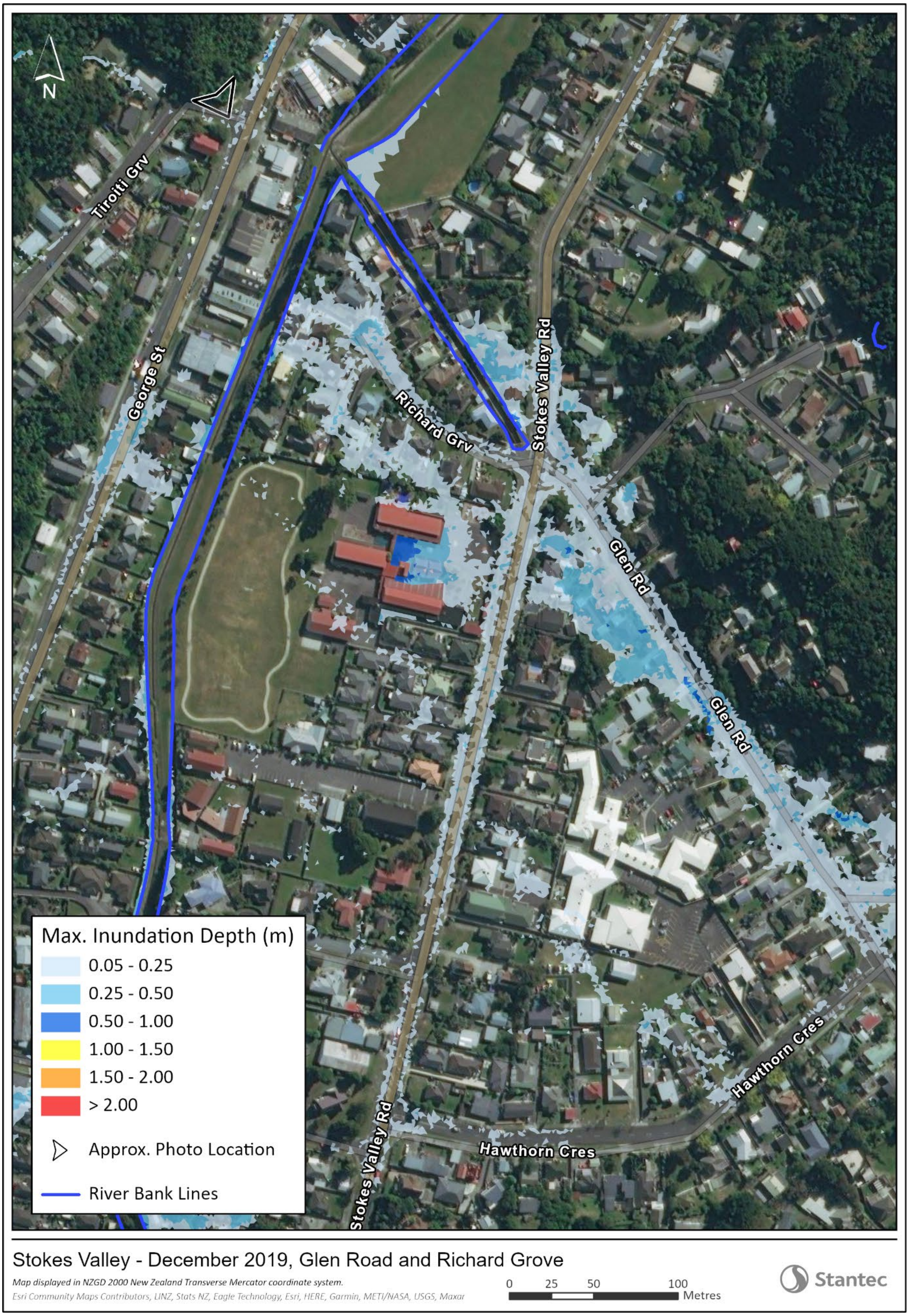


Figure 4-6: Modelled flood extents on Glen Road and Richard Grove in December 2019.

4.1.2.3 Horoeka Street

Inundation of Horoeka Street around number 19 was observed, with significant flow velocity indicated in Figure 4-7. Although depths are hard to estimate, the road reserve appears to be fully inundated, suggesting that some flooding of properties was probable. The model simulation results in

Figure 4-8 appears to correlate reasonably well with the observered flooding.

Figure 4-7: Flooding on Horoeka Street in December 2019.

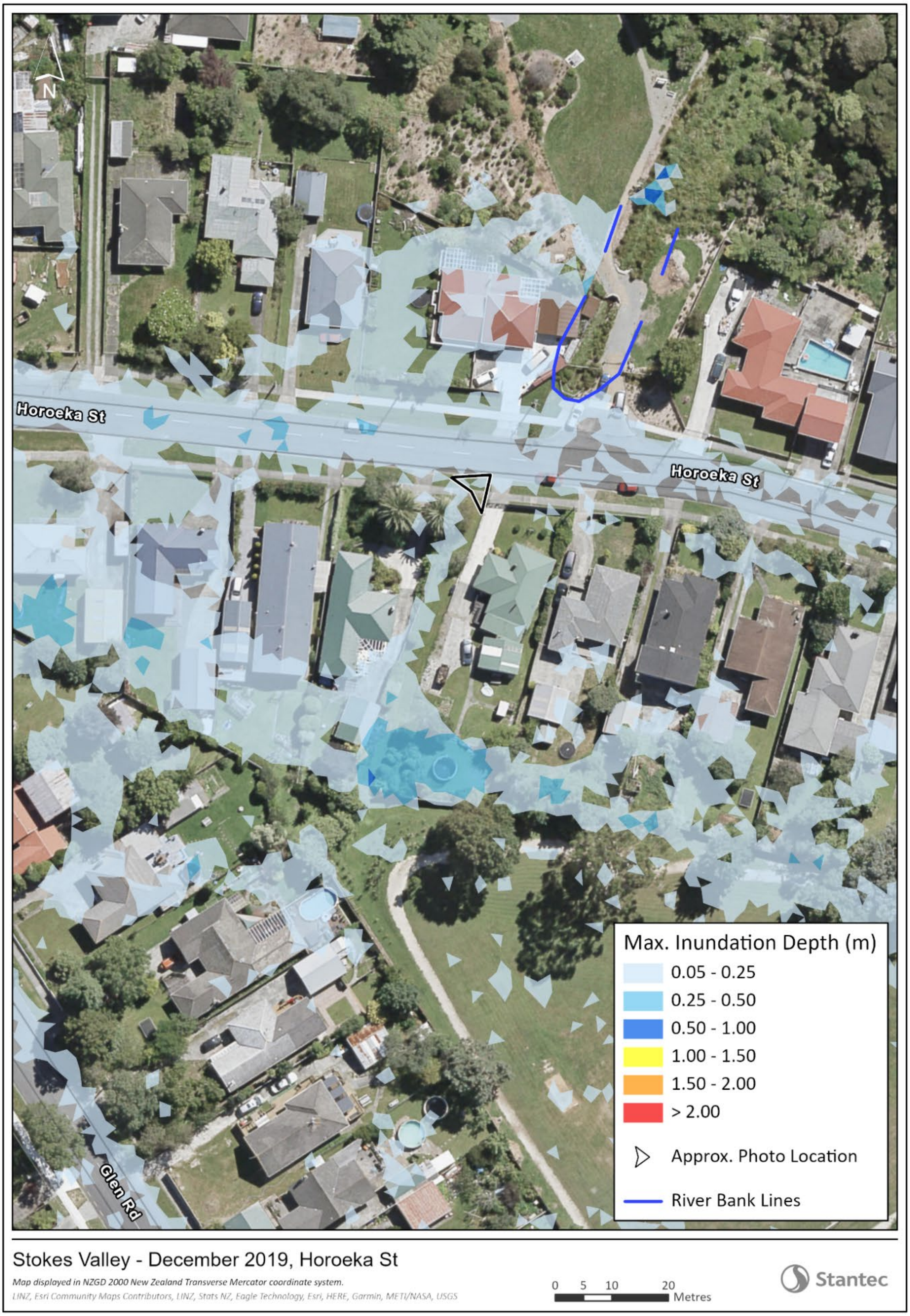


Figure 4-8: Modelled flood extent on Horoeka Street in December 2019.

4.1.2.4 Stokes Valley Stream – Rawhiti Street crossing

Flooding of property was observed at 15 Rawhiti Street downstream of the Evans Street culvert on the Stokes Valley Stream. This is shown in Figure 4-9 where possible overtopping of the Rawhiti Street crossing is also indicated. Flooding of the garage at 15 Rawhiti Street is replicated in the model simulation results, as shown in

Figure 4-10. Significant overbank flow on the right bank is also indicated upstream of the stream crossing, however this cannot be confirmed. Some overtopping both left and right of the stream crossing may have occurred, but it is likely to have been slightly overestimated in the model simulation. Figure 4-11 shows flow in the Stokes Valley Stream downstream of the crossing, with maximum level estimated using debris marks. There may be some slight overestimation of flood extents in the model simulation at this location; however, this could be in part from the fences not being modelled.

Figure 4-9: Left) Flooded garage on Rawhiti Street in December 2019. Right) Possible overtopping of Rawhiti Street bridge in December 2019.

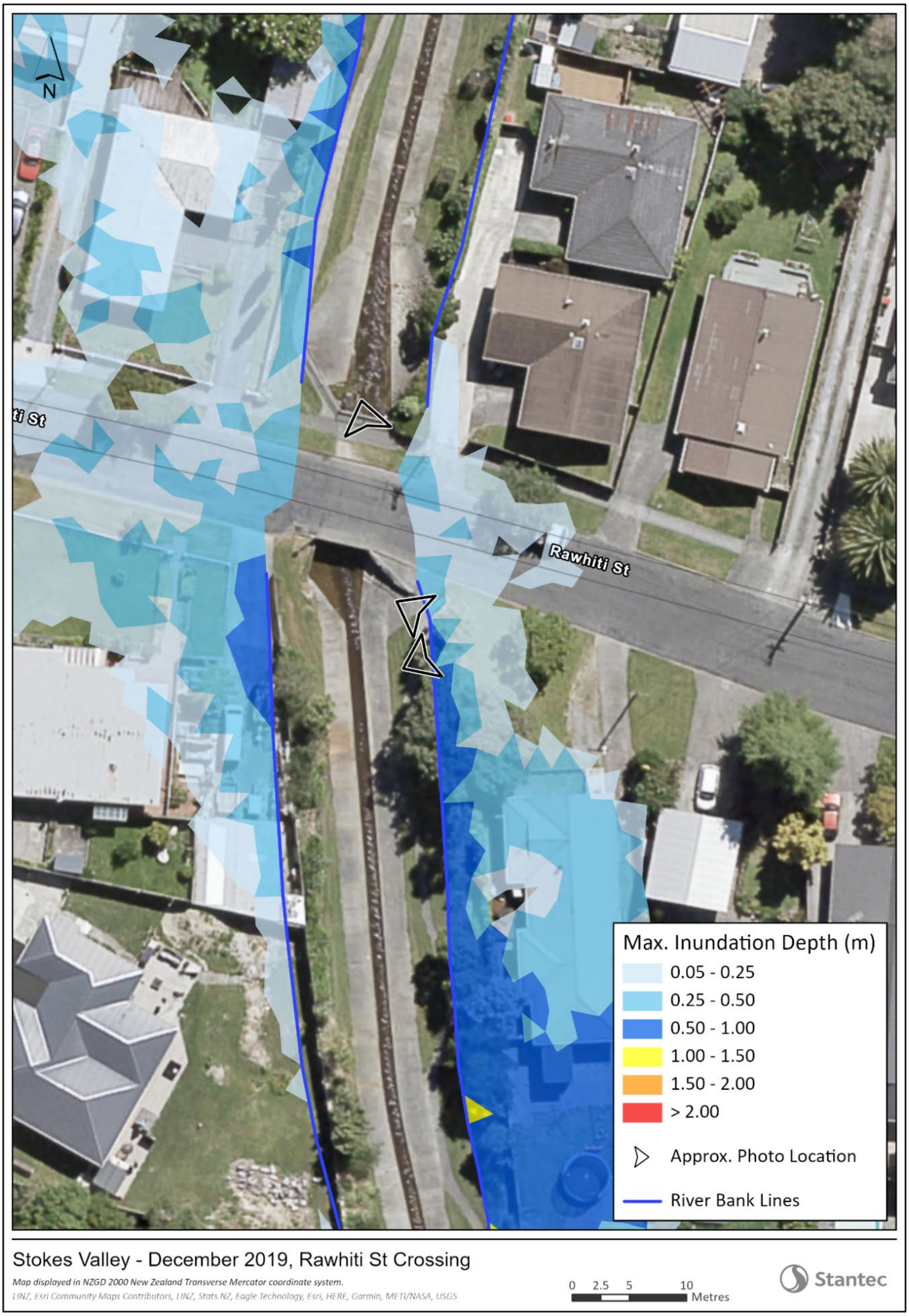


Figure 4-10: Modelled flood extent on Rawhiti Street in December 2019.

Figure 4-11 Stokes Valley Stream downstream of the Rawhiti Street Crossing

4.1.2.5 Stokes Valley Stream – Evans Street

Error! Reference source not found. shows Stokes Valley Stream upstream of the Evans Street culvert inlet and allows for estimation of maximum level based on disturbance of vegetation. This correlates well with the model results shown in

Figure 4-13 as flow is contained within the right bank. Overbank flow is modelled over the true left bank, however the majority of the flooding modelled at Evans Street results from system capacity issues.

Figure 4-12 Stokes Valley Stream look upstream from Evans Street in December 2019.

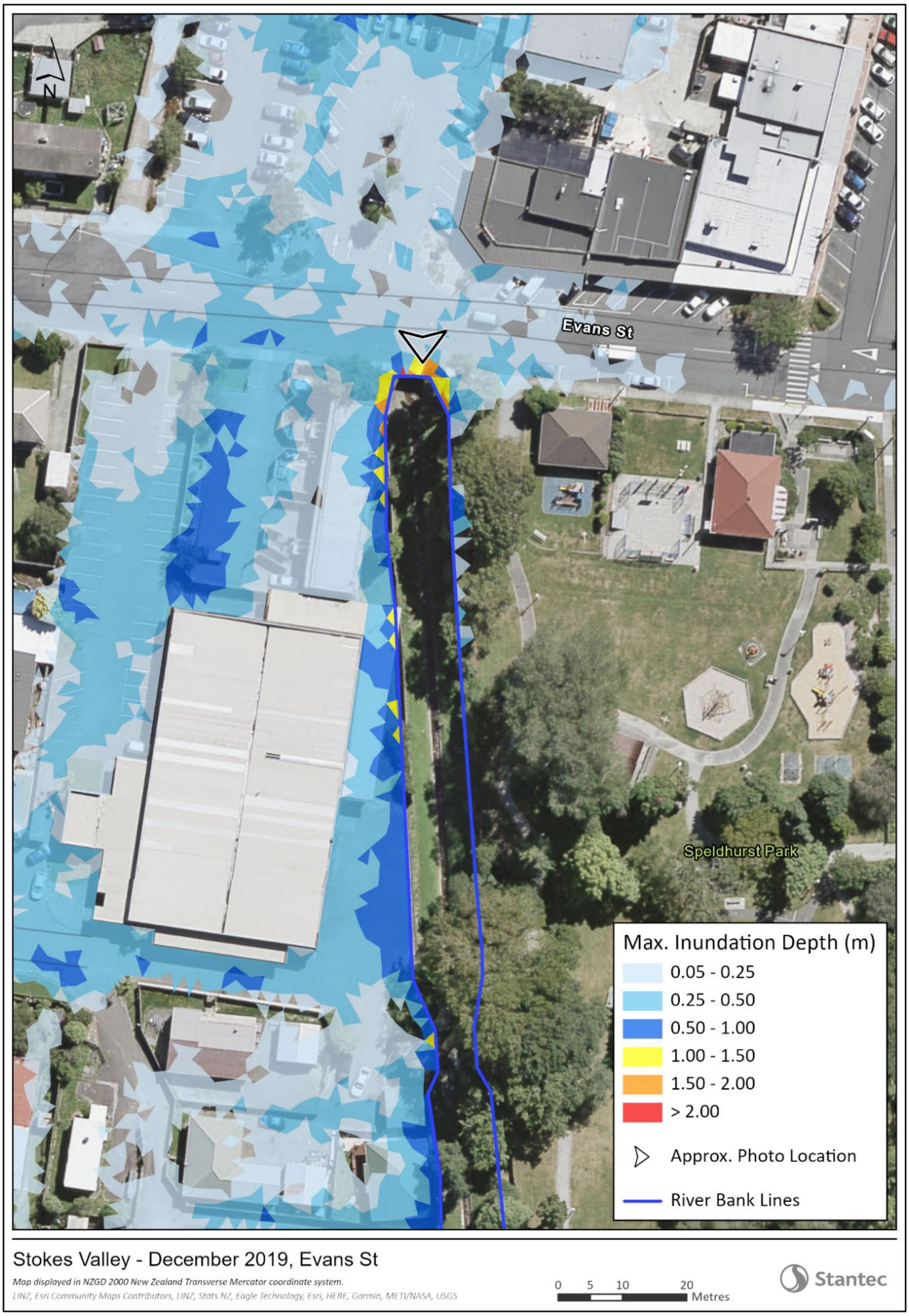


Figure 4-13: Modelled Flooding at the Stokes Valley Stream near Evans Street in December 2019.

4.1.2.6 George Street and Speedy Street

Significant ponding of stormwater was observed at the George Street and Speedy Street junction adjacent to Delaney Park. This is shown in Figure 4-14 and Figure 4-15 with some debris lines allowing for estimation of maximum level. Modelled and observed flooding does not extend onto Delaney Park at the Speedy Street-George Street junction, and the flow path across Delaney Park is not captured in Figure 4-15. Maximum flood extents are consistent between model results and observed flooding, indicating that the model validates well in this location.

Figure 4-14: Flooding on the corner of George and Speedy Streets in December 2019.

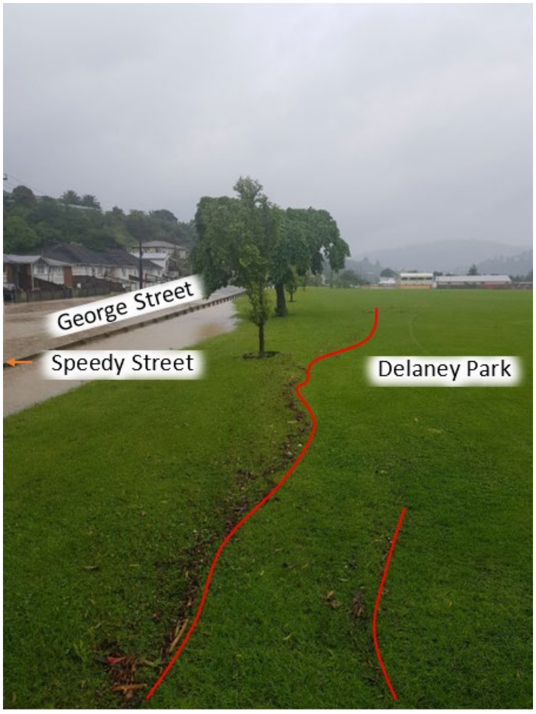


Figure 4-15: Flooding on George Street and Delaney Park in December 2019.

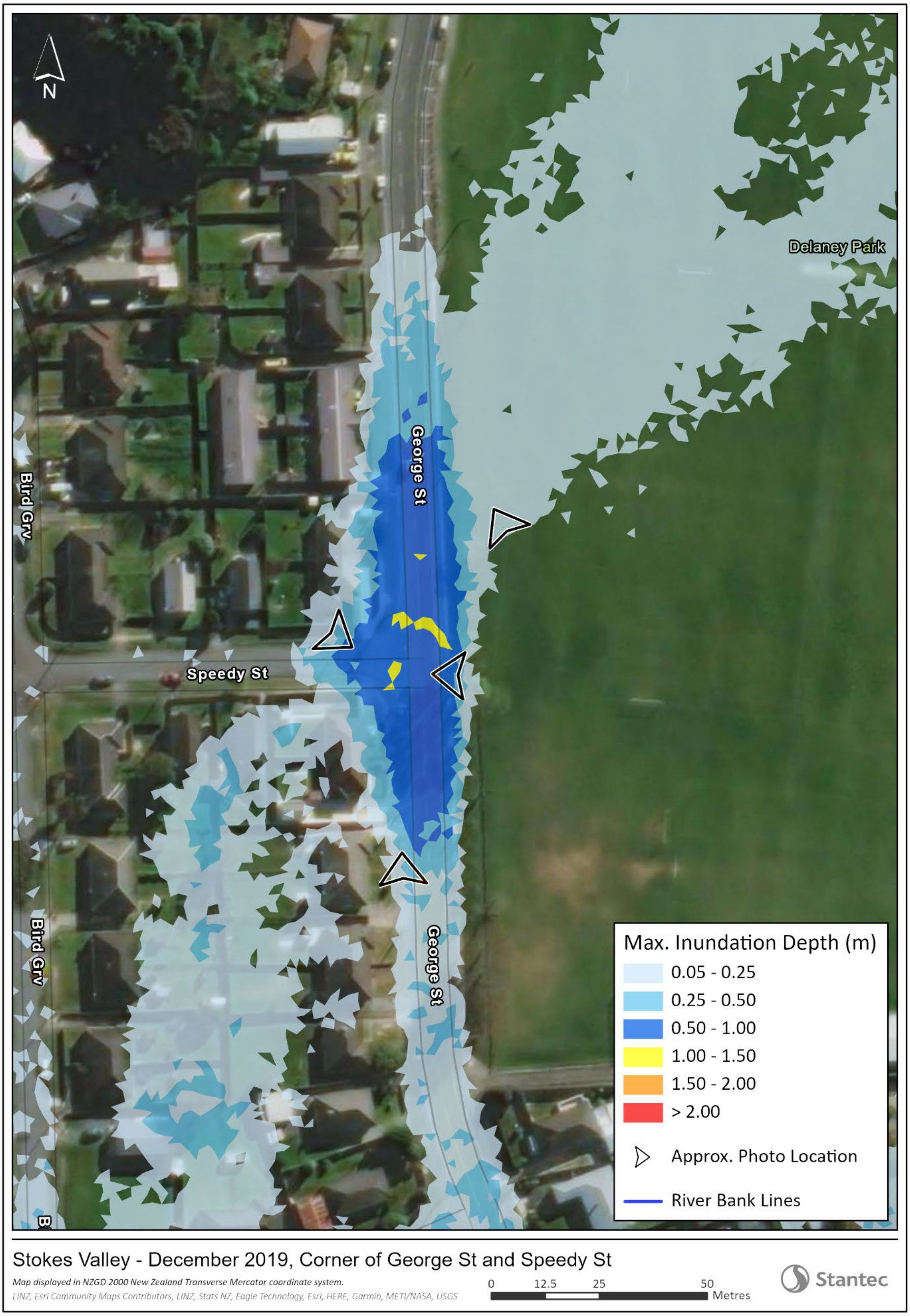


Figure 4-16: Modelled flooding on George Street, Speedy Street, and Delaney Park in December 2019.

4.1.2.7 George Street near Stokes Valley Road

Observed gravel and debris on George Street close to the junction with Stokes Valley Road indicates that flooding was observed across the full width of the road with sufficient velocity to transport gravel (

Figure 4-17) This is consistent with model simulation results that show flooding through 400 and 390 George Street as a result of overtopping of the Kamahi Park inlet structure

Figure 4-18. This inlet structure carries the Stokes Valley Stream under George Street to Delaney Park. Although flooding of property on George Street is not shown in

Figure 4-18, the volume of debris indicates flows that make inundation of property probable due to local topography.

Figure 4-17: Flood debris near 390 - 400 George Street in December 2019.

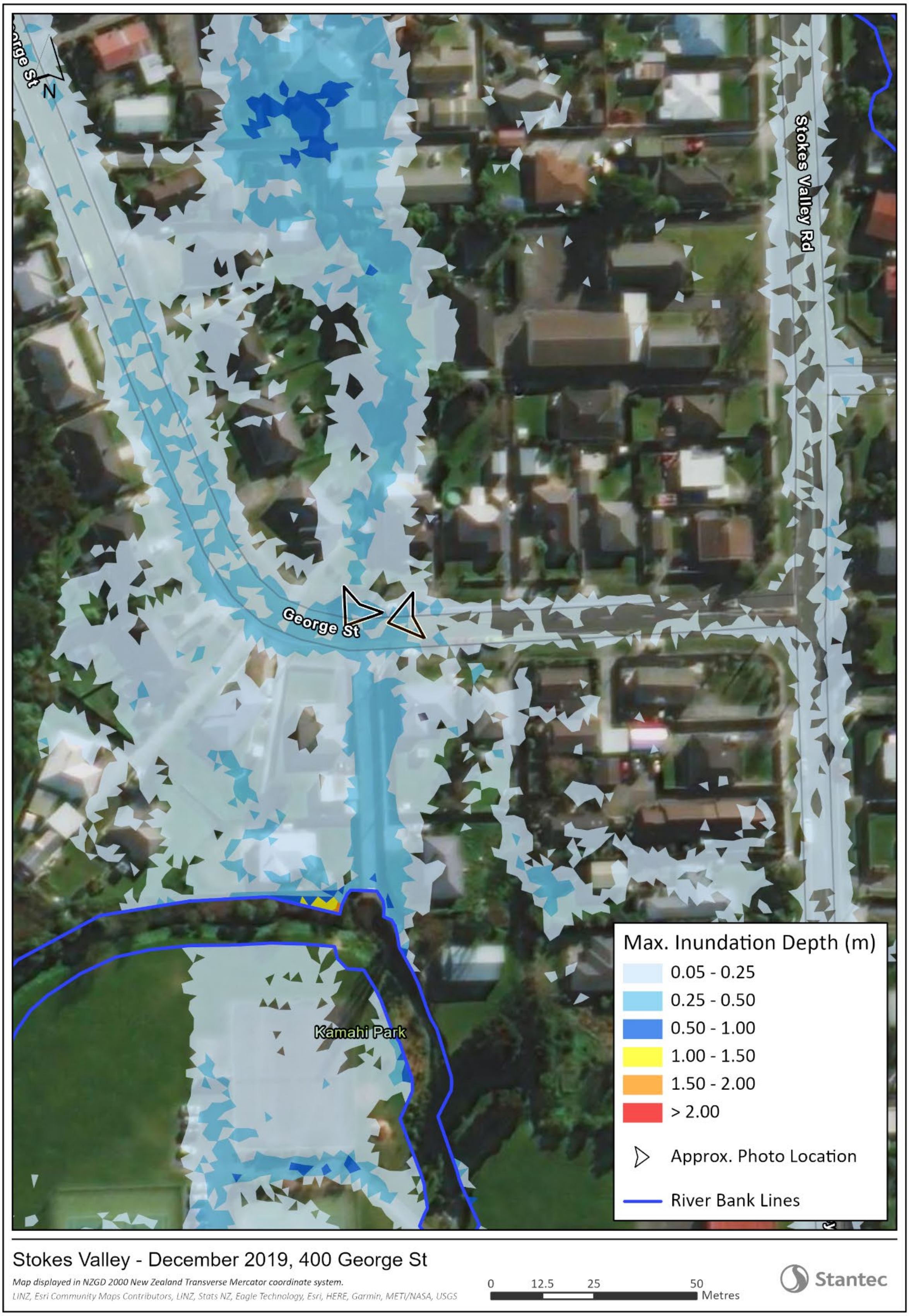


Figure 4-18: Modelled Flood Extent near 390 - 400 George Street in December 2019. Kamahi Park is to the south.

4.1.3 Summary

The model validates well against the available data for observed flooding during the December 2019 storm. This was a short event, with 52.5mm of rainfall recorded over a two-hour period as a catchment wide average. This rainfall intensity would have varied across within the catchment. As shown in the figures above, extents are generally consistent between observed and modelled flooding. There are some areas where minor overestimation of flooding has likely occurred in the model results.

4.2 NOVEMBER 2016 EVENT

On 15 November 2016, the Stokes Valley catchment experienced a rainfall event in the order of a 5- to 10-year ARI. Both rain gauge and rain radar were obtained and are discussed in the following sections. The 16-hour rainfall event began on 15 November at 12:00am and ended at 4:00pm, however for the purpose of this study rainfall was only applied for the most intense 12 hours of rainfall beginning at 12.00am. Rainfall occurred on either side of the 12-hour modelled rainfall, across a three-day period from 14th to 16th November; however, following analysis, it was concluded it was not likely to contribute significantly to the observed flooding.

4.2.1 Event Data

Recoded rainfall data is available from the Pinehaven Stream and Mangaroa River rain gauges, as well as a catchment averaged rainfall radar profile (Figure 2-9). The Hutt River flow as recorded at the Taita Gorge gauge was applied as an inflow, with a peak discharge of $1057m^3s^{-1}$.Table 4-2 provides a summary of total rainfall depths and peak intensities. The rain gauge and rainfall radar are outlined below, however, the model simulations used in the November 2016 validation are based on rainfall radar data only due to a greater confidence in total rainfall volume.

Table 4-2 Summary of rainfall data (16 hours from 12am November 15th, 2016)

Rainfall Source	Peak Intensity (mm/hour)	Total Depth (mm)
Pinehaven Stream at Pinehaven Reservoir	33.6	74.3
Mangaroa River at Tasman Vaccine	40.8	88.9
Stokes Valley Rainfall Radar	39.5	76.9

4.2.1.1 Rainfall Data (not used for final validation simulation)

The Mangaroa River at Tasman Vaccine gauge recorded a peak intensity of 40.8mm/hour for a 5-minute duration. The total depth over the 12 hours modelled was 88.9mm. This was similar to the 10-year HIRDS depth for a 12-hour duration of 87.8mm. The Pinehaven Stream at Pinehaven Reservoir gauge recorded a peak intensity of 33.6mm/hour for a 5-minute duration. Total depth over the 12 hours modelled was 74.3mm. This was similar to the 5-year HIRDS depth for a 12-hour duration of 76.6mm. There were two peaks in the hyetograph, including one between 5:30am and 6:30am and a larger one between 10:35am and 11:10am.

Rain zones were used to represent a weighting of the two rain gauges which are located to the northeast and east of Stokes Valley. The topography was used to define a boundary between the two rain zones, weighted more heavily toward the Pinehaven Gauge (Figure 1-1). This weighting was applied because the storm event came from the northwest and travelled toward the east. Although this modified rainfall distribution improved confidence in the total rainfall volume applied to the catchment, rainfall radar data further increased this confidence. Therefore, rainfall gauge data has not been used for the validation.

4.2.1.2 Rainfall Radar Data

Rainfall radar data covering Stokes Valley is available for the 2016 event; and is provided by WWL via the Mott MacDonald Moata interface as a catchment-averaged rainfall profile. The received data was a single rainfall profile across a pre-drawn catchment shown in Figure 2-9. This data was produced using a 500m-by-500m grid averaged across the Stokes Valley catchment, and should better capture catchment scale rainfall distribution when compared to rainfall gauge data, particularly considering none of the nearby gauges are within the Stokes Valley catchment. A catchment average profile fully cannot capture spatial variation of rainfall within the catchment, but confidence in the total volume of rainfall applied can be assumed.

The rainfall starting at November 15th 12:00am was chosen for the validation event because it was the largest rainfall event within a 24 hour period in the data.

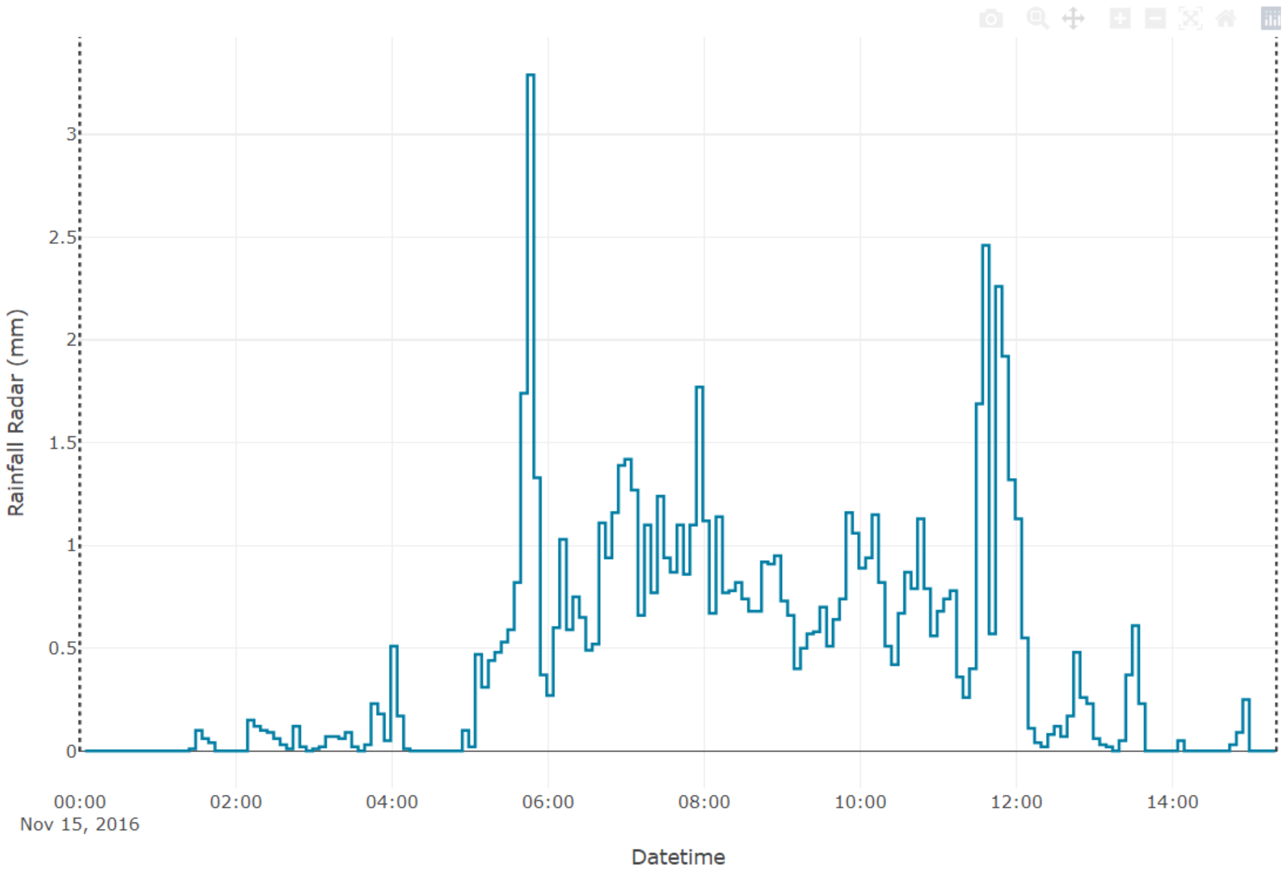


Figure 4-19: Rainfall data for the November 2016 validation event

4.2.2 Results

Modelled flood levels for the 15 November 2016 event compare reasonably well against observed levels in the lower reaches of the Stokes Valley Stream. There may be some overestimation of flooding in the middle reaches of the catchment; however, this is difficult to confirm, as observed levels cannot be confirmed from photographs. It could also be a result of spatial variation in rainfall within the catchment that is not captured using the catchment averaged rainfall radar profile. Observed flooding data was only available around the channel network, particularly the main channel for the Stokes Valley Stream. This has limited the observed evidence for validation of the model using this event. The following sections discuss areas of interest in Stokes Valley where photos were taken following the 15 November 2016 event to compare observed and modelled flood levels.

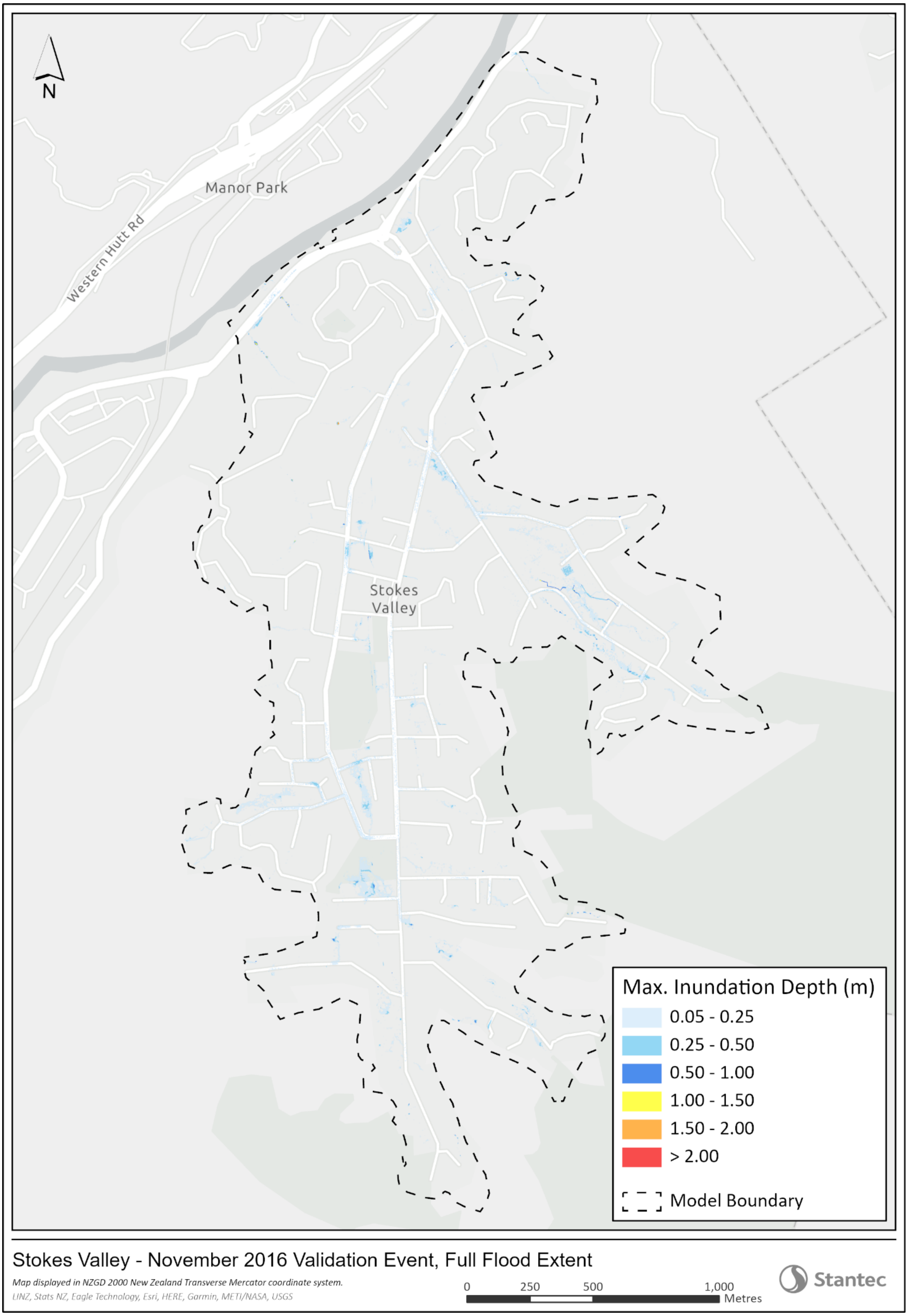


Figure 4-20: Full extent of the November 2016 flood event

4.2.2.1 Stokes Valley Stream – Eastern Hutt Road

In the lower reaches of the catchment, Stokes Valley Stream flows were well distinguished in the photos taken following the 15 November event. Figure 4-20 to Figure 4-22 show that the Stokes Valley Stream did not overtop the banks in the event. This is also apparent with the model simulation where all flow is contained within the 1D river reach (

Figure 4-23 to Figure 4-25). Cross-section data indicates a peak flood depth of approximately 1.93m, similar to what has been observed in the photos. The banks are approximately 4m to 5m above the invert of the stream at this downstream location and so flows are well contained in minor to moderate events like the 15 November 2016 event. The model simulation compares well against observed flooding at this location.


Figure 4-21: Stokes Valley Stream looking upstream from Eastern Hutt Road (left) and looking downstream between Eastern Hutt Road and the Hutt River (right)

Figure 4-22: Stokes Valley Stream looking downstream from Eastern Hutt Road.

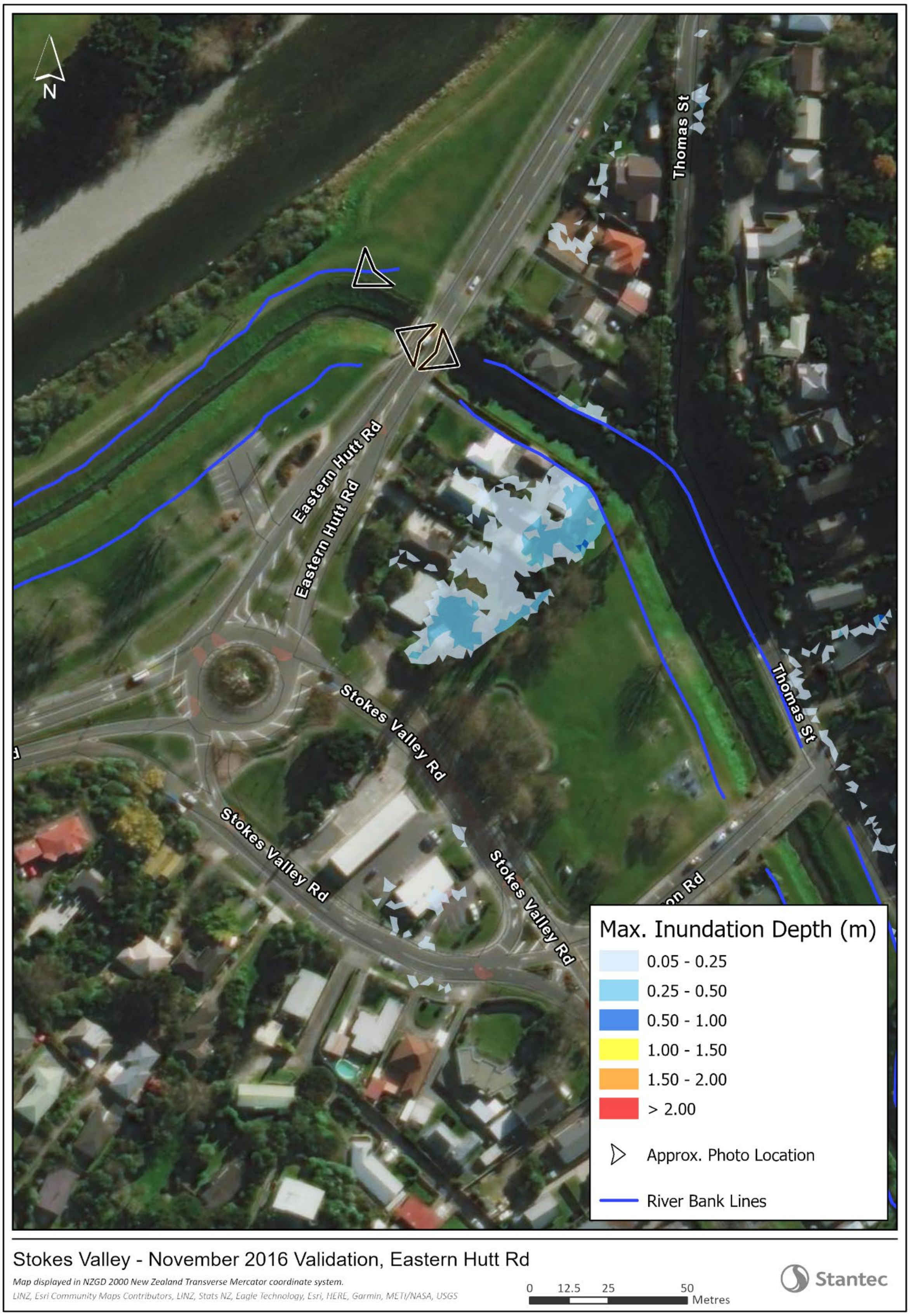


Figure 4-23: Boundary of the modelled 1D channel (blue line) and maximum 2D depths in lower reach of the Stokes Valley Stream (15 November 2016 event).

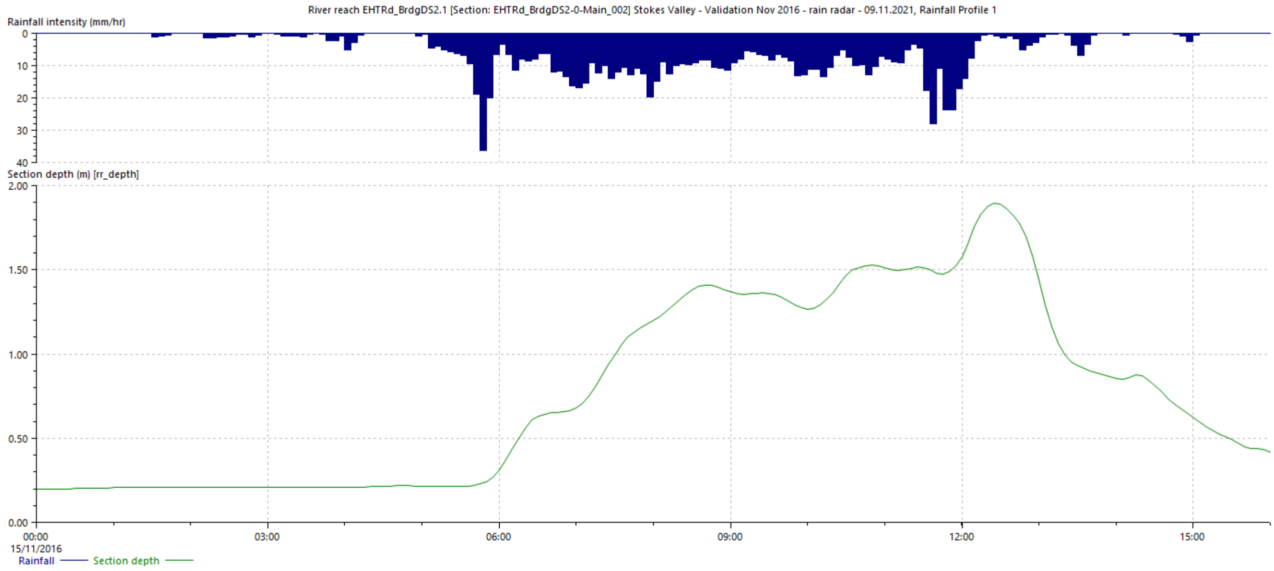


Figure 4-24: Flood depths from the model simulation through a cross-section taken from the lower reaches of the Stokes Valley Stream near Eastern Hutt Road.

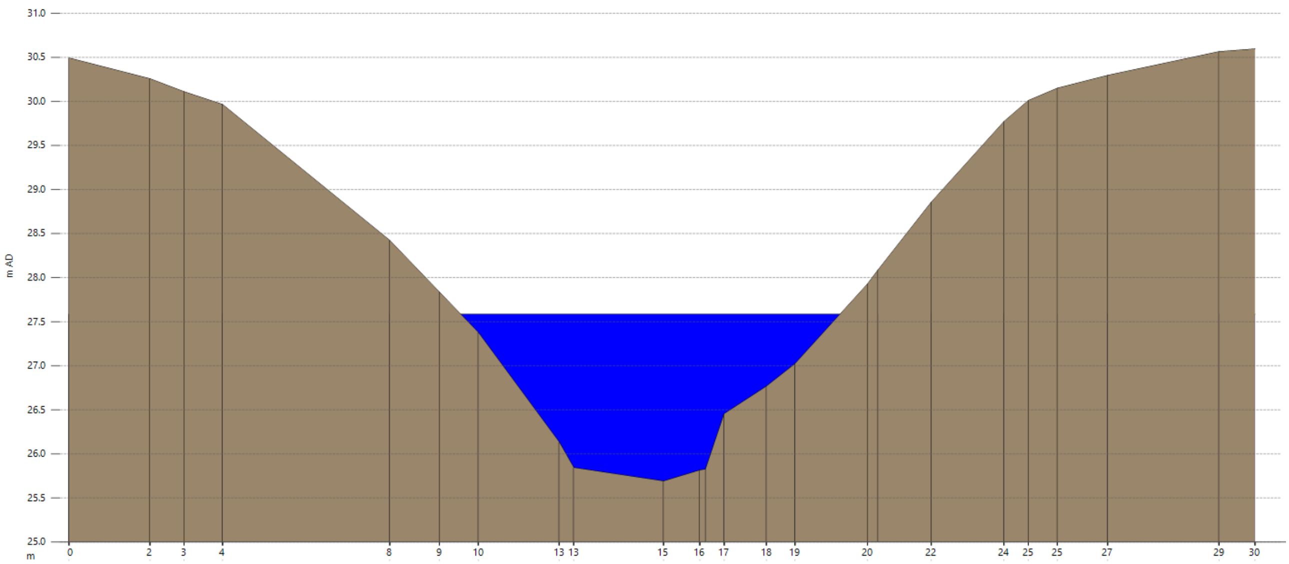


Figure 4-25: Flood levels from the model simulation through a cross-section taken from the lower reaches of the Stokes Valley Stream near Eastern Hutt Road.

4.2.2.2 Stokes Valley Stream – Near 67 Stokes Valley Road (Upstream)

Figure 4-27 and Figure 4-28 show the depth of flow through a cross-section directly upstream of the weir structure near 67 Stokes Valley Road. During the modelled event, the stream reaches a peak depth of 1.6m. This appears to coincide with the observed flood levels shown in Figure 4-26. The flooding that occurs outside of the western bank line Figure 4-27 in the model simulation appears to be localised and may instead be a minor issue with modelled bank levels along this area. The bank line height along this section of channel ranges between 2.5m to 3m, therefore peak flood depths of 1.6m are unlikely to have breached the channel's banks at this location unless there is an anomaly in the bank height.


Figure 4-26: Stokes Valley Stream looking upstream from near 67 Stokes Valley Road.

Figure 4-27: Boundary of the modelled 1D channel (blue line) and maximum 2D depths near 67 Stokes Valley Road (15 November 2016 event).

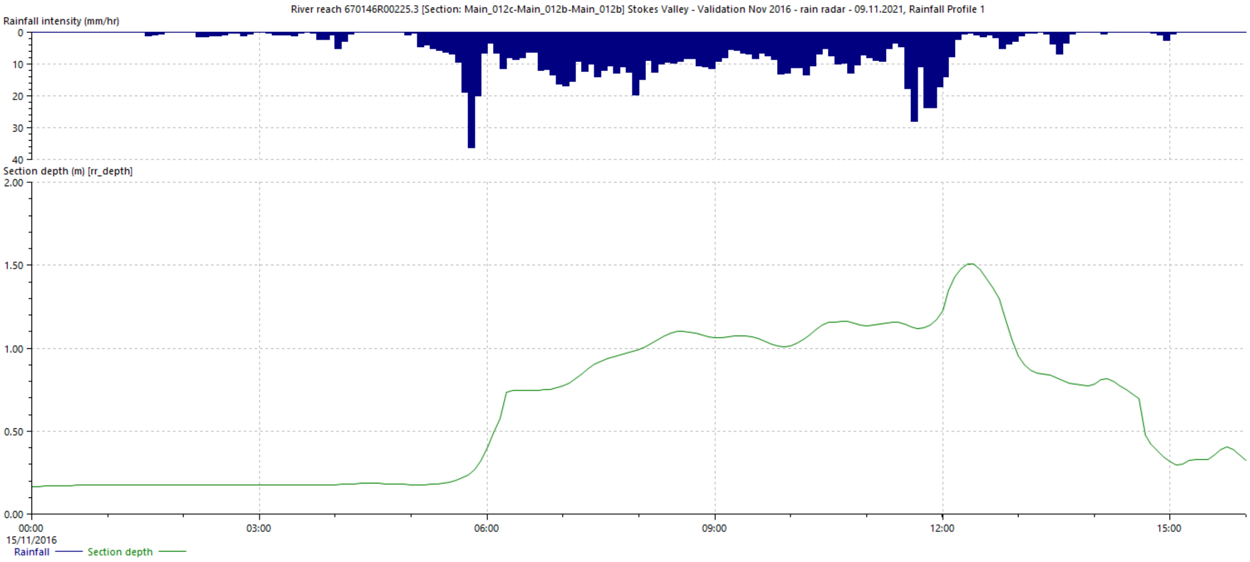


Figure 4-28: Flood depths from the model simulation through a cross-section taken upstream of the weir structure near 67 Stokes Valley Road.

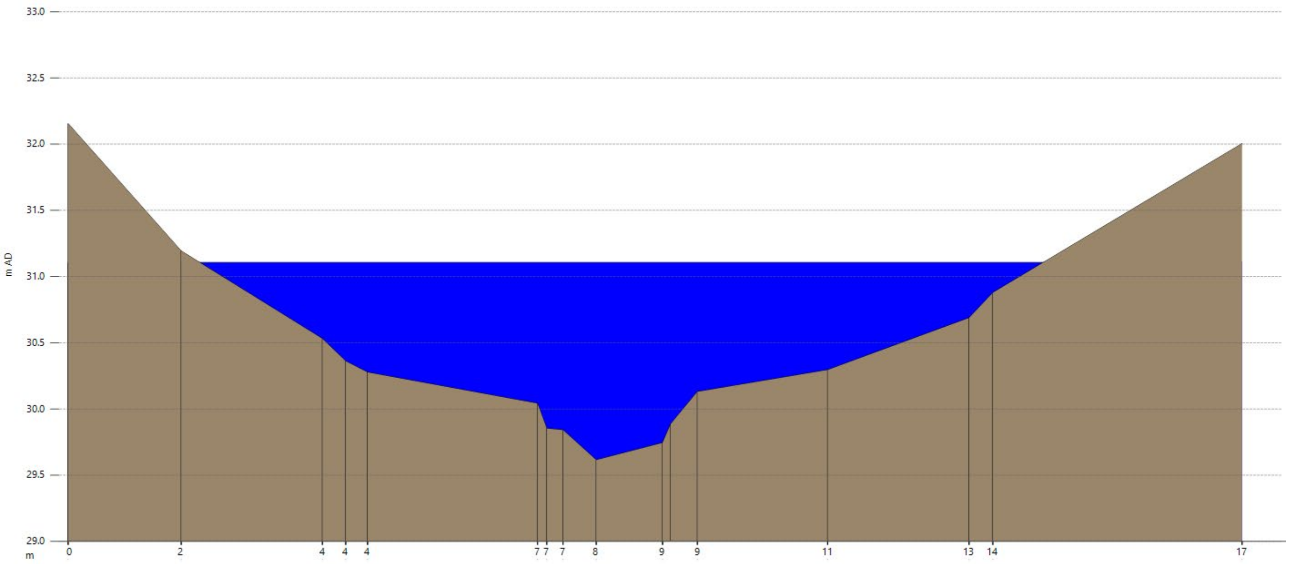


Figure 4-29: Flood levels from the model simulation through a cross-section taken directly upstream of the weir structure near 67 Stokes Valley Road.

4.2.2.3 Stokes Valley Stream – Near 67 Stokes Valley Road (Downstream)

Downstream of Stokes Valley Road, the model simulation shows an approximate depth of flooding outside of the southern bank line of 50mm to 200mm (Figure 4-27) from the Stokes Valley Stream. This is also relatively localised and minimal and may again be accounted for through issues with the bank line. Modelled levels appear to agree relatively well with observed/photographed flood levels shown in Figure 4-30. Flooding has for the most part not extended past the ~2.5m to 3m bank lines. Figure 4-31 and Figure 4-32 show the depth of flow through a cross-section directly downstream of Stokes Valley Road to be 1.7m. This appears similar to that observed in the flood photos.

Figure 4-30: Stokes Valley Stream looking downstream from near 67 Stokes Valley Road following the 15 November 2016 rainfall event.

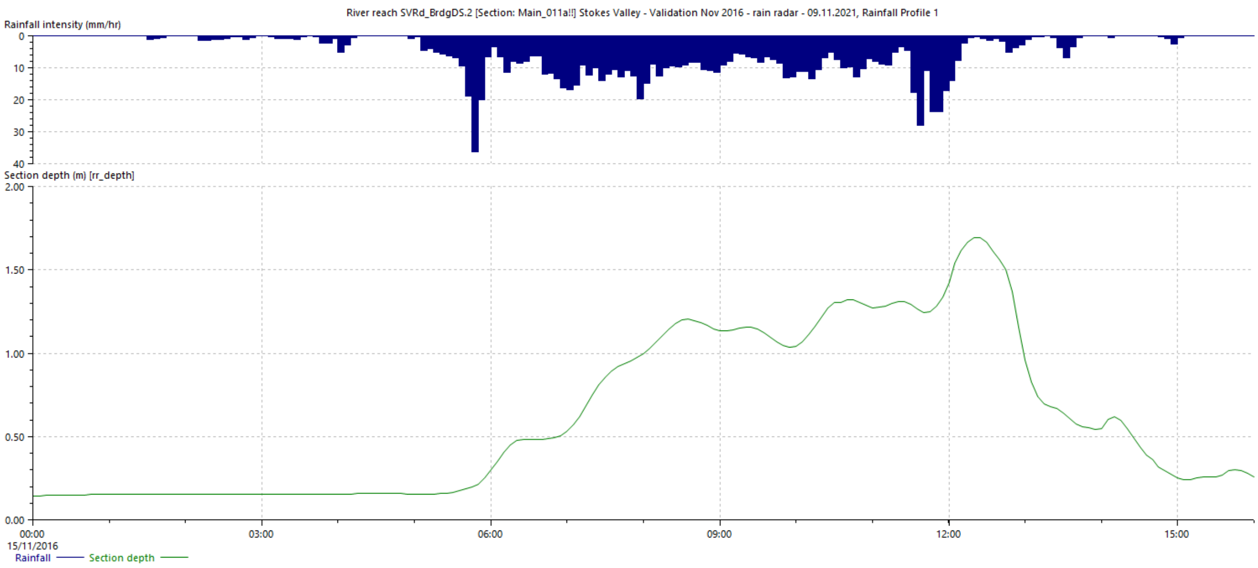


Figure 4-31: Flood depths from the model simulation through a cross-section taken directly downstream of Stokes Valley Road near 67 Stokes Valley Road.

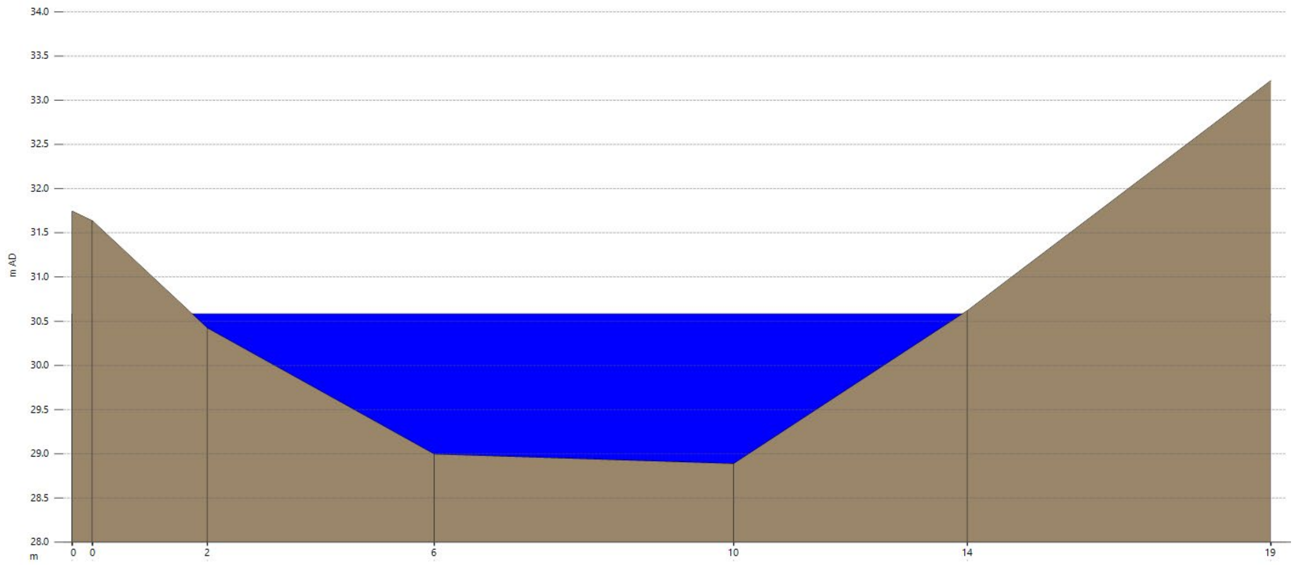


Figure 4-32: Flood levels from the model simulation through a cross-section taken directly downstream of Stokes Valley Road near 67 Stokes Valley Road.

4.2.2.4 Stokes Valley Stream – Delaney Park

Figure 4-34**Error! Reference source not found.** and Figure 4-35 show the depth of flow through a cross-section of Stokes Valley Stream parallel to Delaney Park. During the modelled event, the river reach peaks at a depth of 1.7 m. This appears slightly higher than the observed flood levels shown in Figure 4-33. It appeared that the properties on the eastern bank that are flooded in the model simulation (up to ~30 cm) may not have been affected by flooding in the event. It is, however, unclear where flow extended up to at this location based on photographs following the peak event. We recommend further investigation to determine whether there was flooding into this property and whether there is an issue with the bank levels at this location or event rainfall was overestimated.

Figure 4-33: Stokes Valley Stream near Delaney Park following the 15 November 2016 rainfall event.

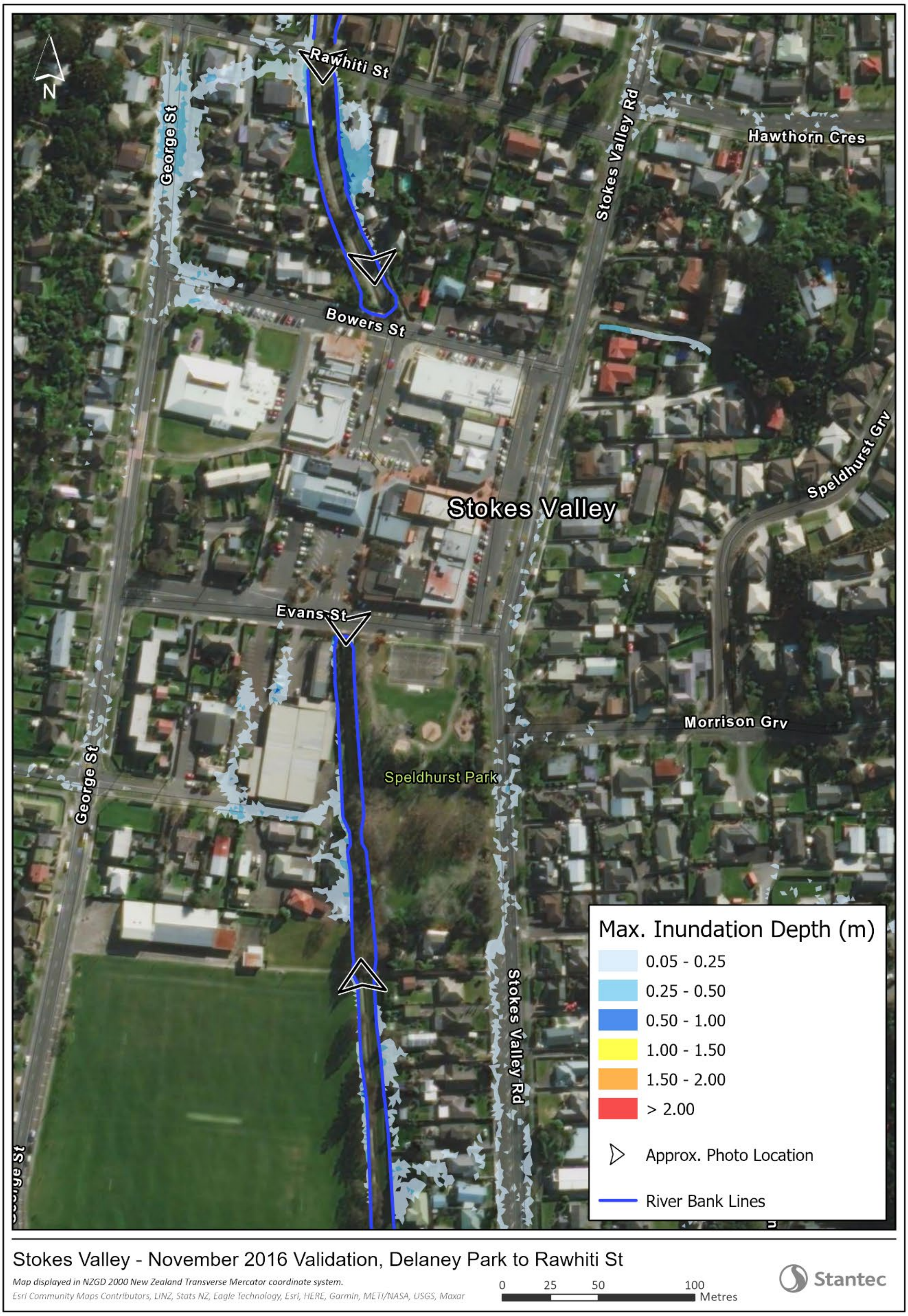


Figure 4-34: Boundary of the modelled 1D channel (blue line) and maximum 2D flood depths around the stream from the North of Delaney Park to Rawhiti St.

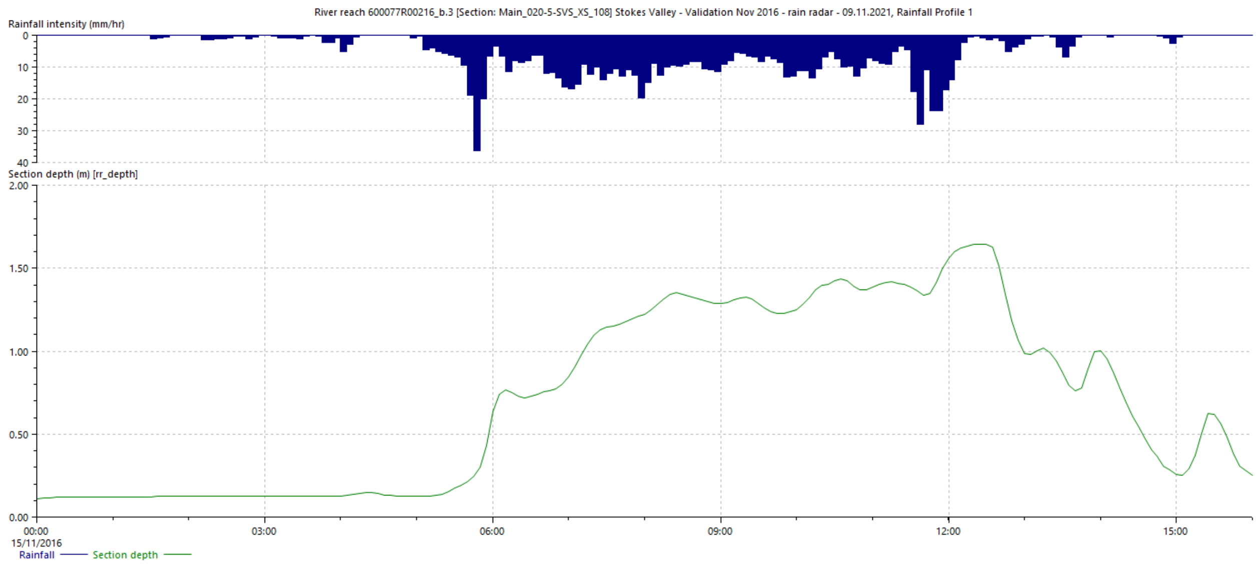


Figure 4-35: Flood depths from the model simulation through a cross-section taken from the Stokes Valley Stream near Delaney Park.

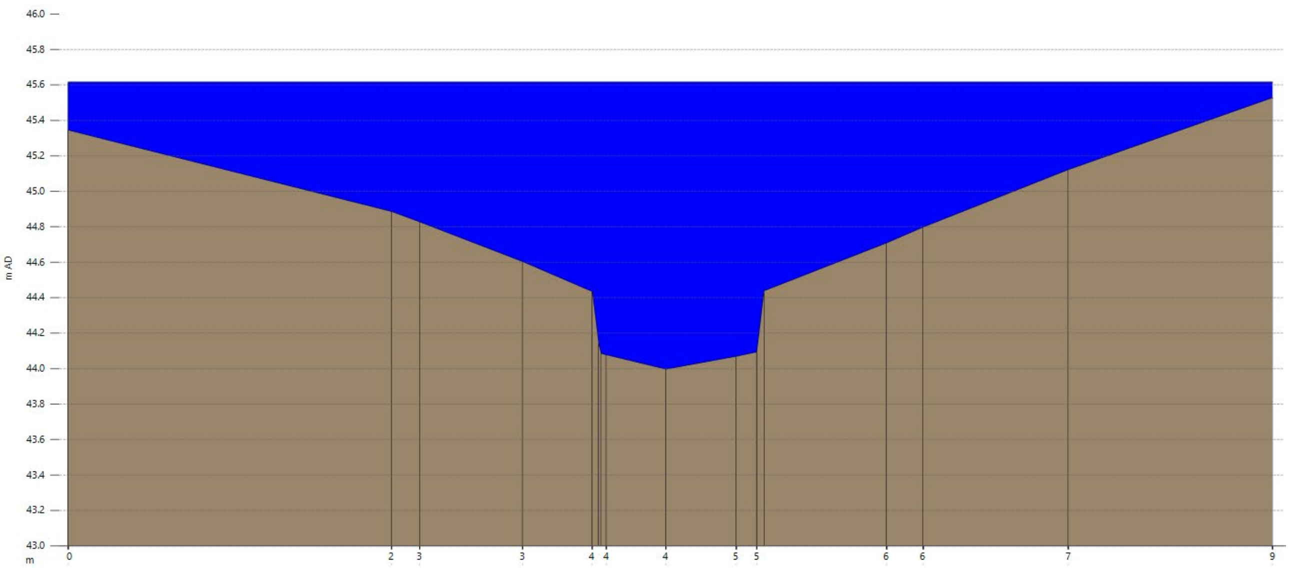


Figure 4-36: Flood levels from the model simulation through a cross-section taken from the Stokes Valley Stream near Delaney Park.

4.2.2.5 Stokes Valley Stream – Rawhiti Street

Figure 4-38 and Figure 4-39 show the depth of flow through a cross-section of Stokes Valley Stream directly upstream of Rawhiti Street. During the modelled event, the channel reaches a peak depth of 1.8m, while modelled bank line heights range between approximately 1.6m to 2m. It is unclear whether flood levels exceeded banks at this location due to the time the photograph was taken (and the obstruction from vegetation); however, it is possible there is some slight overestimation in modelled flooding (North part of Figure 4-34 and Figure 4-37).

Figure 4-37: Stokes Valley Stream looking upstream from Rawhiti Street following the 15 November 2016 rainfall event.

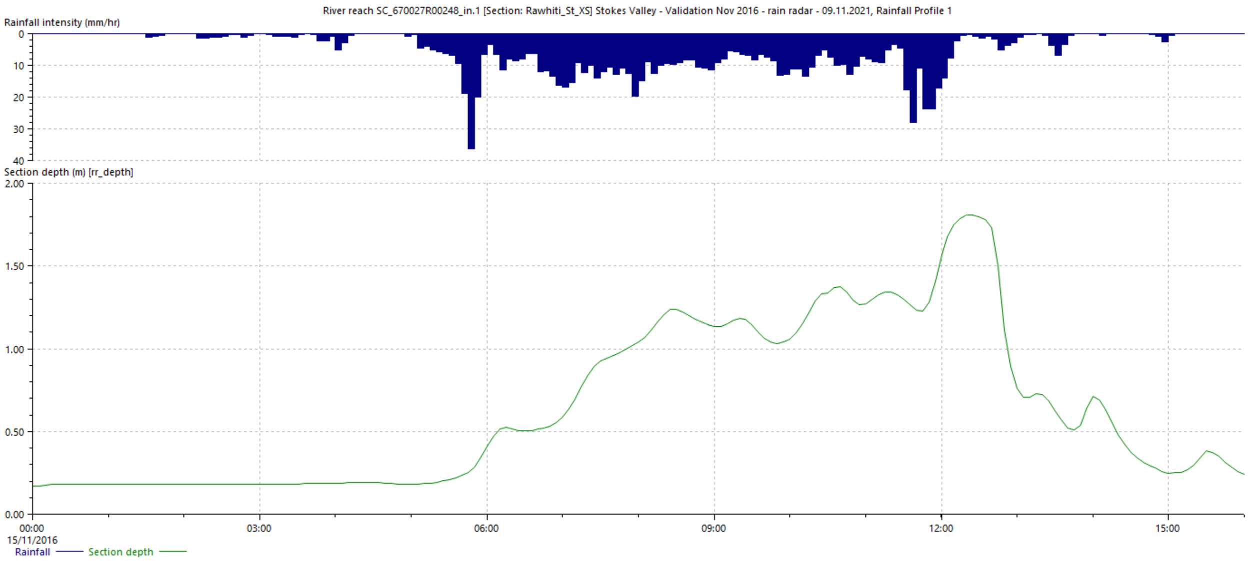


Figure 4-38: Flood depths from the model simulation through a cross-section taken from the Stokes Valley Stream directly upstream of Rawhiti Street.

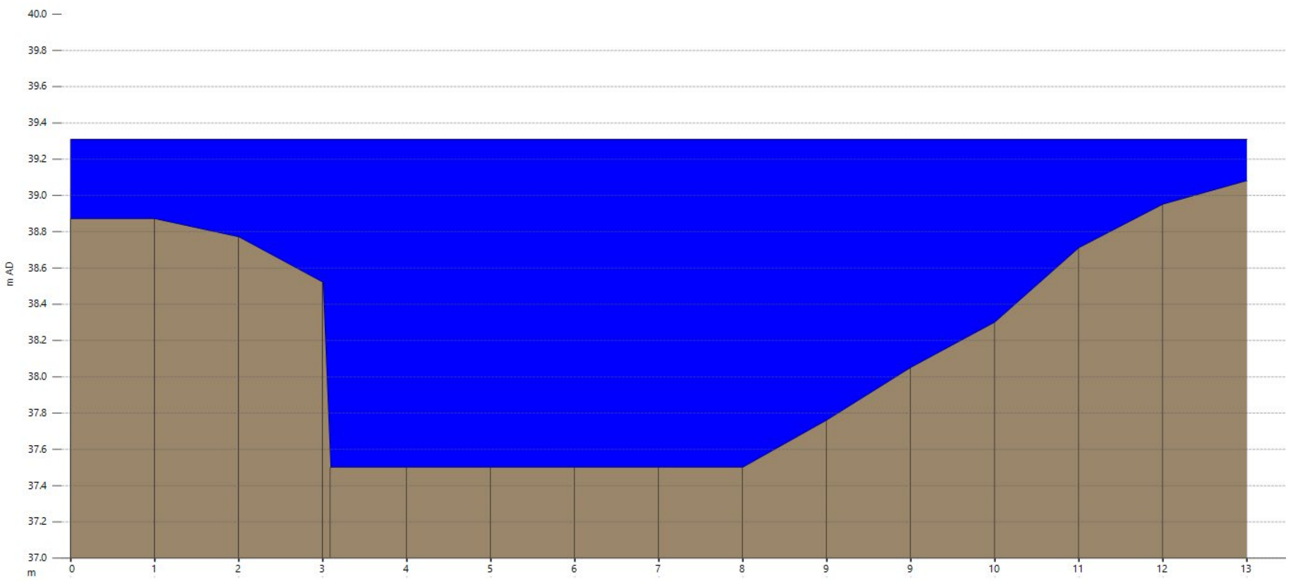


Figure 4-39: Flood levels from the model simulation through a cross-section taken from the Stokes Valley Stream upstream of Rawhiti Street.

4.2.2.6 Stokes Valley Stream – Evans Street Arch (Upstream)

The model simulation indicates flooding is mostly contained to the stream corridor (Figure 4-34) at this location (although the 1D western banks are slightly overtopped) While photographs are unclear, it appears there is no evidence of flooding outside of the channel or above the obvert of the Evans Street arch culvert. The minor flooding into properties from the western stream bank in the model simulation may be explained by the fence that has not been modelled at this location and the location of the 1D channel banks. Validation is considered reasonable at this location.

Figure 4-40: Stokes Valley Stream looking upstream of the Evans Street Arch from Evans Street.

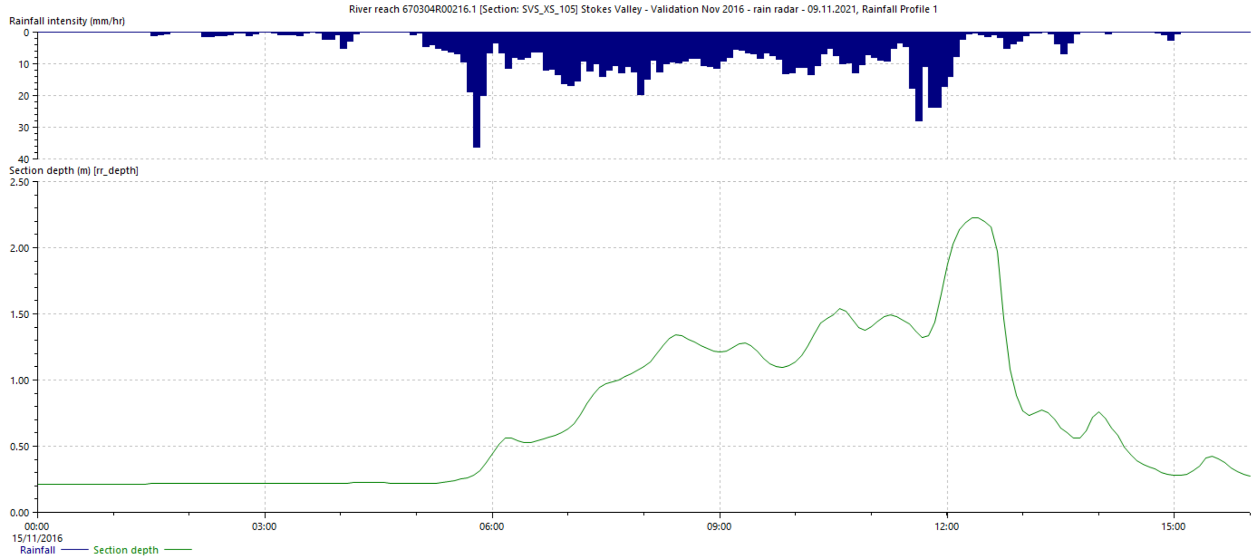


Figure 4-41: Flood depths from the model simulation through a cross-section taken from the Stokes Valley Stream directly upstream of Evans Street.

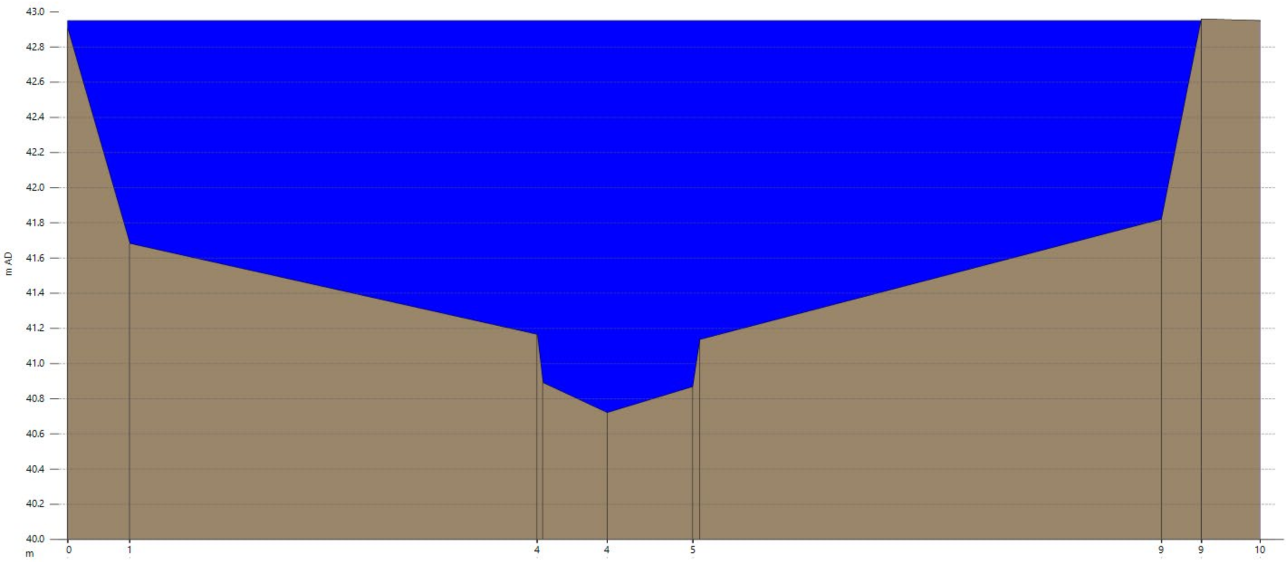


Figure 4-42: Flood levels from the model simulation through a cross-section taken from the Stokes Valley Stream directly upstream of Evans Street.

4.2.2.7 Stokes Valley Stream – Evans Street Arch (Downstream)

The observed flood levels shown in Figure 4-43 indicate flow has extended two-thirds of the way up the eastern bank and possibly the western bank, although this is difficult to confirm. Modelled flood depths through a cross section (Figure 4-44 and Figure 4-45) taken directly downstream of the Evans Street Arch indicates flood depths of approximately 1.1m, which is approaching the top of the bank. There is no overtopping of the western bank. Modelled levels appear reasonable on the western side of the channel; however, it is unclear whether the eastern bank was overtopped in the 2016. Further investigation would be required to confirm whether the properties on the eastern bank were inundated in the event as shown in the model simulation.

Figure 4-43: Outlet of Evans Street Arch following the 15 November 2016 rainfall event.

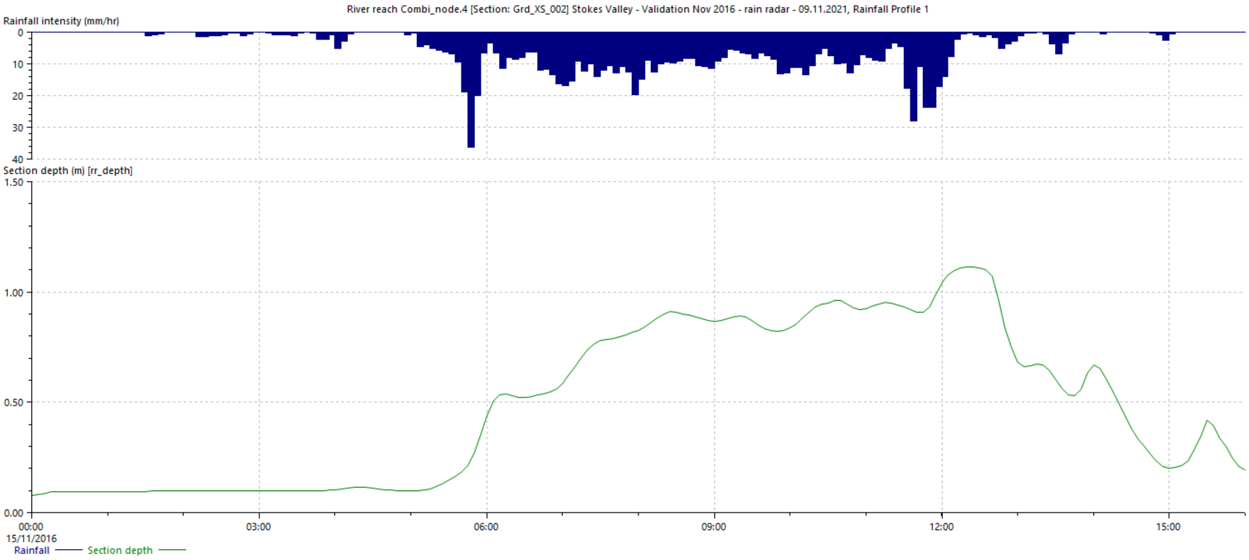


Figure 4-44: Flood depths from the 15 November 2016 model simulation through a cross-section taken downstream of Evans Street arch on Bowers Street.

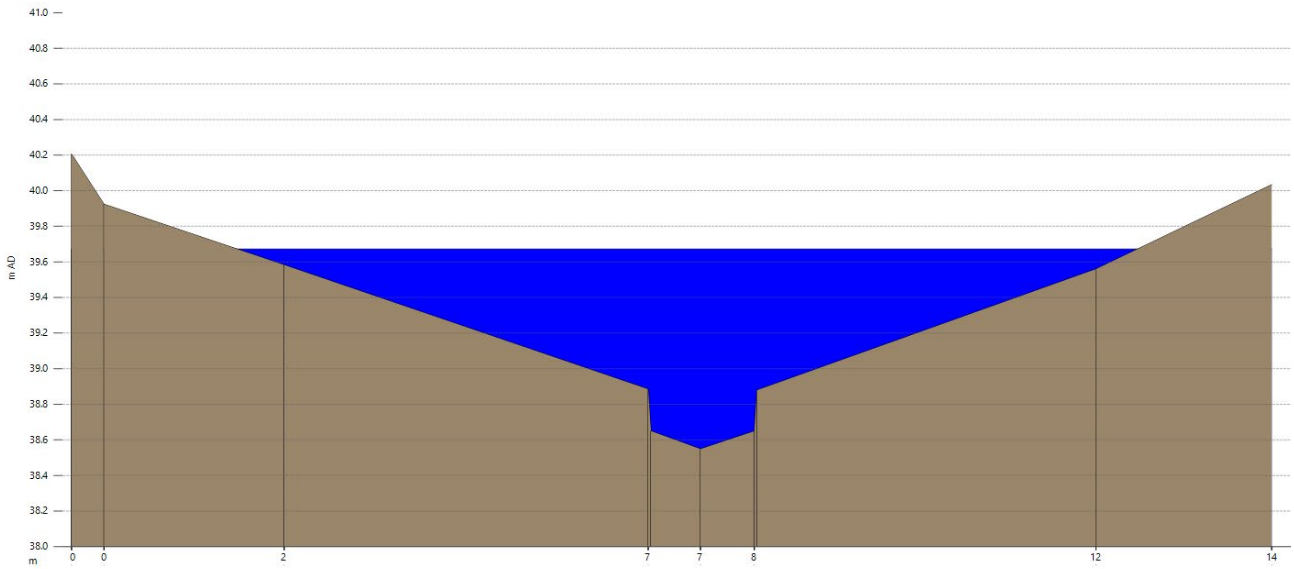


Figure 4-45: Flood levels from the 15 November 2016 model simulation through a cross-section taken downstream of Evans Street arch on Bowers Street.

4.2.2.8 Stokes Valley Stream – Kamahi Park (Upstream)

There is a significant amount of flooding shown in the model simulation near Kamahi Park (

Figure 4-47). However, it is unclear whether the banks were overtopped at this location (Figure 4-46) due to the time the photo was taken (not during the peak of the event). It is possible the model has overestimated flooding at this location, although it is unclear without further evident of flooding during this event at this location.

Figure 4-46: Upstream at Kamahi Park, 1200mm culvert (left), downstream Twin 1200mm barrels under crossing (right)

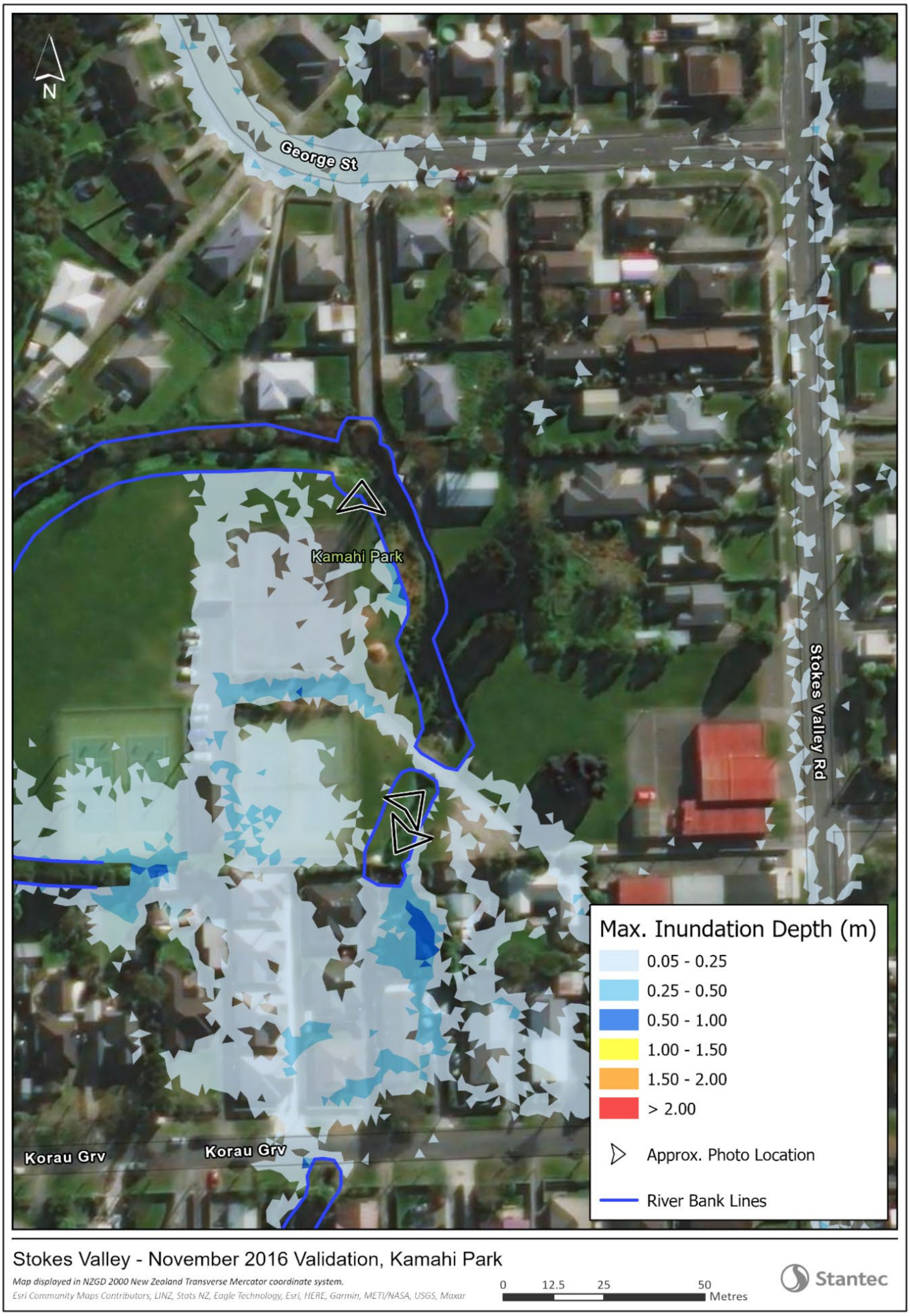


Figure 4-47: Boundary of the modelled 1D channel (blue line) and maximum 2D flood depths near Kamahi Park (15 November 2016 event)

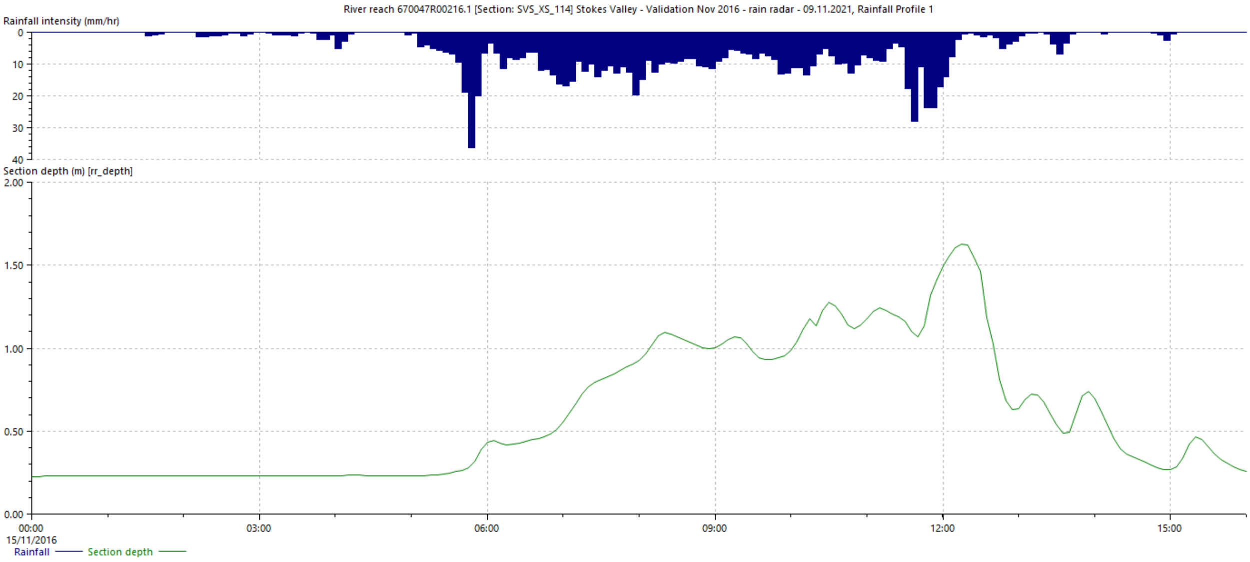


Figure 4-48: Flood depths from the 15 November 2016 model simulation through a cross-section taken directly upstream of the twin 1200mm barrels near Kamahi Park.

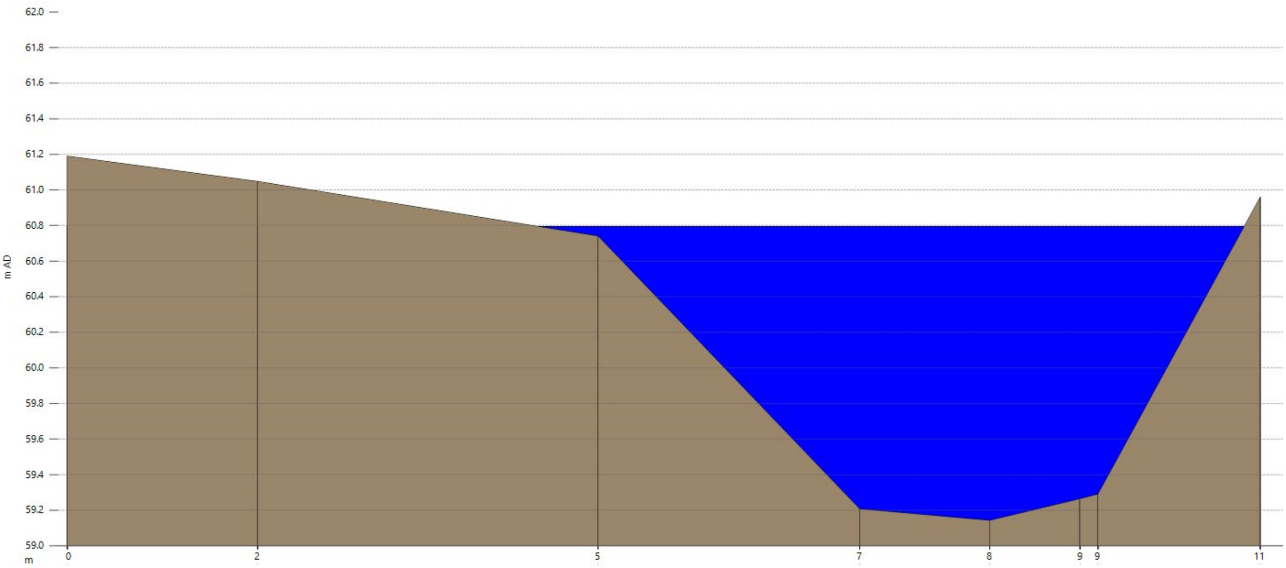


Figure 4-49: Flood levels from the 15 November 2016 model simulation through a cross-section taken directly upstream of the twin 1200mm barrels near Kamahi Park.

4.2.2.9 Stokes Valley Stream – Kamahi Park (Downstream)

Figure 4-50 shows that flood debris was left on the screen and maximum flood levels got close to the top of the culvert wing-wall. The model simulation indicates there was overtopping of some channel banks, but not directly at the location of the screen (

Figure 4-47). Flood depths through a cross-section (Figure 4-51 and Figure 4-52) taken directly upstream of the Kamahi Park screen indicates flood depths of approximately 1.8m, which may be slightly more than observed flooding. Modelled results appear reasonable at this location; however, may be overestimated at some channel banks. Further information on the observed event would be required to further validate the model at this location.

Figure 4-50: Stokes Valley Stream – trash screen near Kamahi Park following the 15 November 2016 rainfall event.

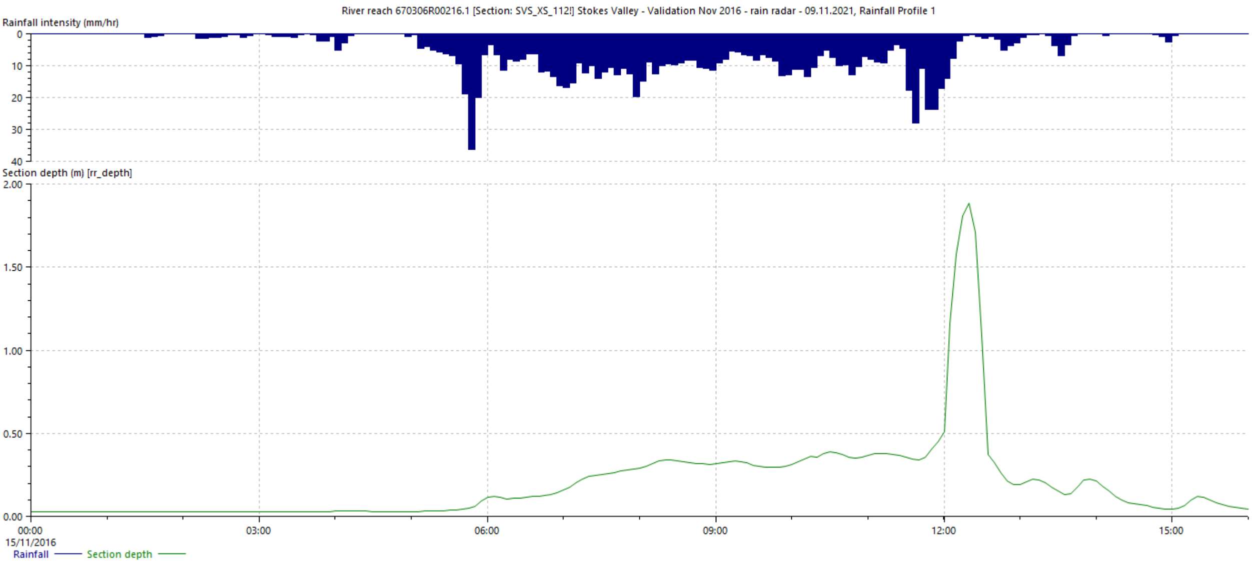


Figure 4-51: Flood depths from the 15 November 2016 model simulation through a cross-section taken directly upstream of the screen near Kamahi Park.

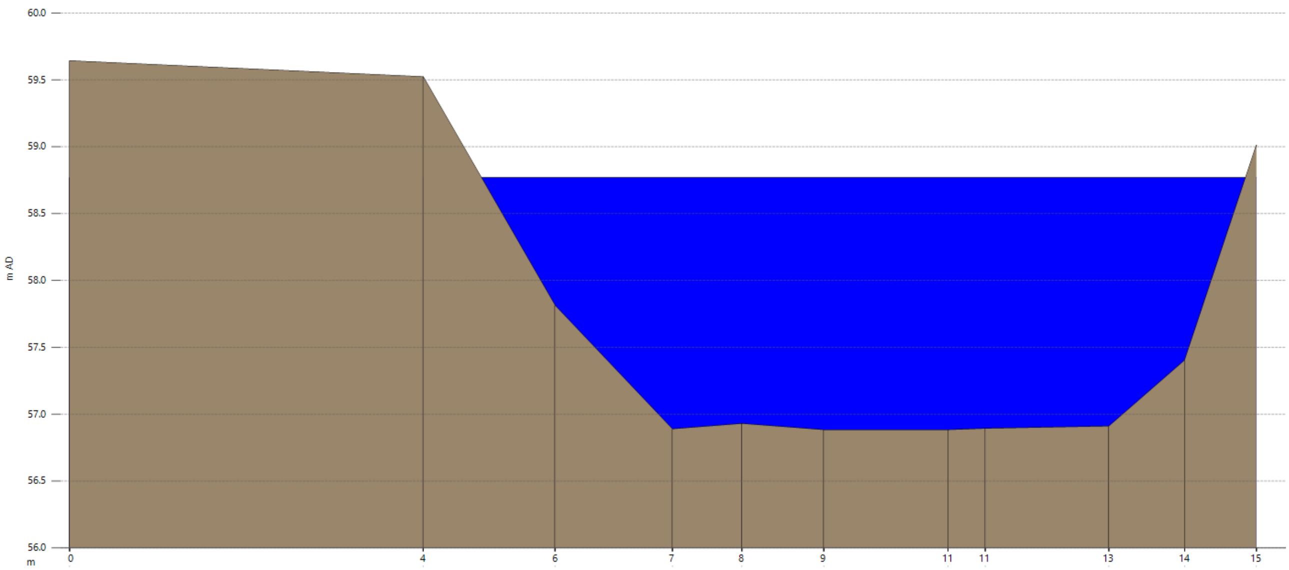


Figure 4-52: Flood levels from the 15 November 2016 model simulation through a cross-section taken directly upstream of the screen near Kamahi Park.

4.2.2.10 27 Manuka Street

The modelled depth of flooding outside of the channel banks at a small stream running through the property at 27 Manuka Street was approximately 6cm to 10cm (Figure 4-54 to Figure 4-56) This appears to be in accordance with the observed flooding in 2016 at this location Figure 4-53.

Figure 4-53: Minor stream running through the property 27 Manuka Street following the 15 November 2016 rainfall event.

Figure 4-54: Boundary of the modelled 1D channel (blue line) and maximum 2D flood depths at 27 Manuka Street (15 November 2016 event).

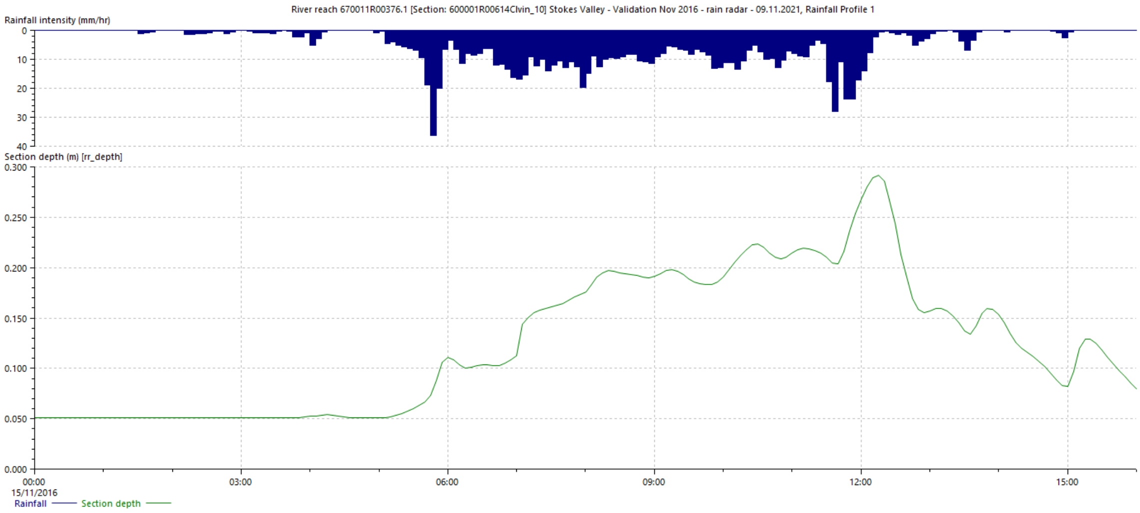


Figure 4-55: Flood depths from the 15 November 2016 model simulation through a cross-section taken at 27 Manuka Street.

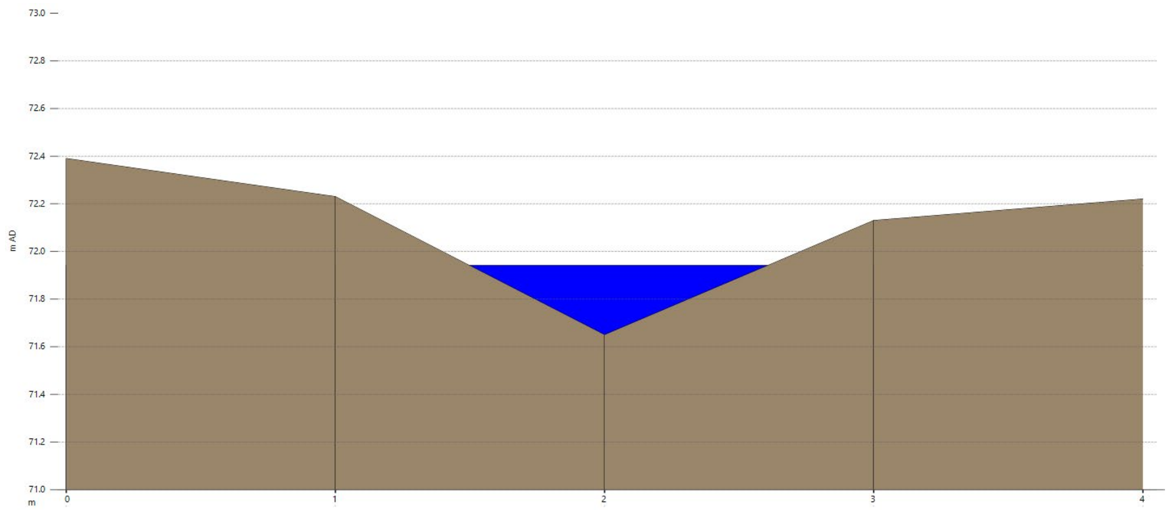


Figure 4-56: Flood levels from the 15 November 2016 model simulation through a cross-section taken at 27 Manuka Street.

4.2.3 Summary

The 15 November 2016 model simulation appears to compare well against the observed flooding shown in photos taken following the rainfall event in the lower reaches of the Stokes Valley Stream channel where flood depths within the channel appear similar to those indicated in the photos. At some locations in the middle reaches of Stokes Valley, modelled depths were greater than observed flooding. It is, however, difficult to determine observed flood levels from the photographs. This could be partly a result of localised issues with bank levels. All photos obtained in 2016 were around the channel network, particularly the main channel for the Stokes Valley Stream. This has limited the observed evidence for validation of the model using this event.

We recommend further investigation into flooding issues, particularly around Kamahi Park and the Evans Street Arch. This would involve approaching property owners to confirm their observations of flooding in and near their properties. There were also no photos taken around the area surrounding Tawhai Street and Horoeka Street following the 15 November 2016 event. This may be an area of interest due to the significant flooding in the model simulation; therefore, we also recommend further investigation into observed flooding in this area.

7.1 LIMITATIONS

The purpose of the validation is to gain a high-level understanding of the performance of the Stokes Valley model. The following outlines assumptions and limitations of this exercise:

- No actual flood levels were obtained in this exercise and comparisons were made between photos taken following the storm event on 15 November 2016 and on 8th December 2019 and the model results for each simulation.
- Comparison for the 2016 event has focused around 1D river reaches in the model and approximate observed depths within the channels, as this is where clear debris lines were observed. No evidence of flooding outside of the main channels was obtained in 2016.
- The 2019 comparison included both 1D river reaches, and some areas of ponding away from the main channels. These areas of ponding validate well and are consistent between modelled and observed flood extents.
- No consideration has been given to the functionality of structures, including bridges, screens and the losses associated with these.
- Rainfall radar data was available for the two validation events. As this data is catchment averaged, no spatial variation within the catchment is included. For the November 2016 event this data was compared against and consistent with two rainfall gauges adjacent to the catchment. The rainfall radar data is assumed to be broadly representative of temporal rainfall distribution and total volume within the Stokes Valley catchment.
- No allowance for antecedent conditions was made.
- Complaint data was not included in the analysis as no information was provided for either the 2019 or 2016 event in the received data at the time of the model construction. Inclusion of more recent data including feedback from the community engagement process can be included in further work going forward

5. MODEL SENSITIVITY AND FREEBOARD

Stantec was engaged by Wellington Water (WWL) in 2019 to undertake a model sensitivity and freeboard analysis assessment for the Stokes Valley catchment, following the completion of the model build.

5.1 SENSITIVITY ANALYSIS

This work was undertaken in 2019 with the intention of understanding the sensitivity of the stormwater network to a range of different model parameters. These would be used to select values for freeboard allowance to develop a baseline flood extent to be used for assigning minimum building floor levels for Stokes Valley. The following section describes the sensitivity scenarios that were examined to inform the freeboard selection process. The following scenarios were simulated for analysis:

1. 50% increase in rainfall intensity
2. All inlets fully blocked
3. All inlets partially blocked (50%)
4. Removal of tailwater

All scenarios were compared with maximum flood depths (m) from what is referred to as the base case scenario; the 100yr ARI 12hr nested profile with +20% allowance for climate change, see **Error! Reference source not found.** . All maps show the difference in depth between the base case scenario and the sensitivity simulation.

5.1.1 Rainfall (50% Increase)

A 50% increase in rainfall intensity was applied to the base 100yr climate change scenario as shown in Figure 5-1. The network is highly sensitive to the increase in rainfall; much more so than other scenarios described in the following sections. The catchment is relatively steep so additional runoff collects only in the flatter areas around the open channels and towards the bottom of the catchment before the connection to the Hutt River. Water pools in these locations due to the various channel constrictions and tailwater effects.

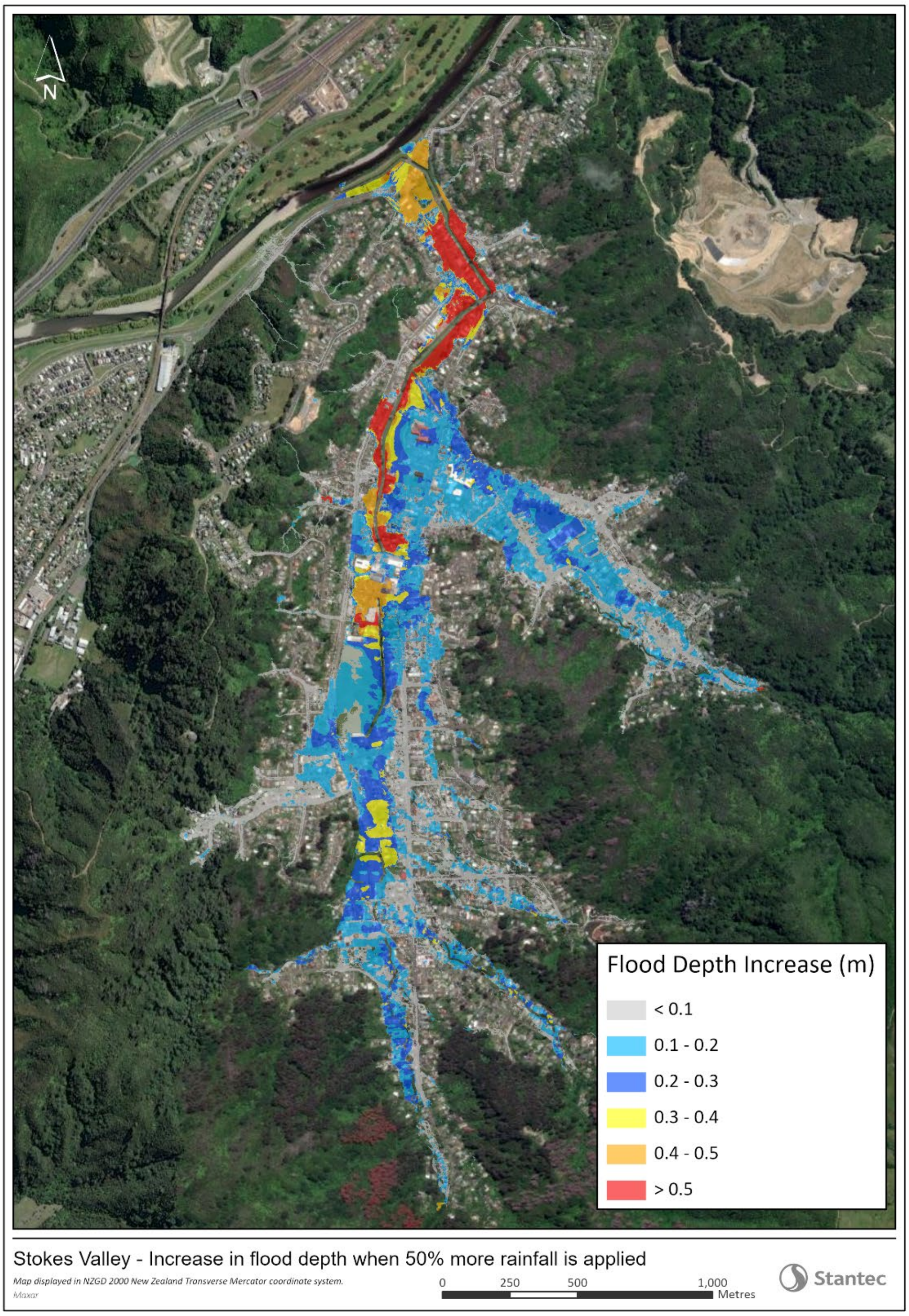


Figure 5-1: Difference in depth between the 100yr ARI with climate change scenario and the 50% increased rainfall scenario.

5.1.2 All Inlets Fully Blocked

For this scenario, inlets were 90% blocked. To achieve this, a total of 177 conduits immediately downstream of inlets were filled to 90% with sediment. See Figure 3-5 for inlet locations. InfoWorks ICM does not allow sediment depth to be equal to the pipe diameter. This is due to the base flow and the preissmann slot accounting for 10% of conduit space. For example, a 600mm conduit could only be filled with 540mm sediment due to the 60mm of base flow.

The greatest increase in flood depth occurs around the culvert under Eastern Hutt Rd immediately upstream of the exit to the Hutt River. This is because when the culvert is blocked the flow first needs to reach a certain depth before it can flow over the road and leave the catchment. This also explains the flooding further upstream between Delaney Park and Rawhiti St where there are multiple culverts which when blocked will have a noticeable impact on flow.

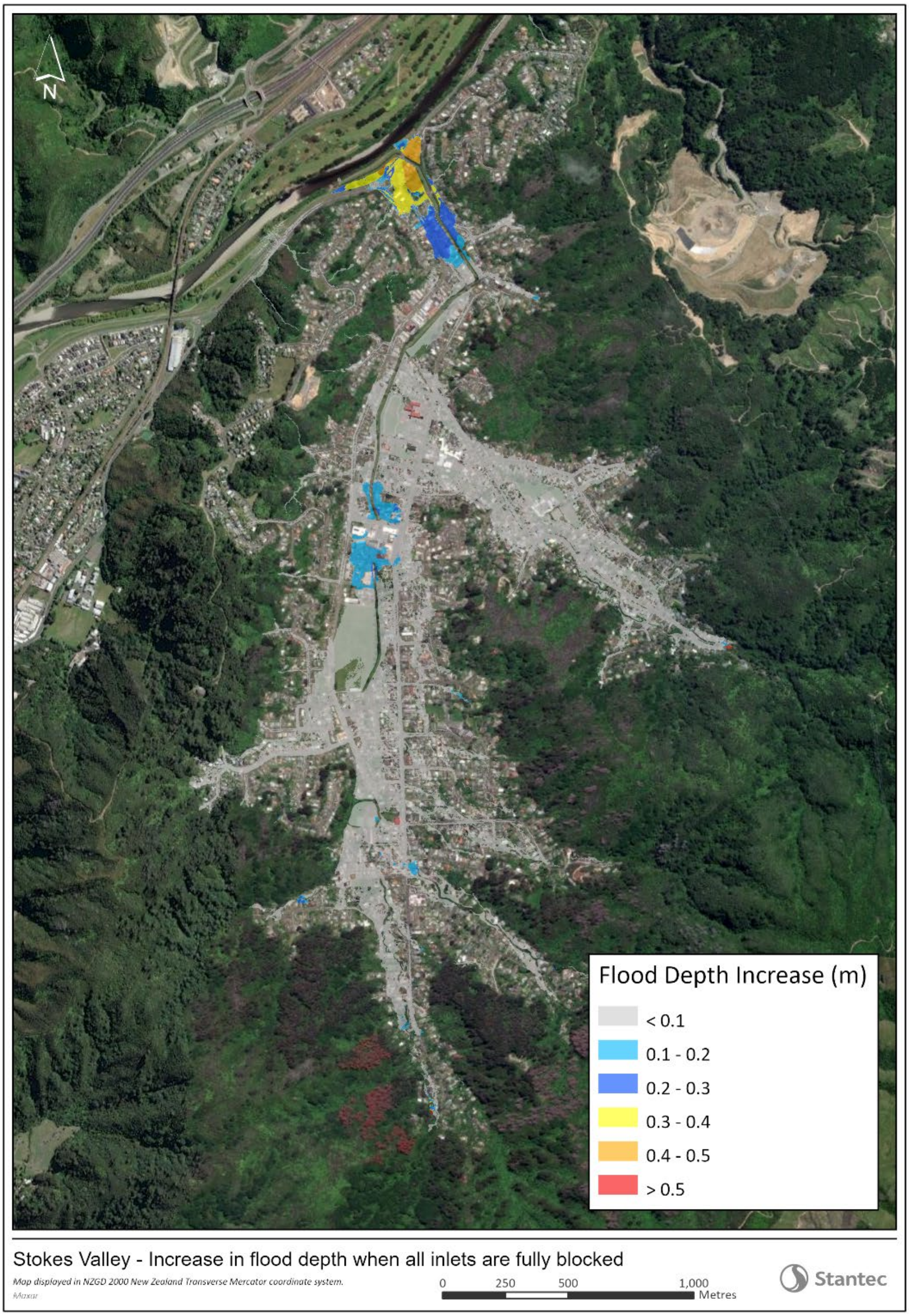


Figure 5-2: Difference in depth between the 100yr ARI with climate change scenario and the 100% blocked scenario.

5.1.3 All Inlets Partially Blocked

For this scenario, conduits immediately downstream of inlets were filled with sediment to half the diameter of the culvert. A total of 177 conduits were blocked with the results shown in Figure 5-2. For similar reasons to the fully blocked scenario the greatest difference is seen at the Eastern Hutt Rd culvert and the culverts between Delaney park and Rawhiti St.

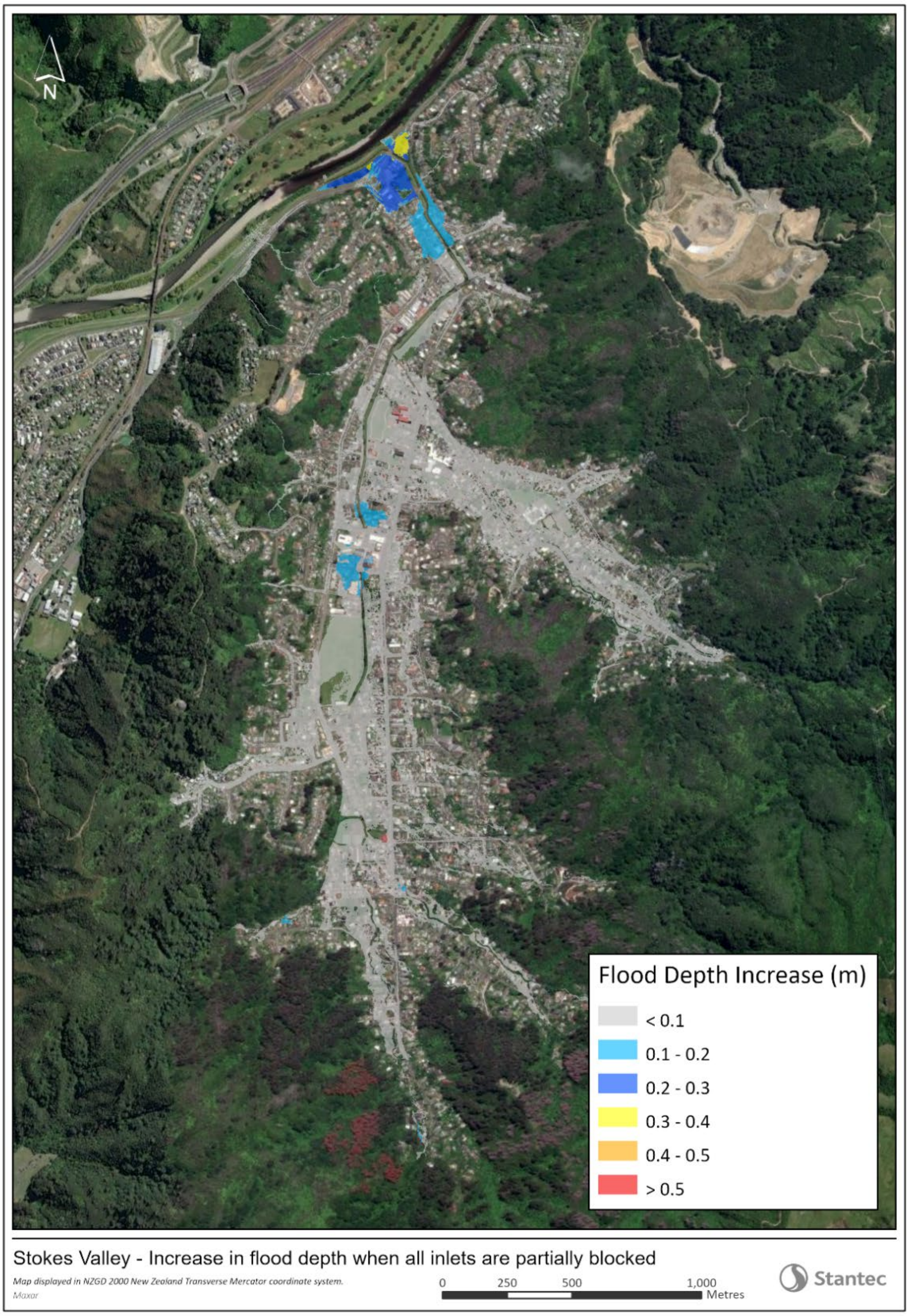


Figure 5-3: Difference in depth between the 100yr ARI with climate change scenario and the 50% blocked scenario.

5.1.4 Low Tailwater

The tailwater level was reduced by removing the Hutt River inflow to determine the sensitivity of the network to the flows in the Hutt River. Results suggest that the Stokes Valley catchment is not sensitive to levels in the Hutt River. This is likely because the catchment is steep and high enough that the effects of varying the tailwater level doesn't have a significant impact on the direction or velocity of flow.

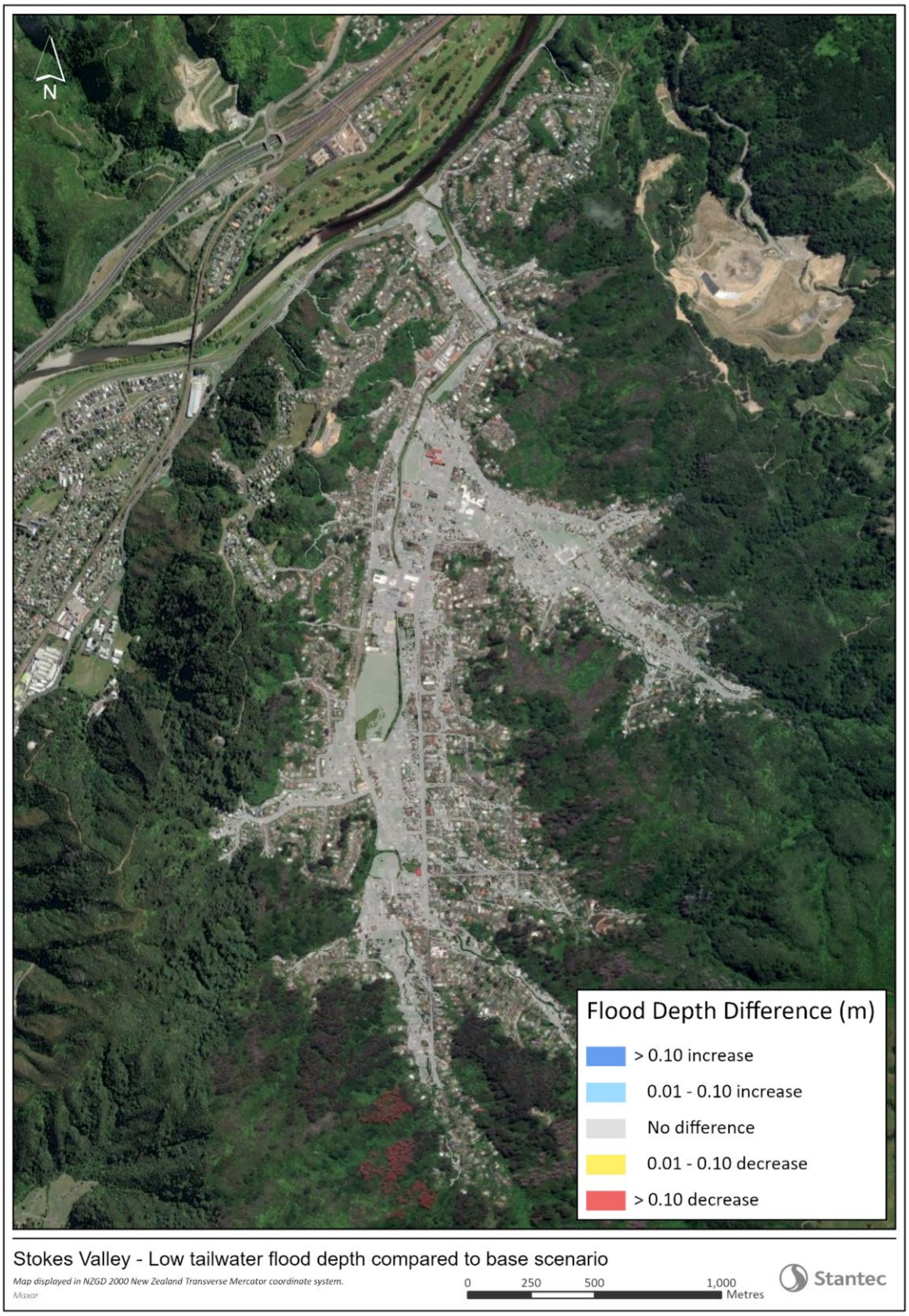


Figure 5-4: Difference in depth between the 100yr ARI with climate change scenario and the low tailwater scenario.

5.2 FREEBOARD METHODOLOGY AND SELECTION

5.2.1 Freeboard Allowance Selection

Freeboard values were determined by examining the results from the scenarios described in Section 5.1 and determining how sensitive the Stokes Valley network was to these key model parameters. After discussions with WWL, the freeboard allowances shown in Figure 5-5 were applied across the network.

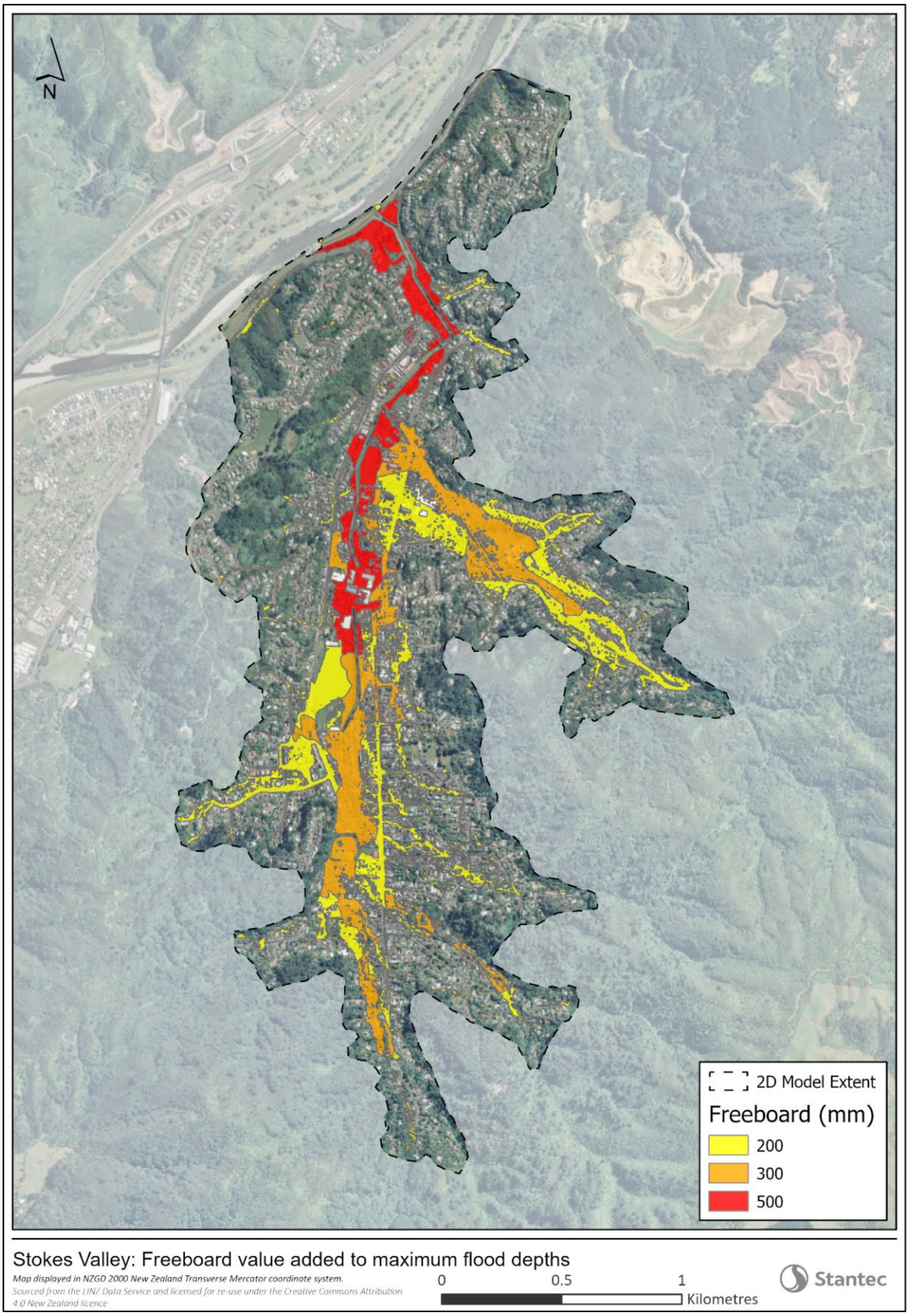


Figure 5-5: Freeboard as applied across the Stokes Valley model network.

5.2.2 Freeboard Simulation Setup

The freeboard simulation was developed as described in “Dynamic Freeboard Analysis – Tawa”, Jacobs Memorandum, 04 October 2017. The agreed freeboard values were added to maximum flood depths (m) for the 100yr ARI 12hr nested profile with +20% allowance for climate change. These were processed in GIS and imported back into the network as IC Zone – hydraulics (2D) polygons. All 1D network elements were deleted including all sub-catchments, 1D pipes, and 1D river reaches. The IC Zone – hydraulics (2D) polygons were then used to create an Initial Conditions 2D database object. A simulation including only the Initial Conditions (2D) file was run for five minutes. This allowed the maximum water levels to spread naturally across the catchment in order to establish baseline flood extent to be used for assigning minimum building floor levels for Stokes Valley.

5.2.3 Results

Maximum flood depths with freeboard allowance are shown below in **Error! Reference source not found.**.

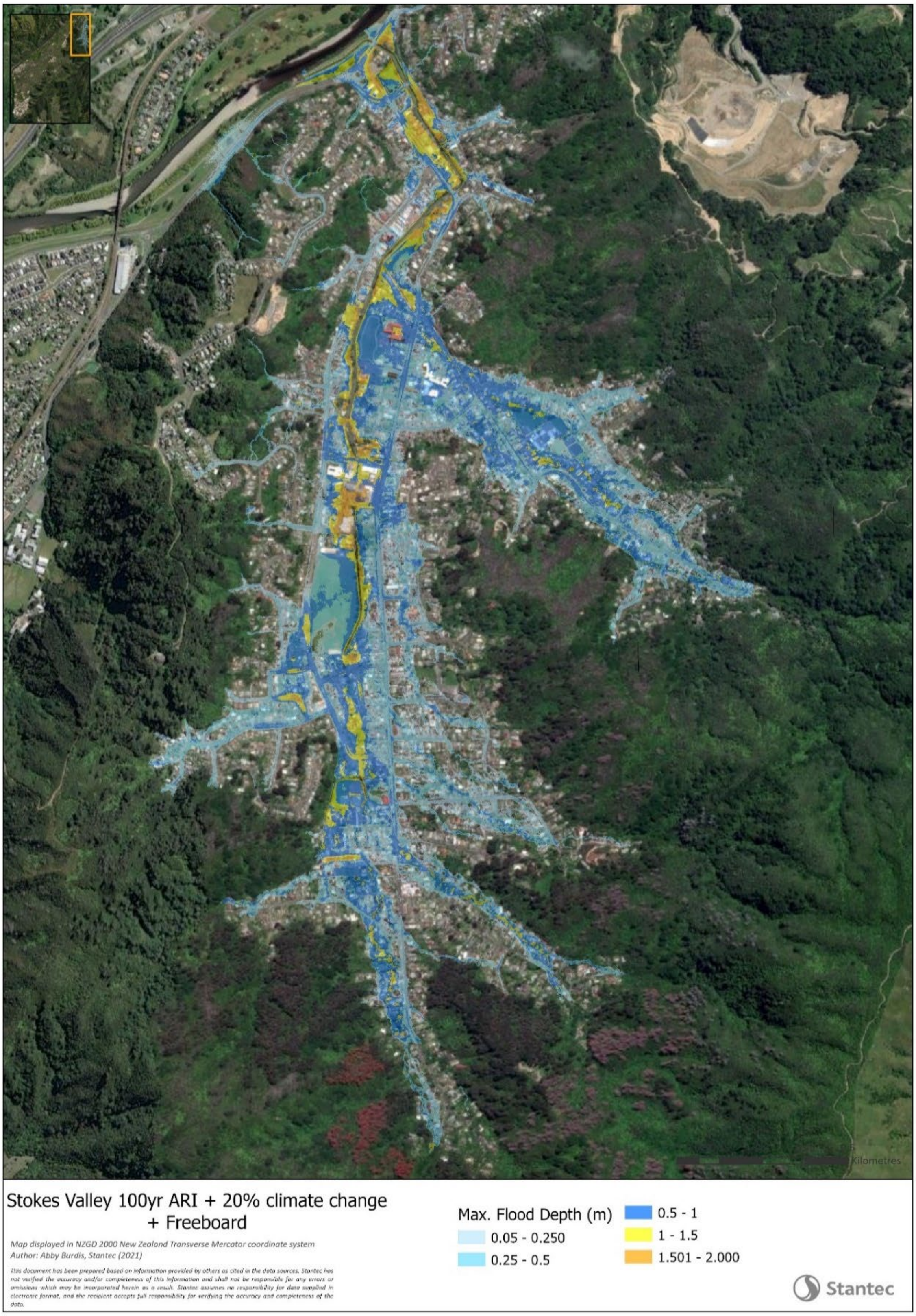


Figure 5-6 Maximum flood depths with freeboard allowance

6. CONCLUSIONS AND RECOMMENDATIONS

- This report describes the build phase of the Stokes Valley Model.
- ICM and UMM were used to develop 1D channel and pipe networks and link the two where required.
- Hydrology parameters were calculated from regional GIS layers and applying the hydrological method described in *Quick Reference Guide for Design Storm Hydrology* (Cardno 2016) within UMM. These parameters were imported to ICM.
- A 2D mesh zone was developed which included all pipes and network nodes.
- Survey was carried out in two phases. The main structures within the Stokes Valley channel were surveyed as well as a number of 'minor' structures.
- Preliminary results were mapped for the 10 and 100 year design events based on HIRDS v3 rainfall.
- It is recommended that where possible flow and/or level data should be collected within the catchment to assist in future calibration and validation of the model.
- The installation of a rain gauge within the catchment could be considered to support.

7. REFERENCES

Cardno. (2016). *Quick Reference Guide for Design Storm Hydrology.* Cardno.
GHD. (2015). *Stokes Valley Stormwater Hydraulic Model Phase 1 - Scoping Study.* Wellington Water.
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MWH (Now Stantec). (2016). *Ground Model ASsessment Summary Report.* Wellington: MWH.
MWH (Now Stantec). (2016). *Stokes Valley Hydraulic Model Build.*
Stantec. (2017). *Stokes Valley Model Validation of November 15 2016 event.*
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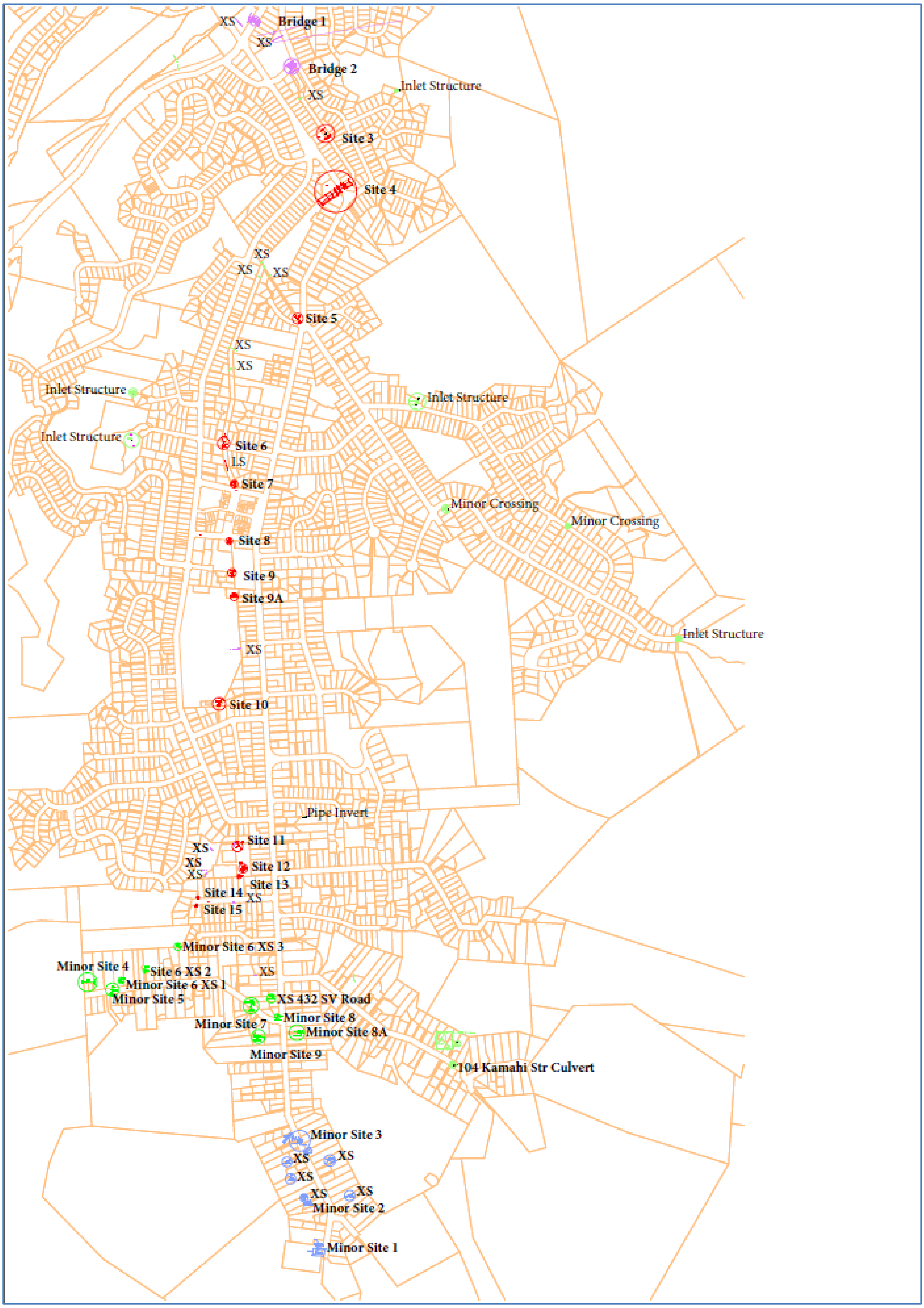
Wellington Water Ltd. (2017). *Regional Stormwater Hydraulic Modelling Specifications V5.* Wellington Water.

Appendices

We design with community in mind

Appendix A: Survey

Two survey Work Packages were undertaken to provide input to the hydraulic model. The first Work Package was for the survey of all structures in the main Stokes Valley Stream channel and 10 representative structures in accessible tributaries and some inlet structures to the pipe network. The second Work Package was to survey minor channel cross sections mainly on private property and included delivery of letters to advise residents that surveyors would be accessing the stream channel. Appendix A includes a plan of the area and the locations of the locations surveyed between February to June 2016. This is an image of the CAD drawing provided to WWL. Horizontal datum is NZTM 2000 and Vertical Datum is Wellington Vertical Datum 1953.


XS
Bridge 1
XS
Bridge 2
Inlet Structure
XS
Site 3
Site 4
XS
XS
XS
Site 5
XS
XS
Inlet Structure
Inlet Structure
Inlet Structure
Site 6
LS
Site 7
Minor Crossing
Minor Crossing
Site 8
Site 9
Site 9A
Inlet Structure
XS
Site 10
Pipe Invert
Site 11
XS
XS
XS
Site 12
Site 13
Site 14
XS
Site 15
Minor Site 6 XS 3
Minor Site 4
Site 6 XS 2
XS
Minor Site 6 XS 1
Minor Site 5
XS 432 SV Road
Minor Site 7
Minor Site 8
Minor Site 8A
Minor Site 9
104 Kamahi Str Culvert
Minor Site 3
XS
XS
XS
XS
XS
Minor Site 2
Minor Site 1

Bridge 1

This bridge was modelled as a 7.3 x 4.3m trapezoid culvert due to instability in the model.

MWH

BUILDING A BETTER WORLD

FILE NOTE

DATE

JOB NO.

PROJECT Stokes Valley Bridge 1

SUBJECT Structure 1 - Outlet

INFORMATION FOR

ACTION BY

THIS NOTE RECORDS

- ☐ MEETING
- ☐ PHONE CALL
- ☐ THOUGHT
- ☐

- ☐ WITH
- ☐ ABOUT

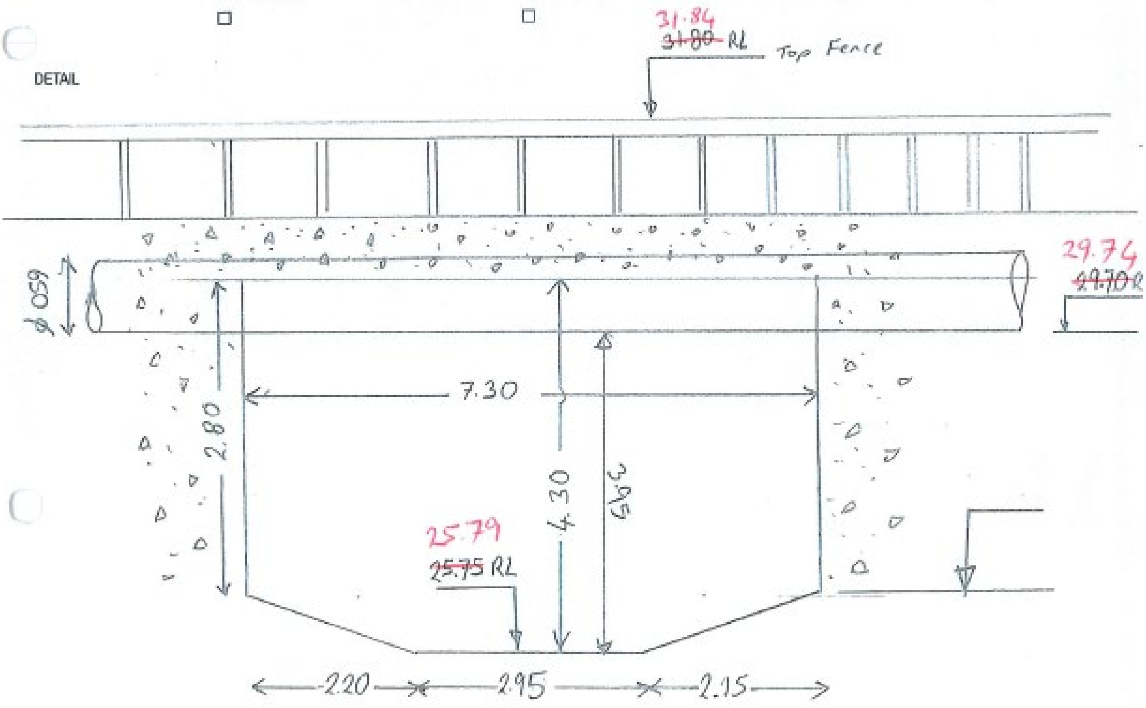
- ☐ CLIENT
- ☐ CONTRACTOR
- ☐ SUPPLIER
- ☐

BETWEEN

AND

TIME

DETAIL



Note: Pipe is approximately 1.9m downstream from the outlet portal

INITIALS

MWH

BUILDING A BETTER WORLD

FILE NOTE

DATE

JOB NO.

PROJECT structure 1 - Inlet
Stokes Valley

SUBJECT Bridge 1

INFORMATION FOR

ACTION BY

THIS NOTE RECORDS

- ☐ MEETING
- ☐ PHONE CALL
- ☐ THOUGHT
- ☐

- ☐ WITH
- ☐ ABOUT

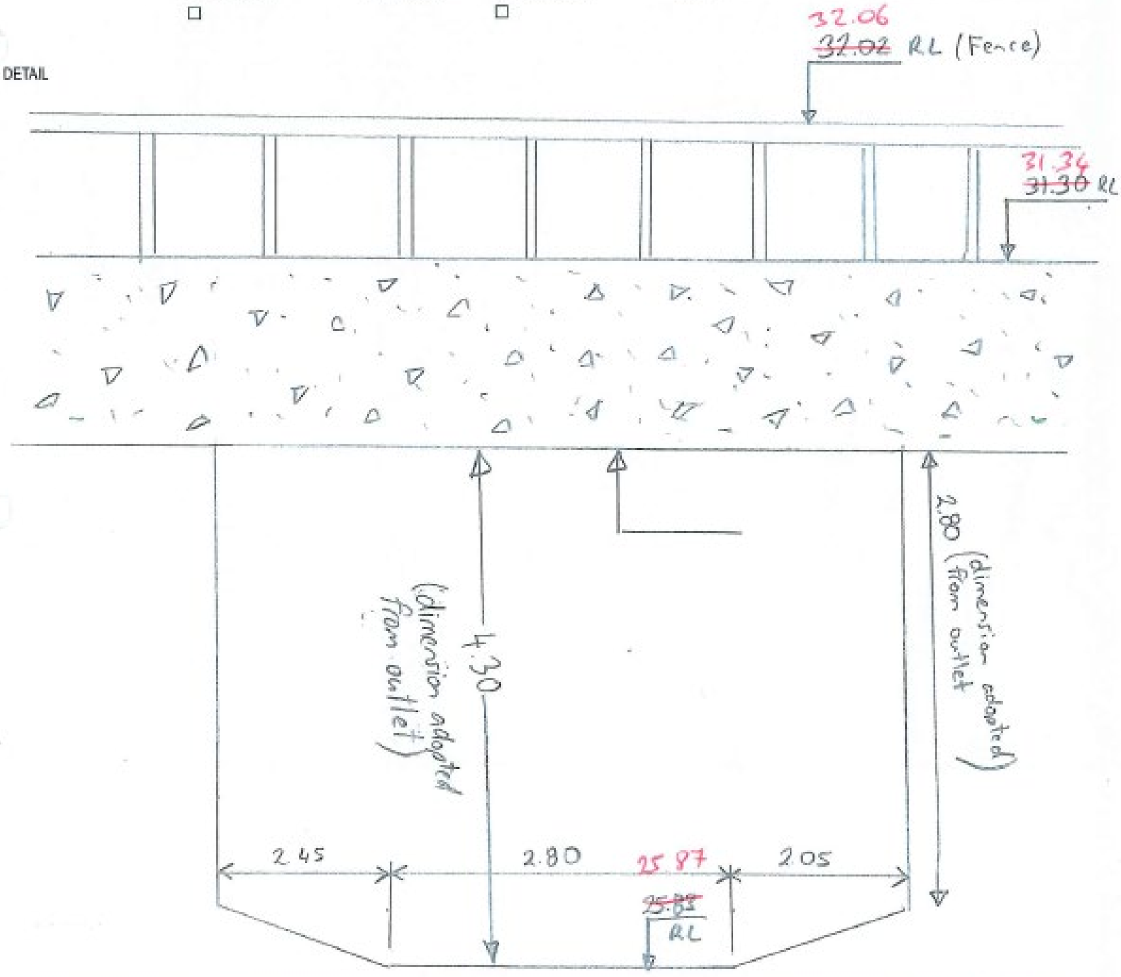
- ☐ CLIENT
- ☐ CONTRACTOR
- ☐ SUPPLIER
- ☐

BETWEEN

AND

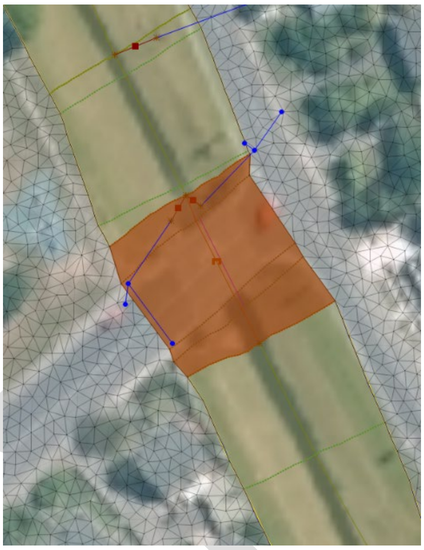
TIME

DETAIL



INITIALS

Bridge 2



MWH

BUILDING A BETTER WORLD

FILE NOTE

DATE

JOB NO.

PROJECT Stokes Valley Bridge 2

SUBJECT Structure 2 – Inlet

INFORMATION FOR

ACTION BY

THIS NOTE RECORDS
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- ☐ PHONE CALL
- ☐ THOUGHT
- ☐

☐ WITH
☐ ABOUT

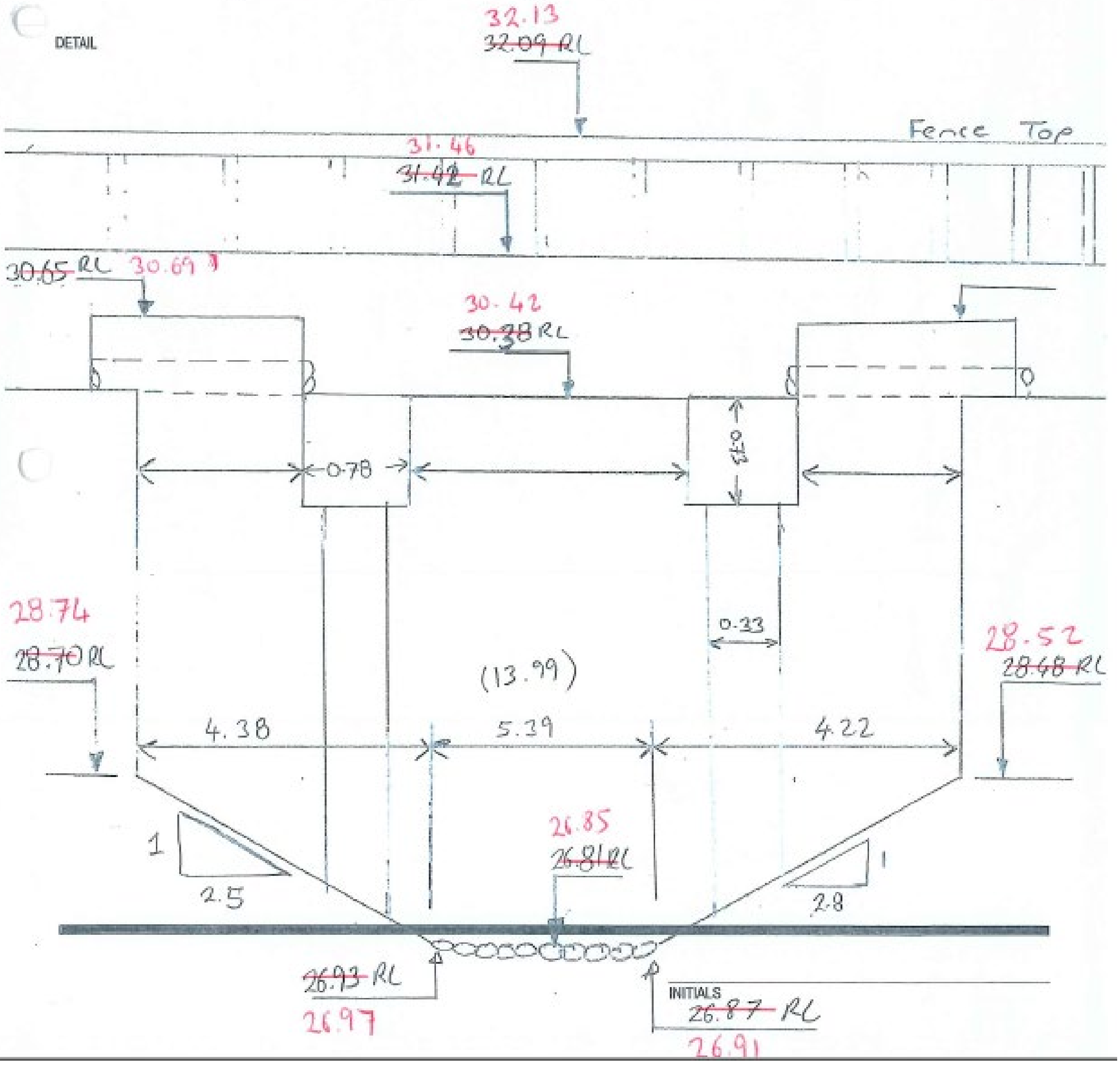
- ☐ CLIENT
- ☐ CONTRACTOR
- ☐ SUPPLIER
- ☐

BETWEEN

AND

TIME

DETAIL



INITIALS

MWH

BUILDING A BETTER WORLD

FILE NOTE

DATE

JOB NO.

PROJECT Stokes Valley Bridge 2

SUBJECT Structure 2 - outlet

INFORMATION FOR

ACTION BY

THIS NOTE RECORDS
- ☐ MEETING
- ☐ PHONE CALL
- ☐ THOUGHT
- ☐

- ☐ WITH
- ☐ ABOUT

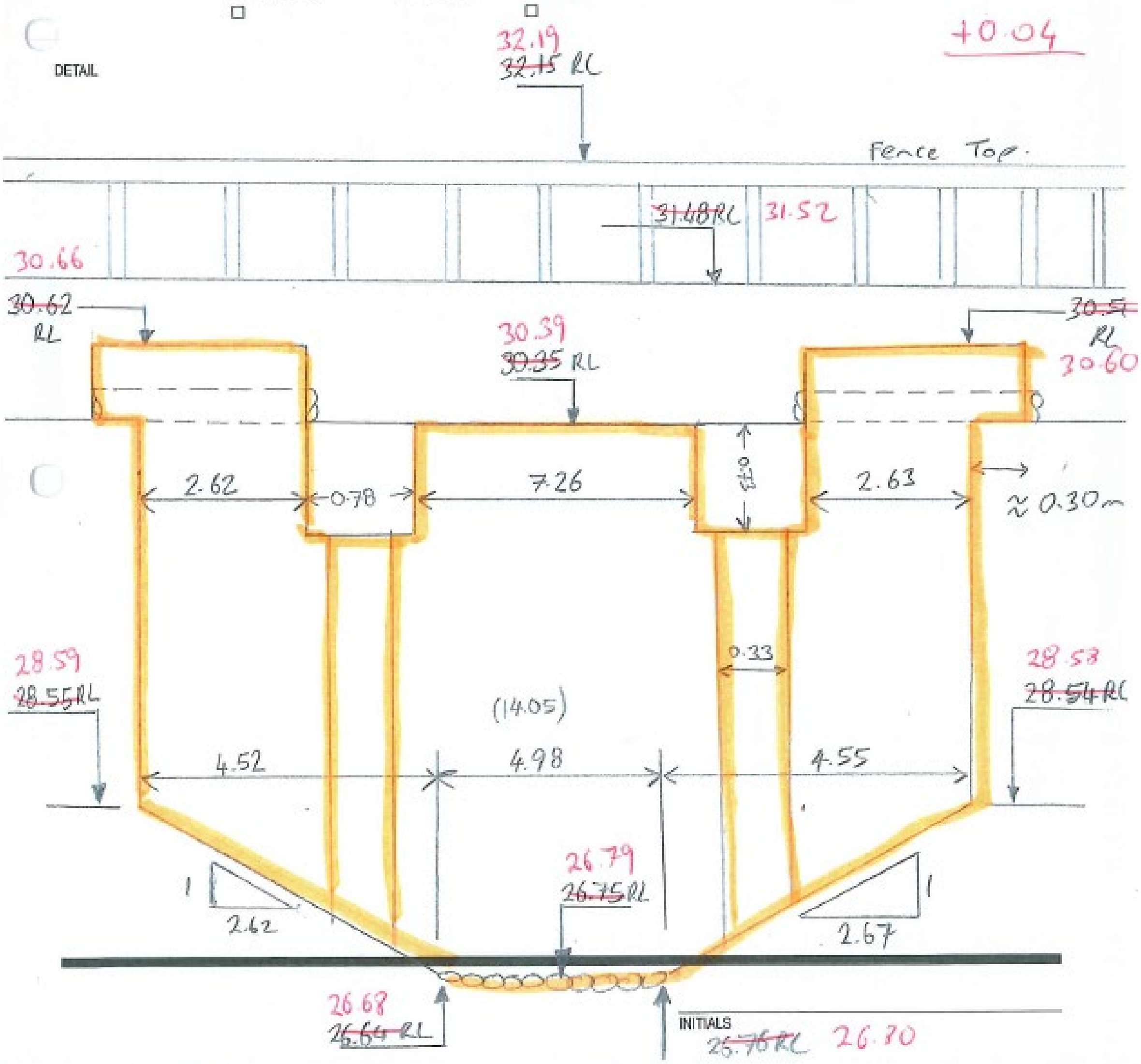
- ☐ CLIENT
- ☐ CONTRACTOR
- ☐ SUPPLIER
- ☐

BETWEEN

AND

TIME

DETAIL



INITIALS

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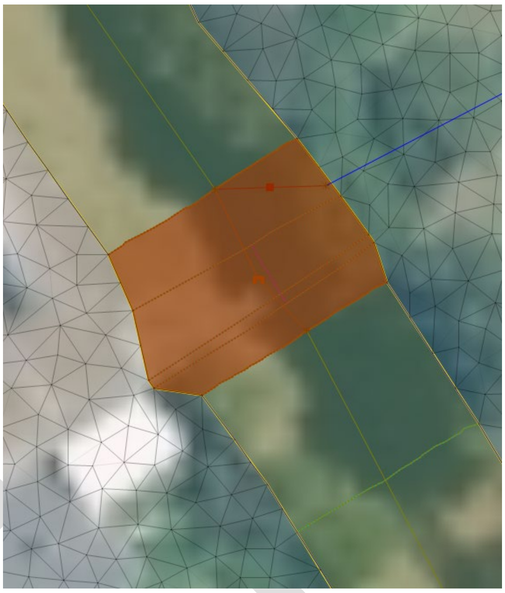

Site 4

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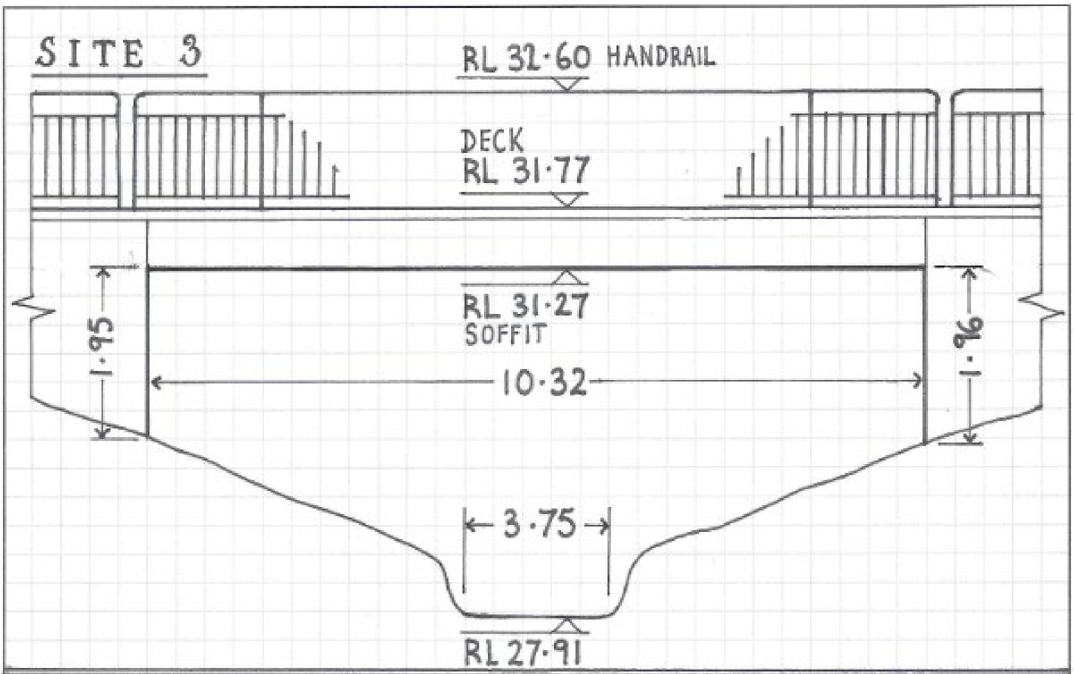
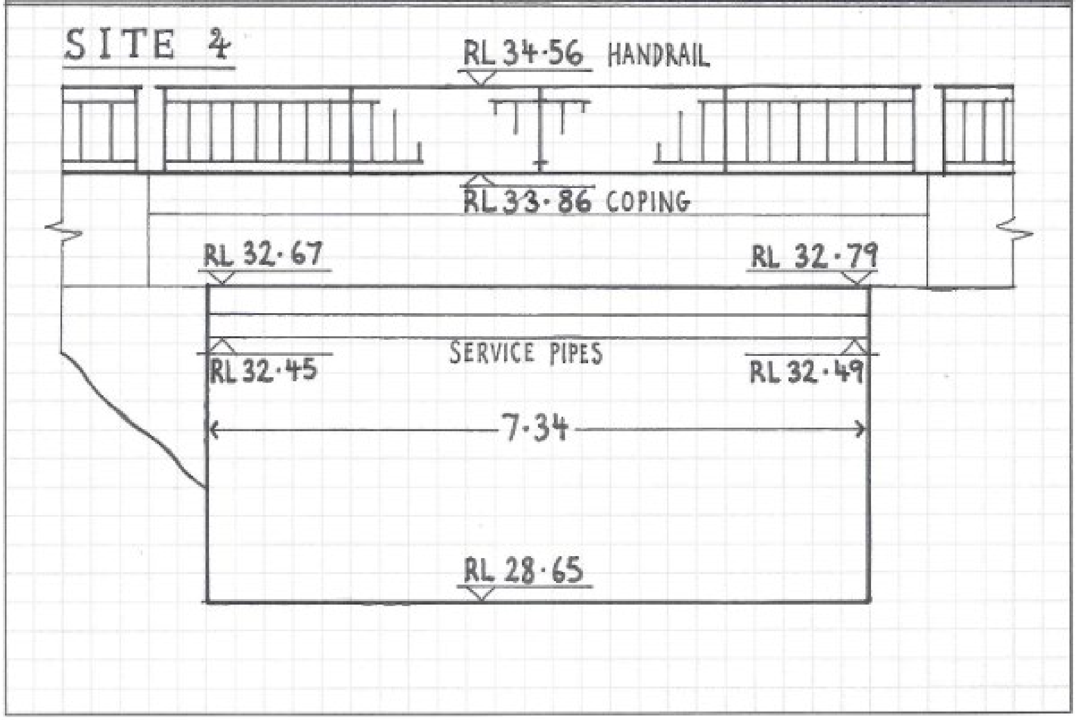
PREPARED BY CARLOS MARTIN DATE 27·4·2016
CHECKED BY GRANT MARTIN DATE 27·4·2016
REF/DWGS
SHEET OF





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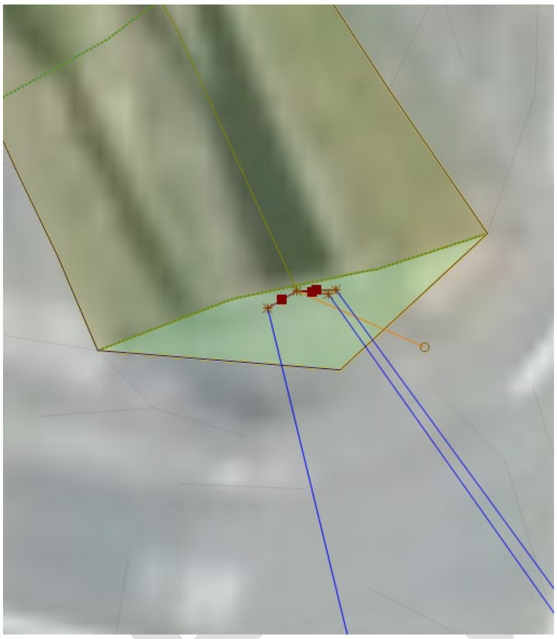

MWH
BUILDING A BETTER WORLD

PROJECT ______ PROJECT NO. 80508521 -

DESCRIPTION STOKES VALLEY WATER MODELLING.

PREPARED BY CARLOS MARTIN DATE 30 04 16

CHECKED BY GRANT MARTIN DATE 30 04 16.

REF/DWGS ______ SHEET ______ OF ______

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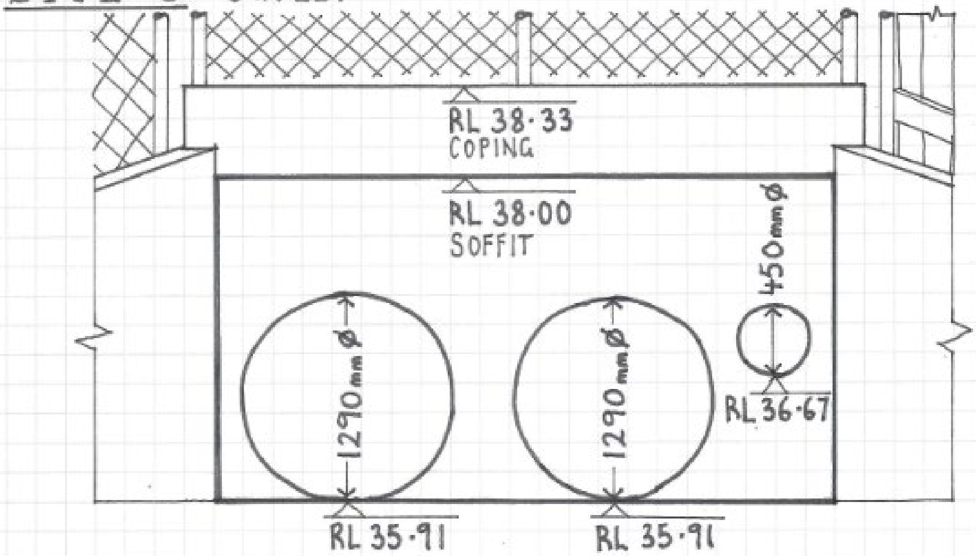

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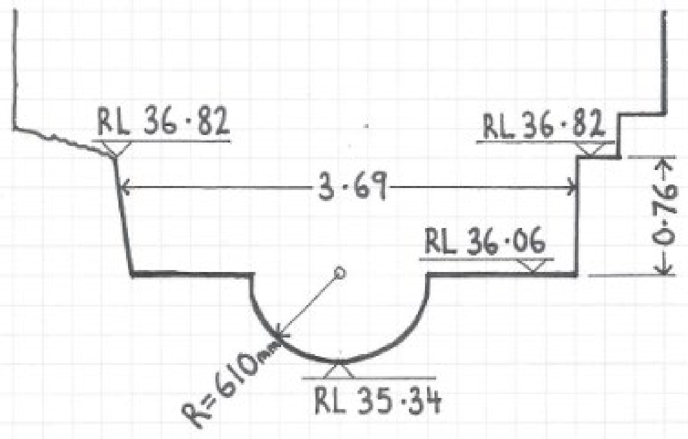

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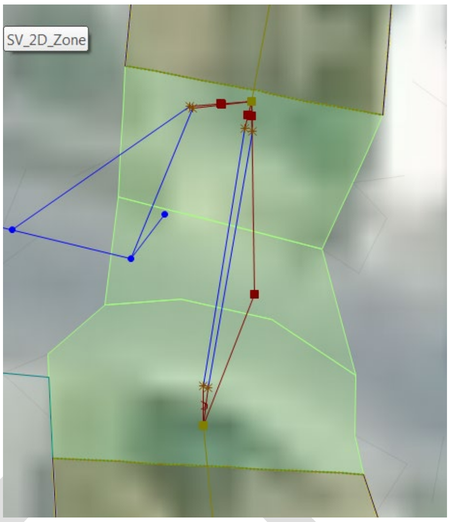

MWH

BUILDING A BETTER WORLD

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DESCRIPTION STOKES VALLEY WATER MODELLING.

PREPARED BY CARLOS MARTIN. DATE 30 04 16

CHECKED BY GRANT MARTIN DATE 30 04 16.

REF/DWGS ______ SHEET ______ OF ______

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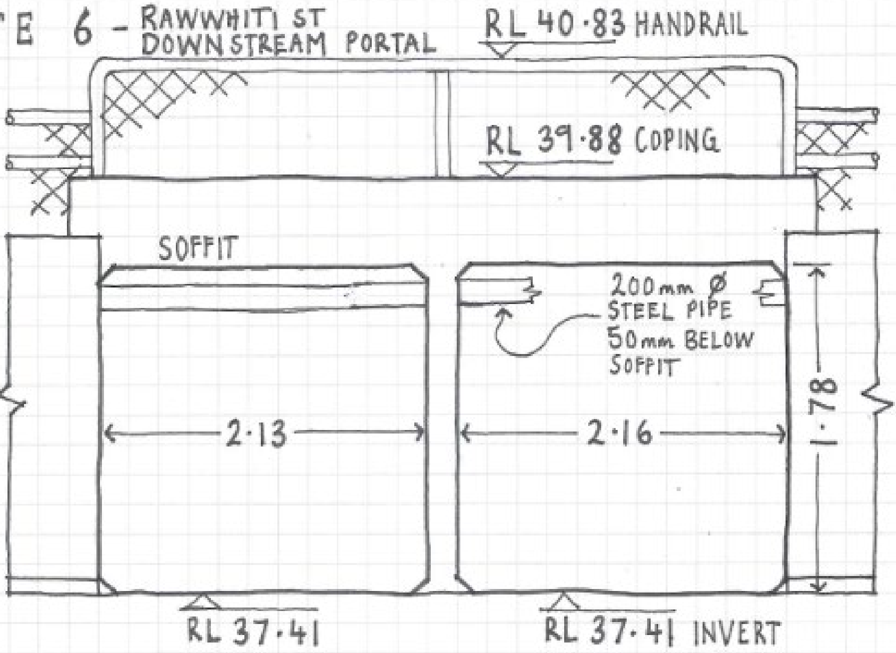
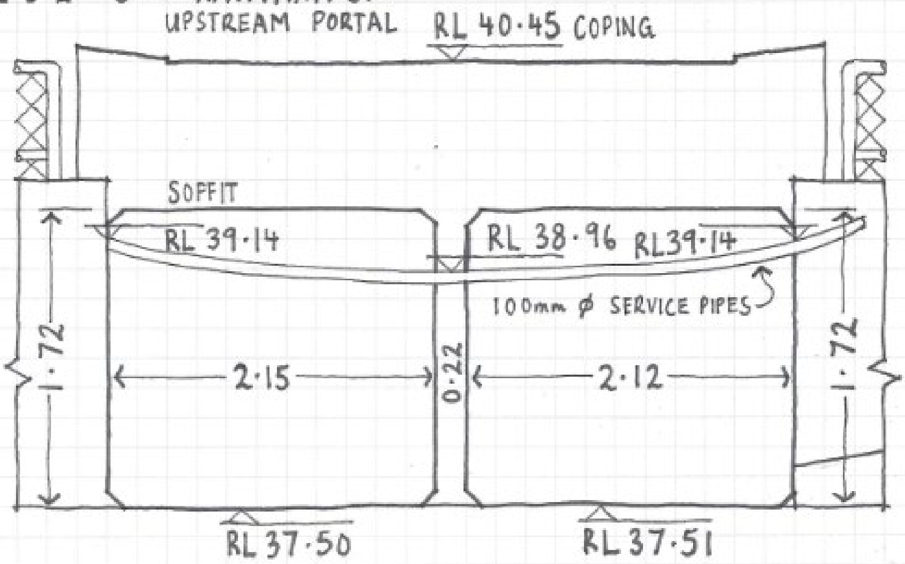


SITE 6 - RAWWHITI ST UPSTREAM PORTAL



Site 7

MWH
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PROJECT
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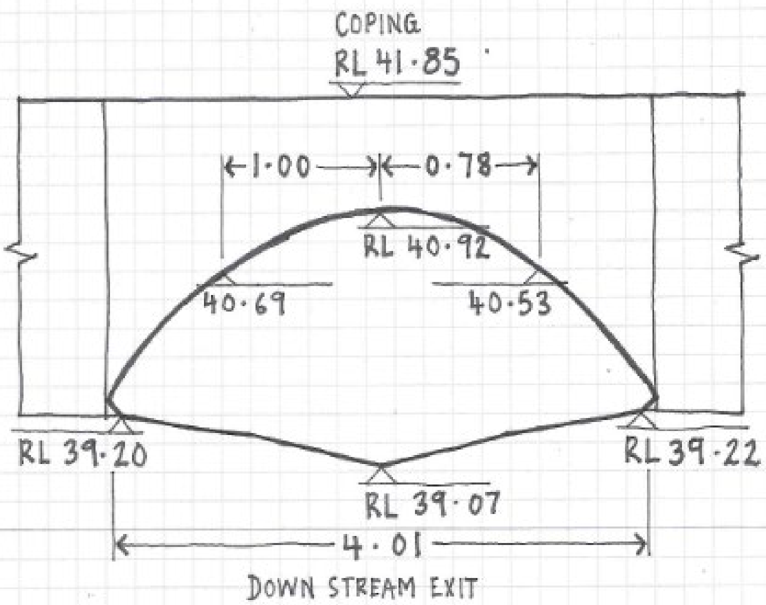
DESCRIPTION STOKES VALLEY WATER MODELLING
PREPARED BY CARLOS MARTIN DATE 30/04/16
CHECKED BY GRANT MARTIN DATE 30/04/16
REF/DWGS SHEET OF

SITE 7 - BOWERS ST EXIT CULVERT



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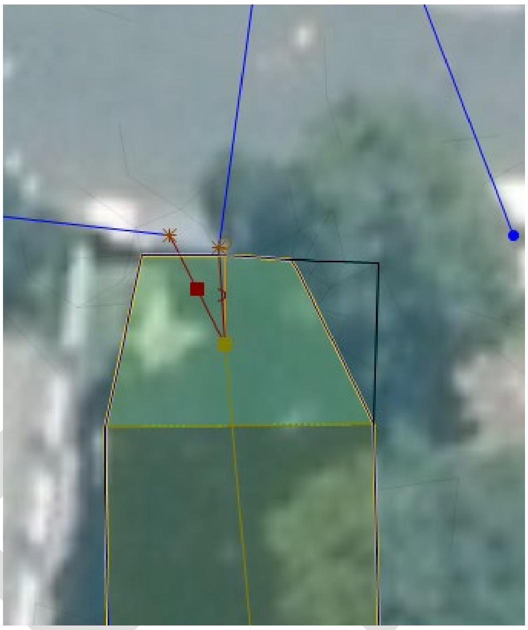

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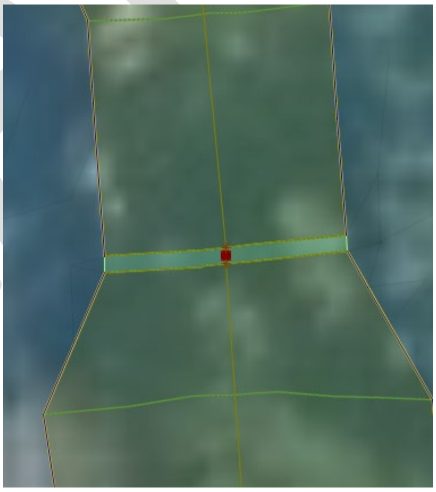


MWH

BUILDING A BETTER WORLD

PROJECT

PROJECT NO. 80508521

DESCRIPTION STOKES VALLEY WATER MODELING

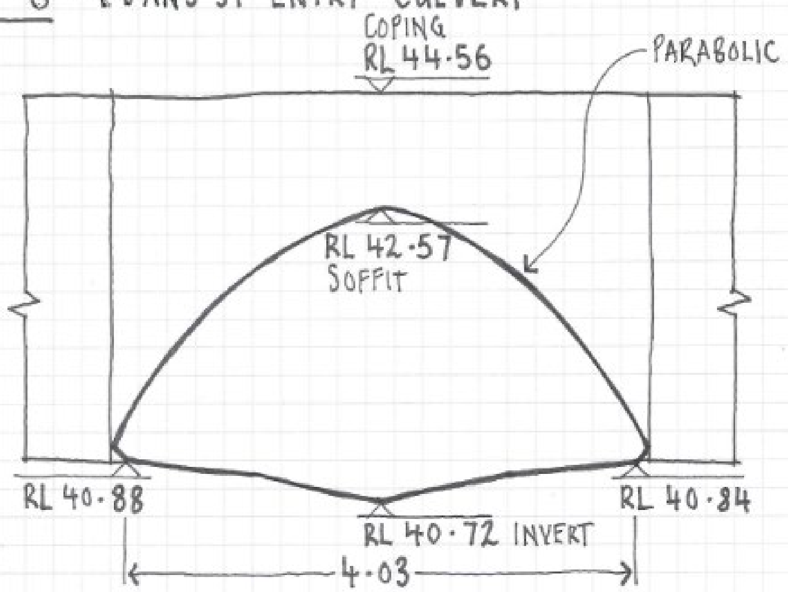
PREPARED BY CARLOS MARTIN DATE 02/05/16

CHECKED BY GRANT MARTIN DATE 02/05/16

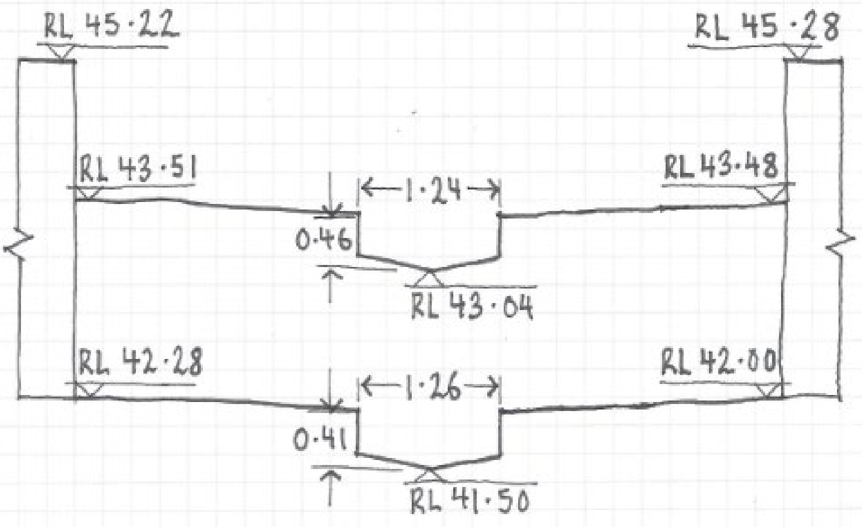
REF/DWGS

SHEET OF

SITE 8 - EVANS ST ENTRY CULVERT



SITE 9 - WIER (UPSTREAM)



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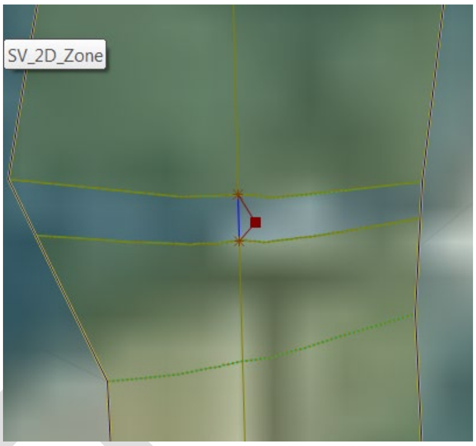

This footbridge was modelled as a unique shaped culvert due to model instability.

Site 10

MWH
BUILDING A BETTER WORLD

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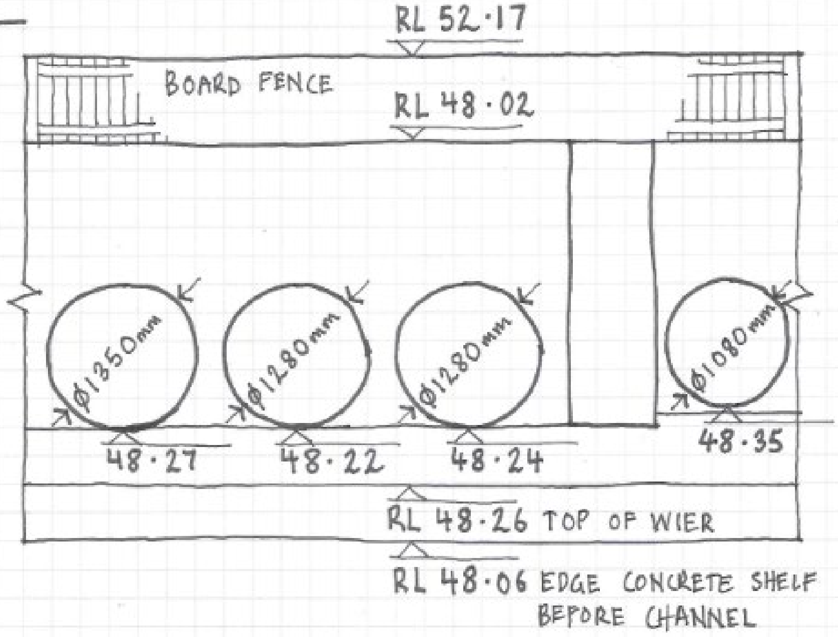
DESCRIPTION STOKES VALLEY WATER MODELING
PREPARED BY CARLOS MARTIN DATE 02/05/16
CHECKED BY GRANT MARTIN DATE 02/05/16
REF/DWGS ______ SHEET ______ OF ______

SITE - CULVERT EXITS
10



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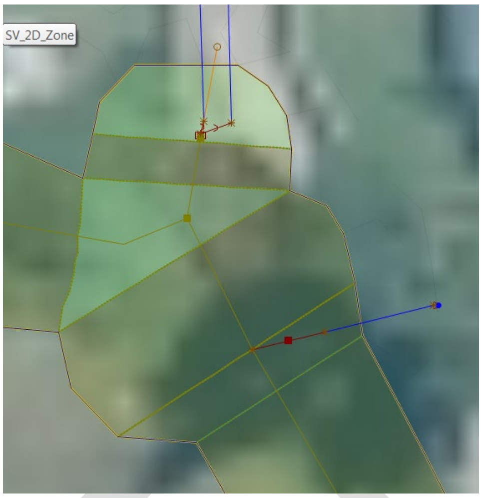

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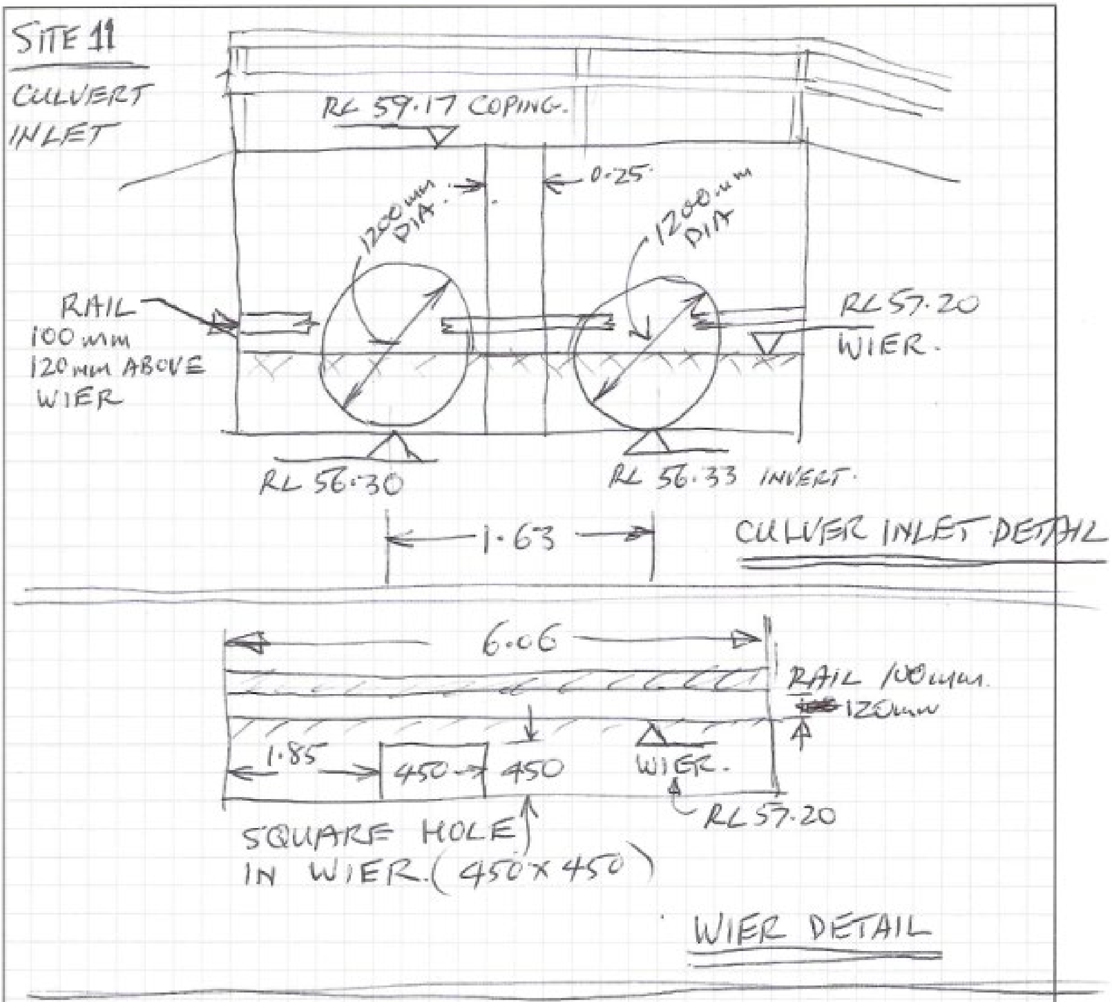
PREPARED BY GRANT MARTIN DATE 04 05 16

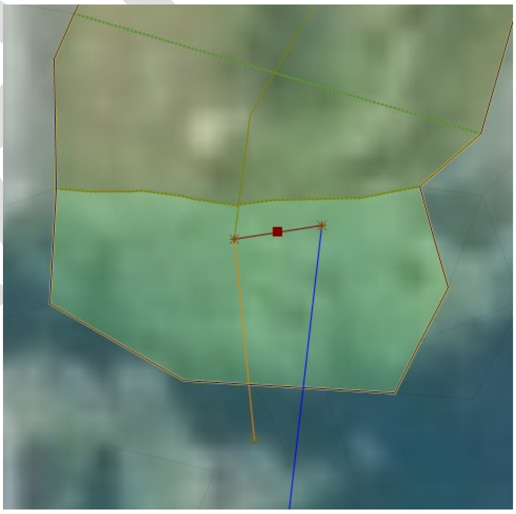
CHECKED BY [signature] DATE

REF/DWGS SHEET OF



Site 12

Site 13



MWH

BUILDING A BETTER WORLD

PROJECT

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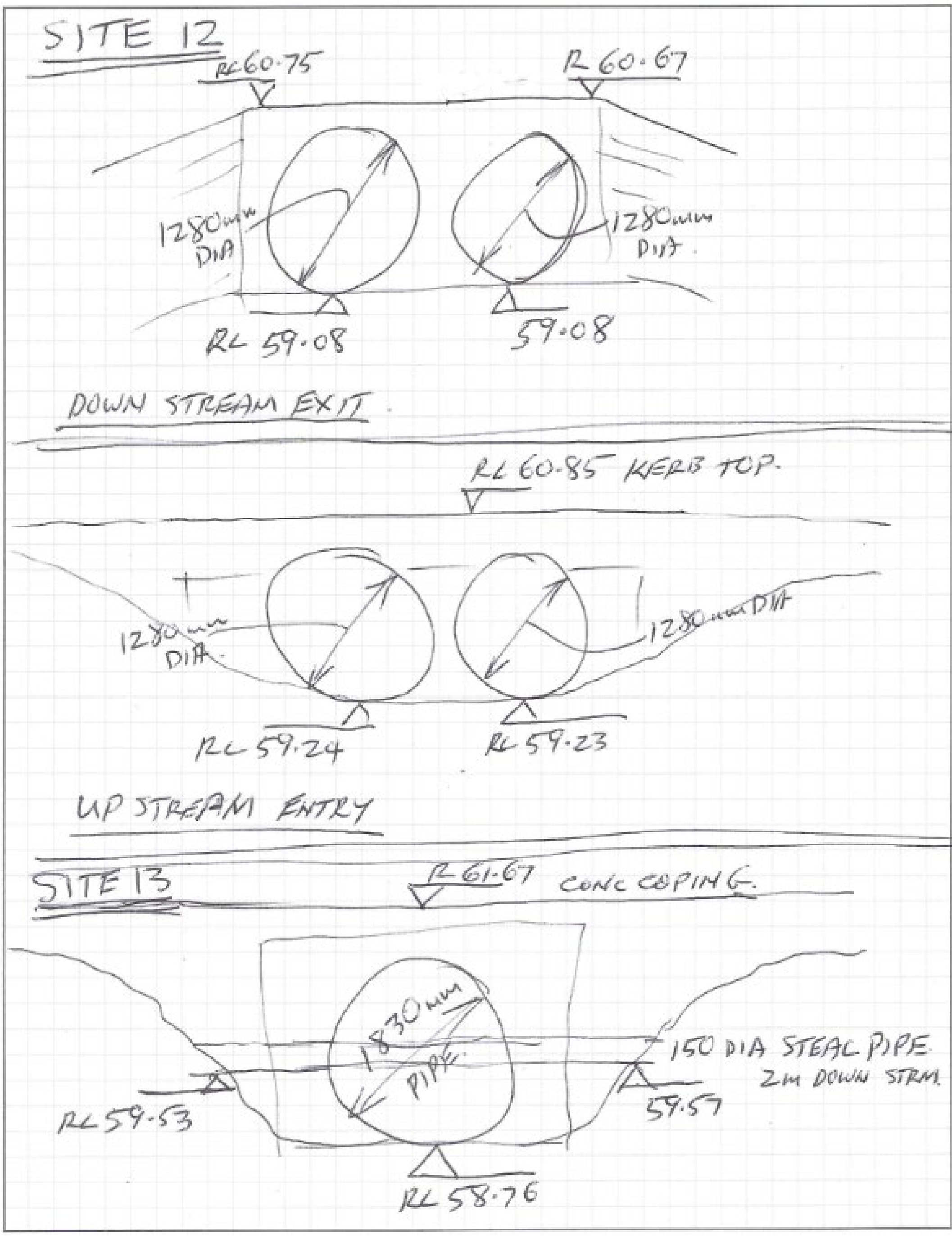
PREPARED BY GRANT MARTIN. DATE 04 05 16

CHECKED BY

DATE

REF/DWGS

SHEET OF



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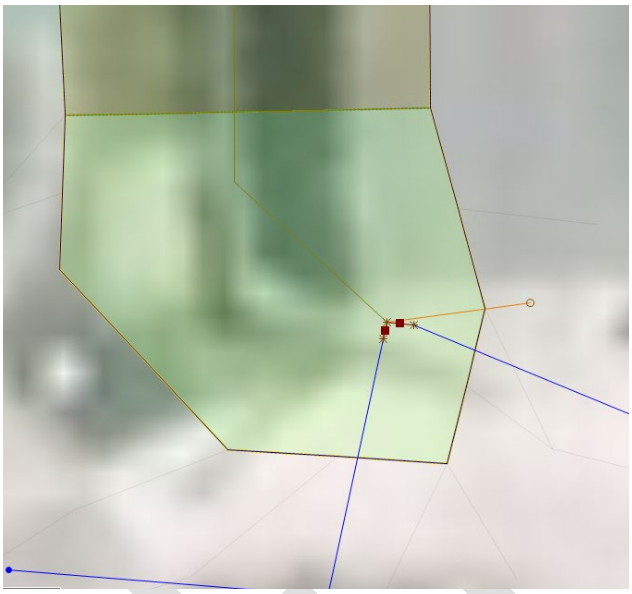

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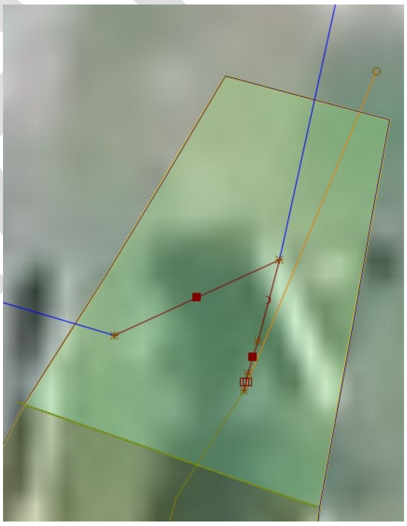

MWH
BUILDING A BETTER WORLD

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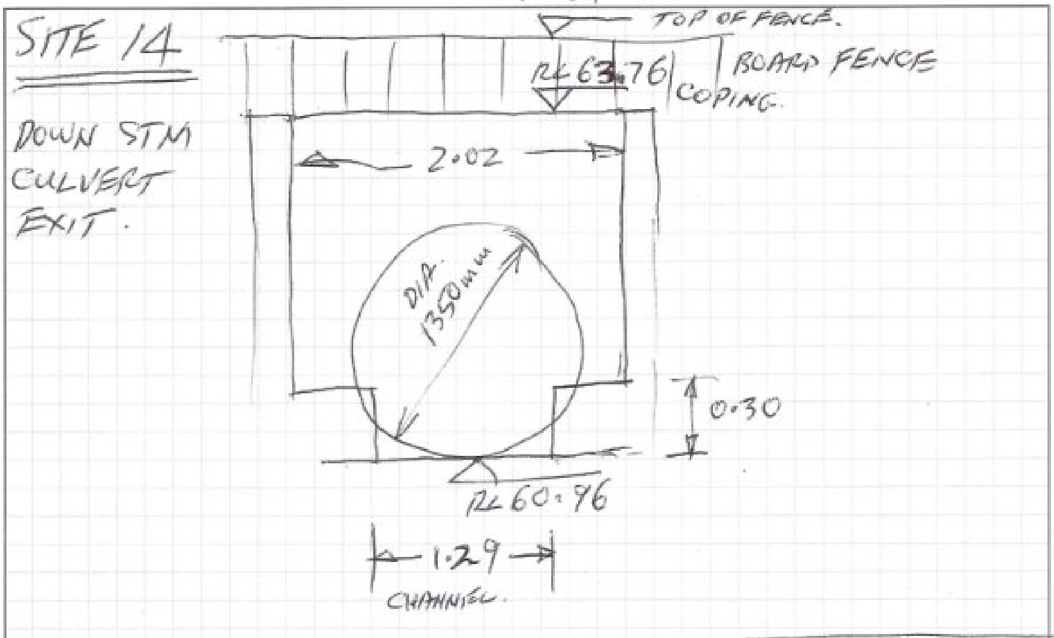
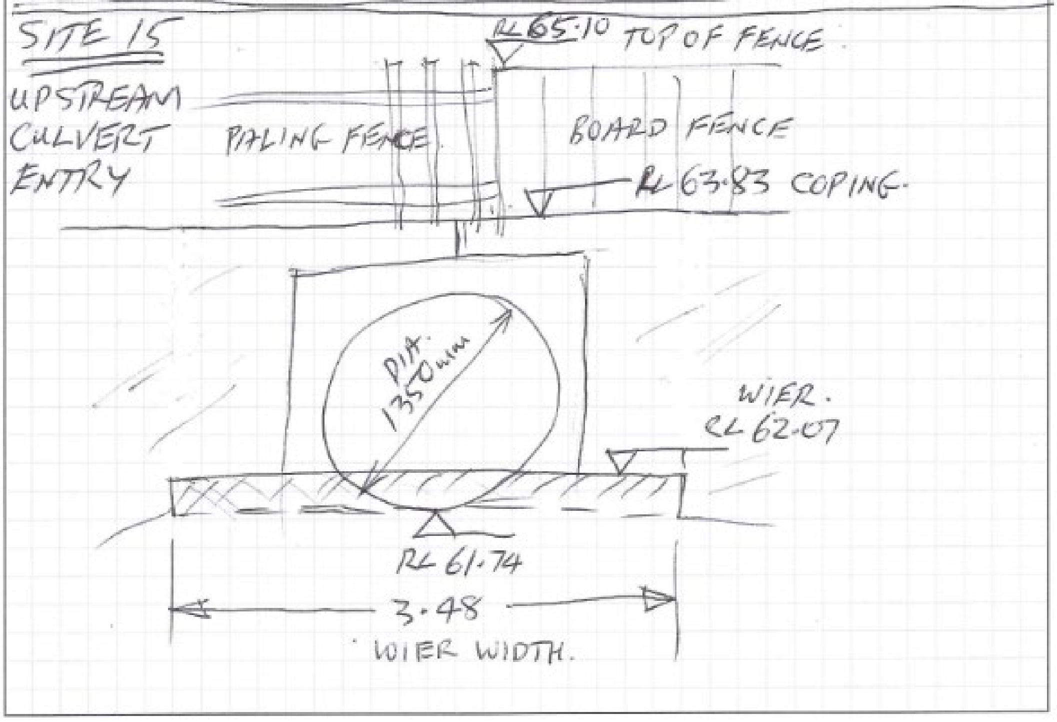
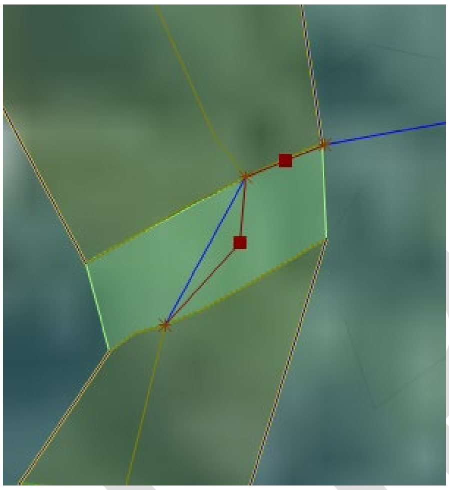




Minor site 1

Minor Site 2



MWH
BUILDING A BETTER WORLD

PROJECT
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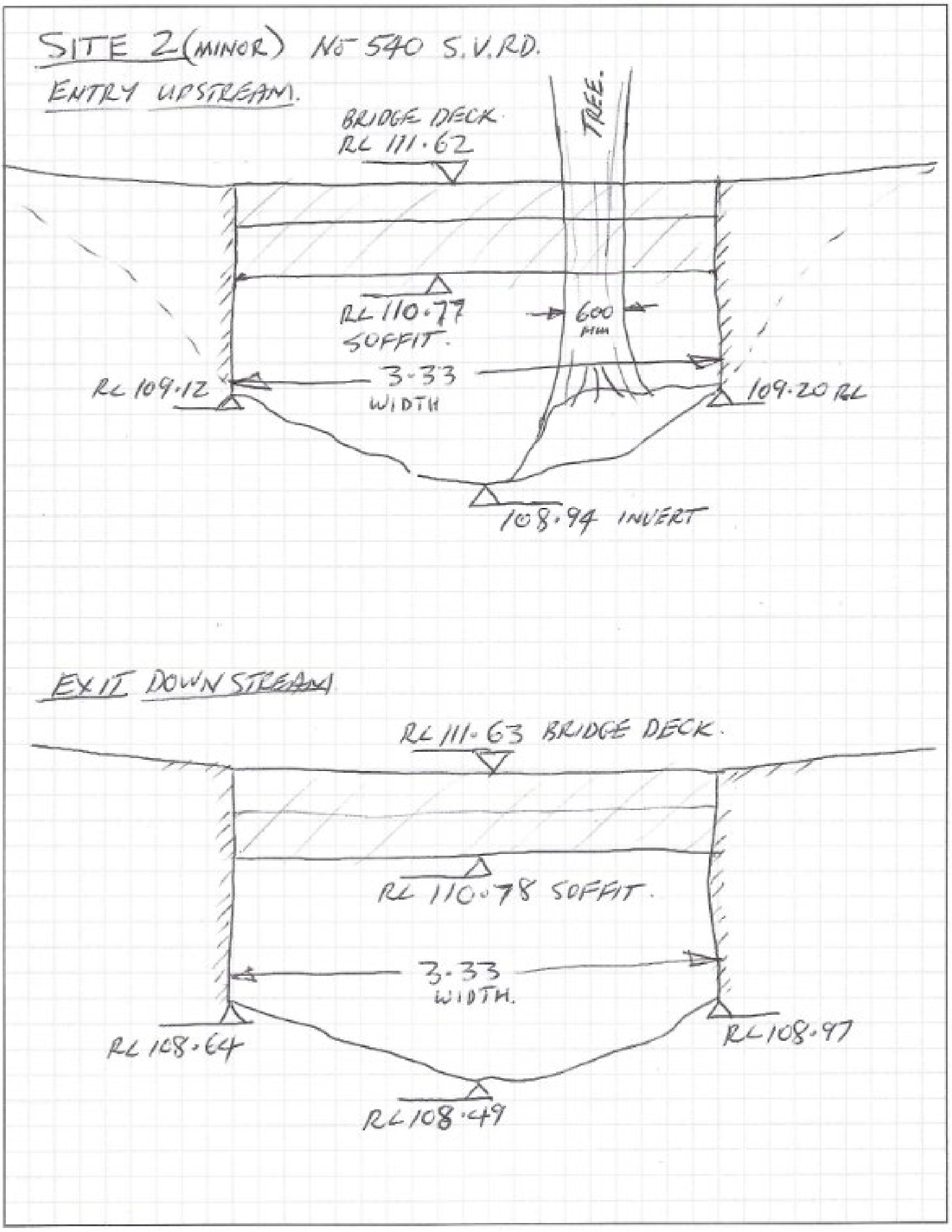
DATE 10-05-16.
CHECKED BY
DATE
REF/DWGS
SHEET
OF



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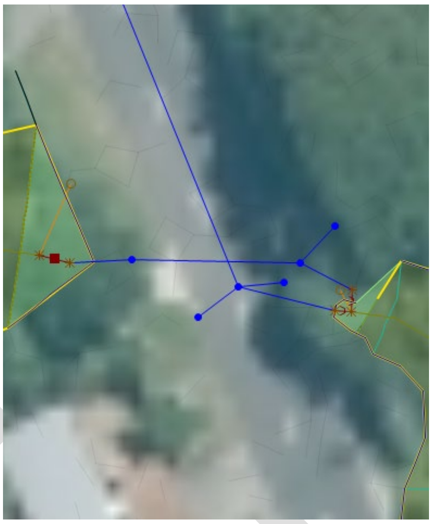
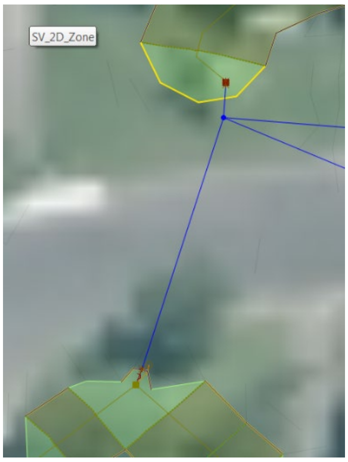
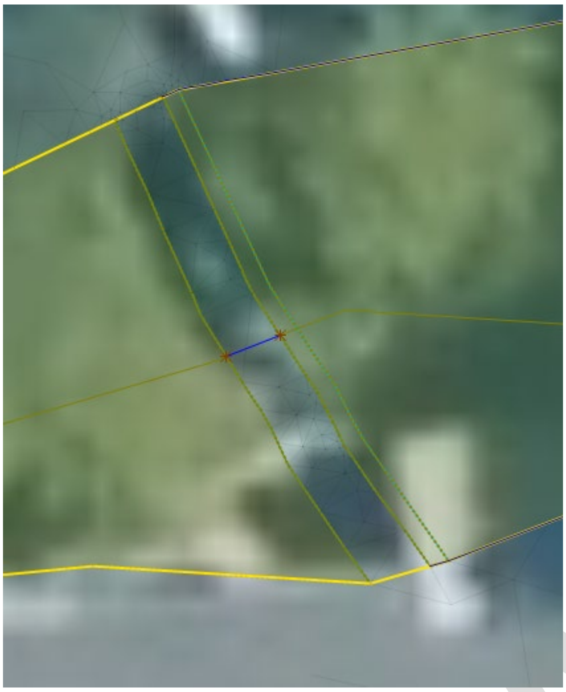


Minor site 4


Minor site 5

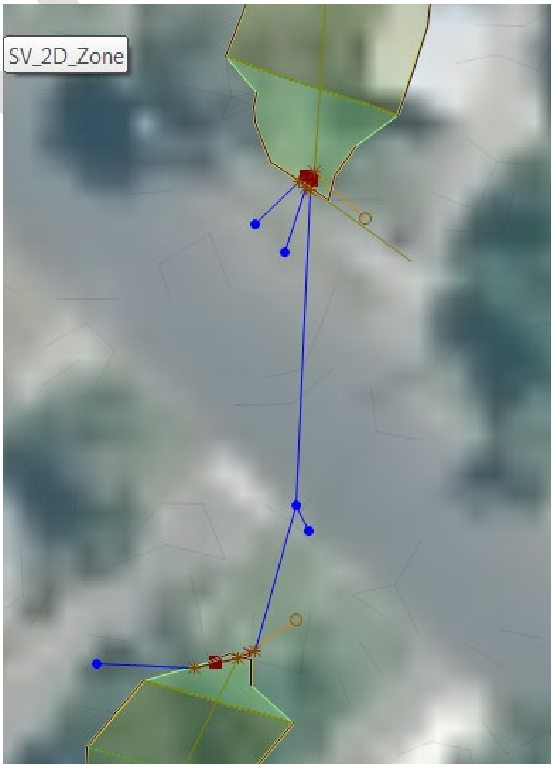



Minor Site 6



This Minor Site 6 footbridge was modelled as a unique shaped culvert due to model instability.

Minor Site 7



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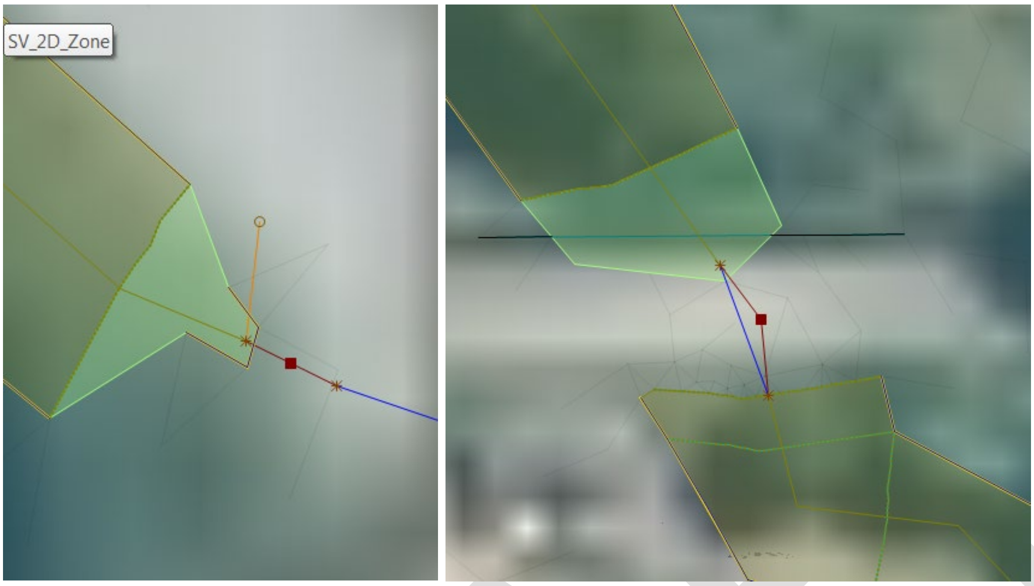
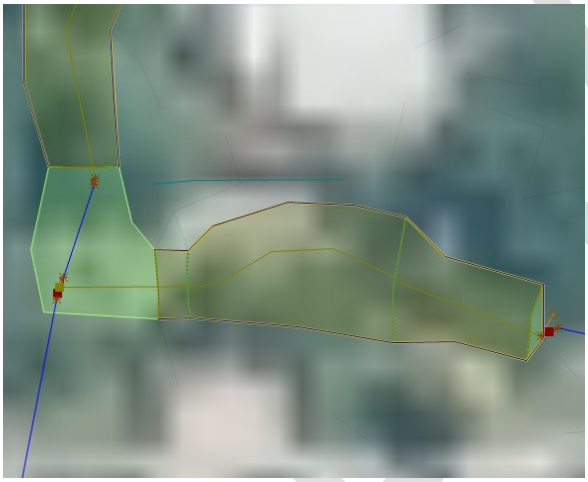
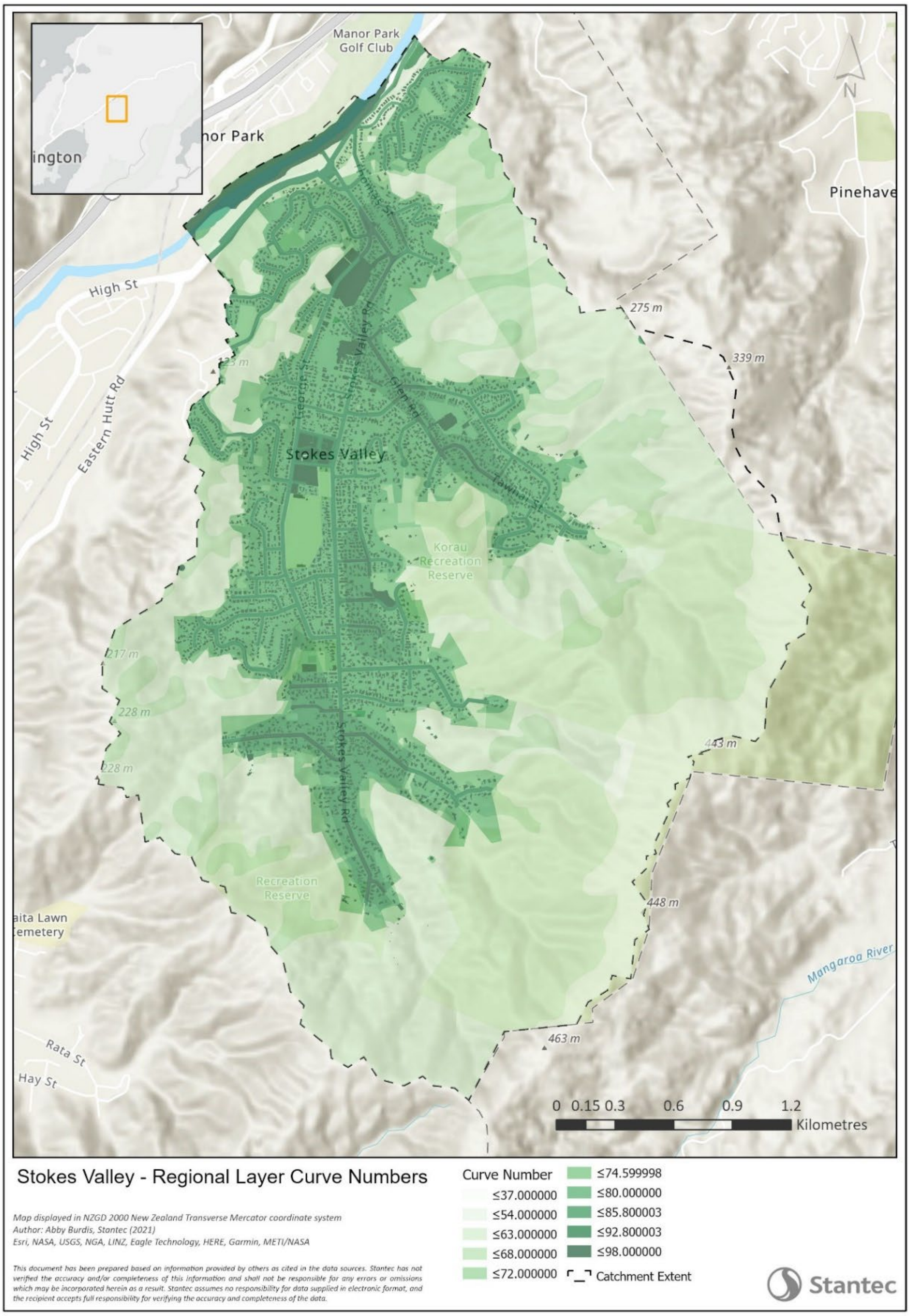
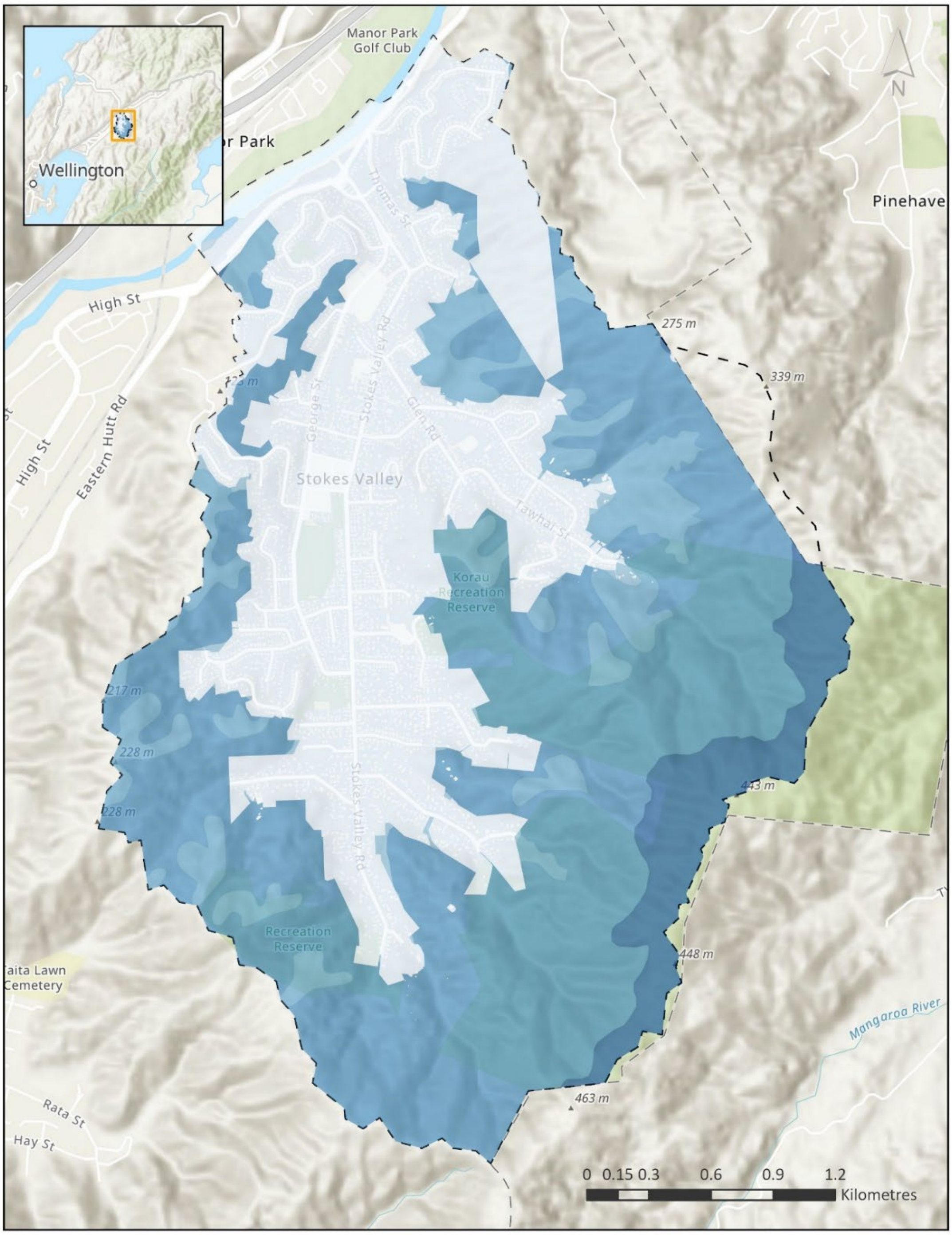


Minor site 9



Appendix B: – Regional Hydrology Layers





Stokes Valley - Regional Layer Initial Abstraction

Map displayed in NZGD 2000 New Zealand Transverse Mercator coordinate system
Author: Abby Burdis, Stantec (2021)
Esri, NASA, USGS, NGA, LINZ, Eagle Technology, HERE, Garmin, METI/NASA

This document has been prepared based on information provided by others as cited in the data sources. Stantec has not verified the accuracy and/or completeness of this information and shall not be responsible for any errors or omissions which may be incorporated herein as a result. Stantec assumes no responsibility for data supplied in electronic format, and the recipient accepts full responsibility for verifying the accuracy and completeness of the data.

Initial Abstraction (mm)
- ≤3.000000
- ≤5.000000
- ≤7.000000
- ≤9.000000
- ≤11.000000
- ≤13.000000
- ≤15.000000
- ≤20.000000
- ≤30.000000

Catchment Extent

Stantec

CREATING COMMUNITIES

Communities are fundamental. Whether around the corner or across the globe, they provide a foundation, a sense of belonging. That's why at Stantec, we always **design with community in mind**.

We care about the communities we serve—because they're our communities too. We're designers, engineers, scientists, and project managers, innovating together at the intersection of community, creativity, and client relationships. Balancing these priorities results in projects that advance the quality of life in communities across the globe.

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