Appendix E. Hydrology and Hydraulic Assessment Report

RILEY CONSULTANTS

HYDROLOGY AND HYDRAULIC ASSESSMENT TE RUAOTEHAUHAU WATER STORAGE RESERVOIR KAIKOHE

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HYDROLOGY AND HYDRAULIC ASSESSMENT
TE RUAOTEHAUHAU WATER STORAGE RESERVOIR, KAIKOHE

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1.0	Hydrology and Hydraulic Assessment	1 September 2020

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HYDROLOGY AND HYDRAULIC ASSESSMENT
TE RUAOTEHAUHAU WATER STORAGE RESERVOIR, KAIKOHE

1.0 Introduction

This preliminary hydrology and hydraulic assessment has been prepared by Riley Consultants Ltd (RILEY), at the request of Te Tai Tokerau Water Trust. This report details the assessment and is intended to support a resource consent application for the construction a large dam.

The scope of the assessment was as follows:

- Estimation of inflow hydrographs for a range of design events in general accordance with the New Zealand Dam Safety Guidelines (New Zealand Society on Large Dams (NZSOLD), 2015) (NZSOLD Guidelines).
- A sunny day potential impact classification (PIC) assessment in general accordance with the NZSOLD Guidelines.
- Preliminary design of the spillway arrangement to provide adequate protection to the dam during the design flood event.
- Preliminary design of the temporary flood diversion works during construction.

2.0 Background

The proposed Te Ruaotehauhau water storage reservoir is located on the Pekapeka Stream immediately downstream of the confluence of the Waitaia Stream and the Te Ruaotehauhau Stream. The Pekapeka Stream passes to the west of Ohaeawai. Rivers further downstream include the Waiaruhe River and the Waitangi River, which discharges to the estuary at Haruru. The dam location, relative to other identifying features, is presented on RILEY Dwg: 200249/3-200. The site was previously referred to as MN06.

Photo 1: Looking upstream from the right abutment. The confluence of the Waitaia Stream and the Te Ruaotehauhau Stream is visible to the left side of the photo.

Photo 2: Looking upstream from a culvert crossing approximately 500m downstream of the proposed dam site

3.0 Downstream Effects and Potential Impact Classification

A PIC assessment considers the consequences of an uncontrolled release of the reservoirs' contents as a result of a dam breach. PIC assessments are independent of the likelihood of a failure, which, for a suitably designed, constructed, and operated dam, should be very low.

A comprehensive PIC assessment involves determining dam breach characteristics, and hydraulic modelling downstream of the dam.

Module 2 of the NZSOLD Dam Safety Guidelines (2015) outlines the consequence assessment and dam classification framework adopted in New Zealand. It considers three principal components, being:

1. Damage level.
2. Population at risk.
3. Potential loss of life.

Dams are categorised as low, medium, or high PIC based on these components.

The NZSOLD Guidelines provide design criteria, construction, and operation requirements for each PIC, with a high PIC dam having the highest criteria. Such a classification system ensures the dam performance requirements are appropriate for the hazard posed by the reservoir.

4.0 Dam Breach Hydraulic Assessment

4.1 Hydraulic Methodology

We have used HEC-RAS (v5.07) to simulate a breach of the dam. The full momentum equation set has been used.

4.2 Terrain

A 5m Digital Elevation Model (DEM) was sourced from Northland Regional Council (NRC). We understand that the DEM was created from a Light Detection and Ranging (LiDAR) survey undertaken in 2017. The DEM covers the full catchment area to the proposed dam and extends downstream to the Waiaruhe River and the Waitangi River confluence. The vertical datum and horizontal projections used are NZVD 2016 and NZTM 2000, respectively. We have used the same vertical datum and horizontal projections within this assessment. We understand that site specific survey information is not available at this time. RILEY did not make any modifications to the terrain.

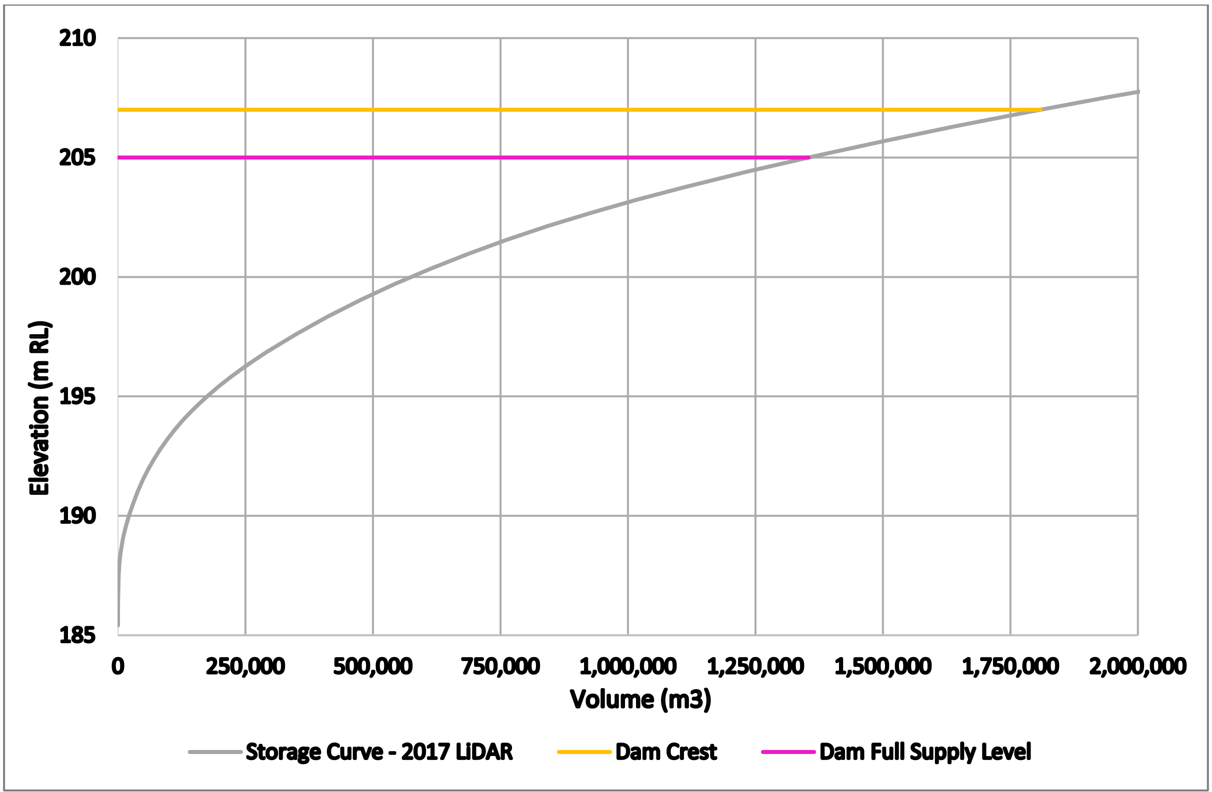
4.3 Breach Scenarios

For the purposes of this preliminary design we have assessed a sunny day piping scenario. A rainy-day scenario will also need to be considered during detailed design.

4.4 Geometry

The reservoir has been modelled as a storage area. The elevation-storage relationship (derived from the storage area extent within the HEC-RAS model) is presented within Figure 1. The storage volume at the full supply level is approximately 1.35million m^3.

Figure 1: Elevation Storage Relationship



The area downstream of the dam has been modelled as a 2D flow area, with a 5m by 5m grid, and a global Manning's 'n' value of 0.06. The 2D flow area extends to downstream of the State Highway 1 (SH1) crossing over the Waiaruhe River.

The dam has been modelled as a connection between the reservoir storage area and the downstream 2D flow area. The dam has a proposed full supply level of RL 205m with an interim crest elevation of RL 207m. The downstream dam toe will have an elevation of approximately RL 185.3m.

4.5 Breach Parameters

The main parameters used to derive the breach parameters are presented within Table 1.

Table 1: Input Parameters for Breach Parameter Estimation

Parameter	Value	Source
Dam Toe Elevation (m RL)	185.3	LiDAR
Service Spillway Crest (m RL)	205	Design Value
Dam Crest (m RL)	207	Interim Design Value
Retained Volume Service Spillway Crest (m^3)	1,354,000	LiDAR (conservative)
Final Breach Invert Level (m RL)	185.3	Slightly above downstream terrain
Height of water above breach invert (m)	19.70	Breach invert subtracted from spillway crest
Average embankment width (m)	220	LiDAR
Approach flow width (m)	220	LiDAR

Figure 2: Breach Profile

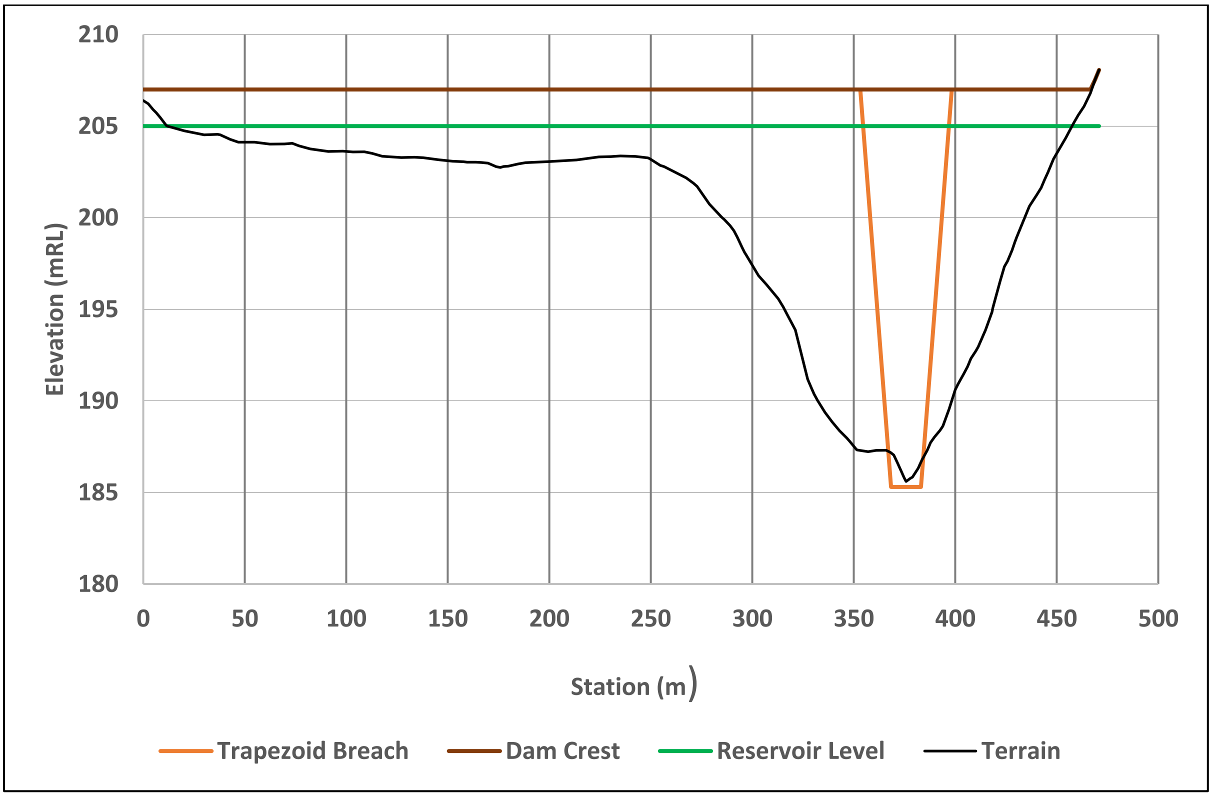


Table 2 presents the dam breach parameters calculated using the methods outlined in Wahl 1998. Froehlich (1995) is the most recent method for estimating dam breach parameters (within Wahl 1998) and it uses the largest number of case studies in the development of its empirical equations. The Froehlich (2016) method has been developed in the time since the NZSOLD Guidelines were published.

We, therefore, gave greater weighting to the Froehlich 2016 method. Full details are provided within the appended calculations.

Table 2: Dam Breach Parameters

Method	Average Breach Width, B (m) [1]	Formation Time, t_f (minutes)	Z (H:V)
Johnson and Illes (1976)	10.9 – 65.1	n/a	n/a
Singh and Snorrason (1982, 1984)	43.4 – 108.5	15 – 60[2]	n/a
MacDonald and Langridge-Monopolis (1984)	n/a	34.2	n/a
FERC (1987)	43.7 – 86.8	6 – 60[2]	0.25 – 1[2]
USBR (1988)	59.1	39	n/a
Froehlich (1995) - Piping	45.5	19.8	1.4
Froehlich (2016) - Piping	29.1	18.6	0.7

Notes:
1 Range shown if applicable
2 Range provided by method without any calculation

The larger the dam breach width (B) and shorter the formation time (t_f), the larger the peak outflow will be. The side slope of the breach shape is of secondary importance.

HEC-RAS uses a bottom breach width, not the average breach width (as derived using the Froehlich methods). We have used a bottom breach width of 14.7m for the piping breach scenario (with an average breach width of 30m and side slopes of 0.7). A cross section of the breach profile is presented in Figure 2.

Table 3: Breach Parameters

Parameter	RILEY
Breach Bottom Width (m)	14.7
Breach Bottom Elevation (m RL)	185.3
Left Side Slope (H):(V)	0.7:1
Right Side Slope (H):(V)	0.7:1
Formation Time (minutes)	18

4.6 Downstream Boundary Condition

A normal depth boundary condition (friction slope = 0.00507) has been used at the downstream boundary, located approximately 500m downstream from the SH1 bridge at the Waiaruhe River. We consider that the assumed downstream boundary condition is unlikely to affect the model results at the location of interest.

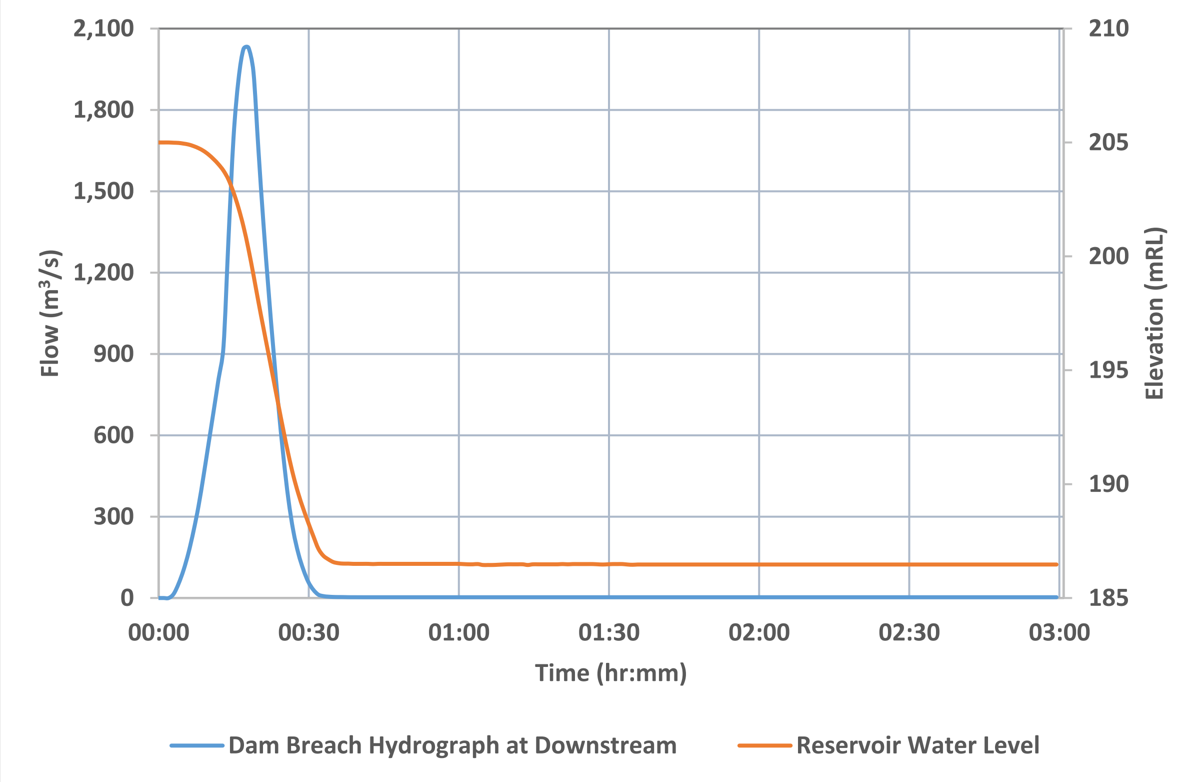
4.7 Initial Condition

We have used an initial condition of RL 205m for the reservoir storage area.

4.8 Results

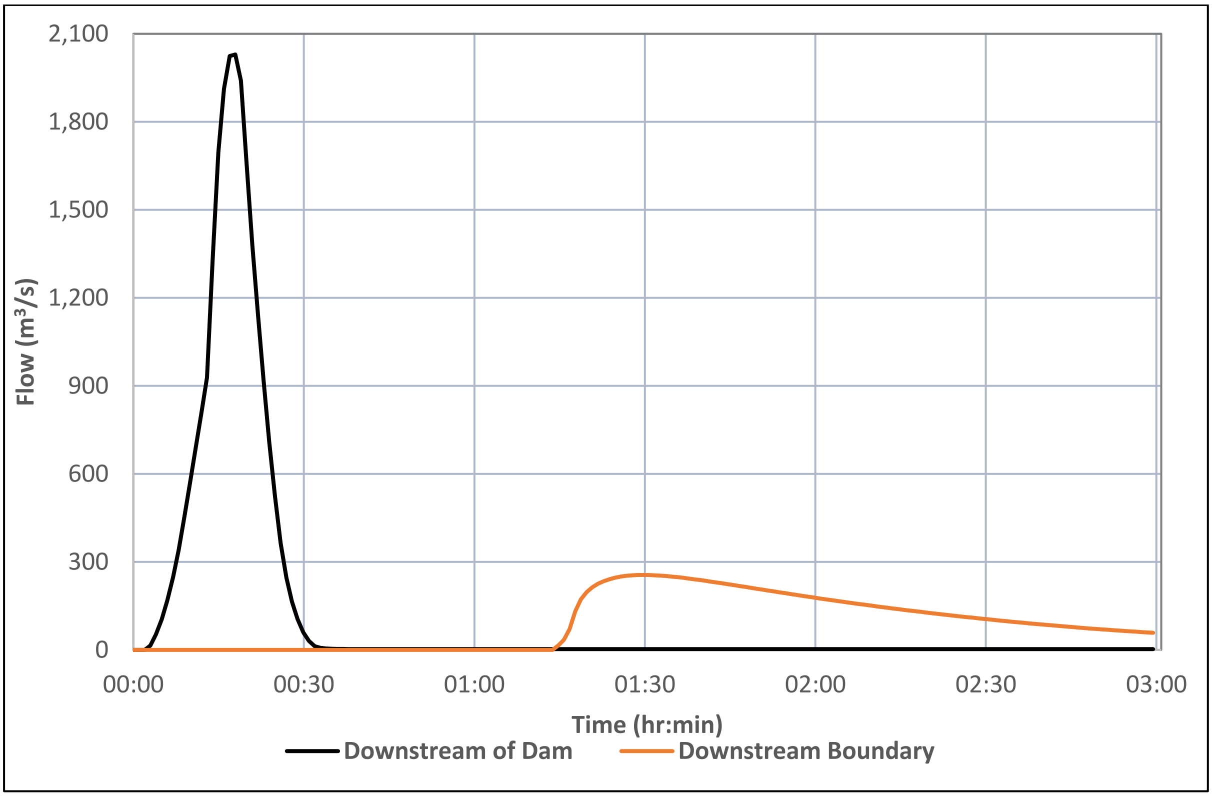
Figure 3 presents the reservoir level and outflow hydrograph immediately downstream of the dam.

Figure 3: Dam Breach Hydrograph and Reservoir Water Level



The resultant breach hydrograph at the dam site along with the flow hydrograph at downstream boundary is presented within Figure 4. The figure demonstrates that the peak discharge from the dam is approximately 2,030m^3/s. The peak flow at the downstream boundary is 255m^3/s indicating significant attenuation of breach flow. We note that dam breach overtops SH1, to the north of the intersection with State Highway 12 (SH12). Once overtopping occurs at this point, the flow enters a neighbouring catchment (Titahi Stream).

Figure 4: Dam Breach and Downstream Boundary Hydrographs



For comparison, the predicted peak breach outflows by the Froehlich methods are presented in Table 4. Overall, there is a significant range in results. The predicted flow from HEC-RAS is about 28% higher than the Froehlich (1995) estimate and is approximately twice the flow derived by formulations of Froehlich (2016).

Table 4: Comparison of Peak Breach Outflows

Method	Peak Outflow (m^3/s)
Froehlich (1995)	1,575
Froehlich (2016) – Empirical	869
Froehlich (2016) – Semi-theoretical	960
HEC-RAS Model	2,030

Froehlich 2016 also presents 42 dams that have breached, which have measured peak outflows. The four dams that are most similar in reservoir volume and breach height to the proposed dam are presented in Table 5. Based on this comparison, it would appear likely that the potential peak flow at Te Ruaotehauhau Water Storage Reservoir would be greater than $1{,}050m^3/s$ and less than $2{,}370m^3/s$.

Table 5: Breach Flow Comparison

Dam Name and Location	Volume (million m^3)	Height of Water Above Breach (m)	Peak Outflow (m^3/s)
Te Ruaotehauhau Water Storage Reservoir	**1.35**	**19.7**	**2,030**
Bradfield (Dale Dyke), England	3.2	28.0	2,370
Lake Avalon, New Mexico	31.5	13.7	2,320
Little Dear Creak, Utah	1.36	22.9	1,330
Laurel Run, Pennsylvania	0.555	14.1	1,050

Overall, the HEC-RAS predicted peak flow of 2,030m^3/s appears conservative (perhaps at the upper bound), and we consider the derived hydrograph is appropriate to be used for the PIC assessment. We note that a hydraulic sensitivity analysis has not been undertaken.

4.9 Drawings

The drawings within Appendix A and summarised in Table 6, present the model results.

Table 6: Drawing Summary

Drawing Number	Drawing Name
200240/3-200	Downstream Floodplain Overview
200240/3-201 to -202	Sunny Day Breach - Peak Levels (Areas 1 and 2)
200240/3-203 to -204	Sunny Day Breach - Peak Depth (Areas 1 and 2)
200240/3-205 to -206	Sunny Day Breach - Peak Depth Velocity Product (Areas 1 and 2)

5.0 Damage Level Assessment

5.1 General

The damage level assessment requires the assessment of individual specified categories, as outlined in the following sections. The damage level is taken as the highest damage level from each of the categories. The damage levels from lowest to highest damage are minimal, moderate, major, and catastrophic.

5.2 Residential Houses

The NZSOLD Guidelines define destroyed as rendered uninhabitable but does not define uninhabitable. We note that the NZSOLD Guidelines make references to the following publications with regards to damage to residential houses:

- RESCDAM (2010) – includes test data on the performance of buildings in flowing water as a function of building type, flood depth, and velocity.
- National Institute of Weather and Atmosphere (NIWA, 2010) – provides potential damage curves as a function of building type and flood depth, based on observed data from floods and tsunamis in New Zealand.

NIWA (2010) provides a graph (Figure 5), that presents curves for the damage threshold and the total destruction threshold of timber/weatherboard buildings, based on the depth and velocity of flood waters. The figure indicates that at flood depths less than 3m, velocity damage occurs when the product of depth and velocity (D x V) is 1.5m^2/s and total destruction occurs when D x V is greater than 3m^2/s, as shown in Table 7.

Figure 5: Inundation Depth and Velocity Thresholds for: (a) Onset of Damage due to Water Velocity; and (b) Total Destruction, of Timber/Weatherboard Buildings (NIWA, 2010).

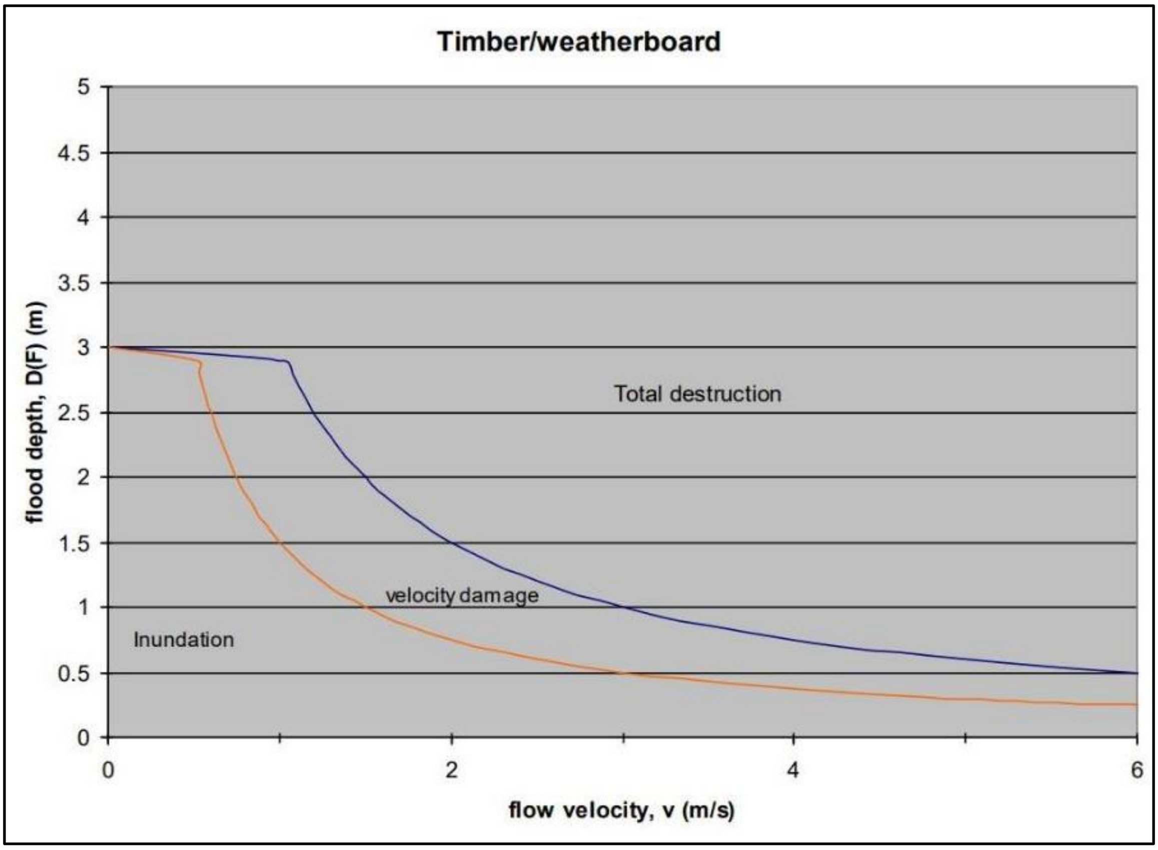


Table 7: Depths and Velocity Points from Curves Presented in Figure 5

Scenario	Depth (m)	Velocity (m/s)	D x V
Velocity Damage Threshold (orange line)	1.5	1.0	1.5
	1.0	1.5	1.5
	0.5	3.0	1.5
Total Destruction Threshold (blue line)	2.0	1.5	3.0
	1.5	2.0	3.0
	1.0	3.0	3.0

An alternative conservative approach is to consider the number of houses that are surrounded by greater than 0.5m of water (above surrounding ground levels). Such inundation could render a house uninhabitable (and therefore destroyed) due to static water damage.

We have used the latest building outline information from Land Information New Zealand (LINZ) to assess the number of residential houses affected. We have made our best judgment on whether buildings are residential in nature (i.e. habitable). Some are difficult to assess from aerial imagery and therefore we have provided a range of affected houses.

Affected houses are highlighted on the drawings. The residential houses affected are all located within Ohaeawai. There are 47 residential houses within Ohaeawai village have been identified to be affected by depths greater than 0.5m. We note that there are no houses located below the SH12 road embankment crest level.

No affected residential houses have been identified downstream of Ohaeawai through to the SH1 bridge.

Table 8 presents a summary of the residential house assessment. Based on this, we consider that nine houses are likely to be destroyed, with up to nine other houses damaged by velocities to some extent. We consider that a major damage level is appropriate for the residential houses, as highlighted within Table 9.

Table 8: Residential House Summary

Scenario	Depth > 0.5m	1.5 m^2/s <D x V<3.0 m^2/s	D x V >3.0m^2/s
Sunny Day Piping	36 - 47	6 - 9	7 - 9

Table 9: Residential Houses Damage Level

Damage Level	Residential Houses
Catastrophic	More than 50 houses destroyed.
Major	Four to 49 houses destroyed, and a number of houses damaged.
Moderate	One to three houses destroyed and some damaged.
Minimal	Minor damage.

5.3 Critical or Major Infrastructure

The NZSOLD Guidelines state that critical or major infrastructure includes:

a. Lifelines (power supply, water supply, gas supply, transportations systems, wastewater treatment, telecommunications (network mains and nodes rather than local connections)); and

b. Emergency facilities - (hospitals, police, fire services); and

c. Large industrial, commercial, or community facilities, the loss of which would have a significant impact on the community; and

d. The dam, if the service the dam provides is critical to the community and that service cannot be provided by alternative means.

Table 10 presents the critical or major infrastructure we have identified downstream of the dam, via a review of aerial photography. We do not consider that the proposed dam meets the definition of critical or major infrastructure.

Table 10: Critical or Major Infrastructure Identified Downstream of Dam

Infrastructure	Comment
State Highway 12 Culvert/Road Embankment	Likely to be damaged due to significant overtopping of road.
State Highway 1 Bridge (Waiaruhe River)	Some erosion damage likely at the abutments with a peak flow of approximately 250m3/s, although the bridge deck and beams appear likely to remain above the peak water level.

Based on the assessment above, we consider that a moderate damage level is appropriate for critical or major infrastructure, as highlighted within Table 11.

Table 11: Critical or Major Infrastructure Damage Level

Damage Level	Critical or Major Infrastructure
Catastrophic	Extensive and widespread destruction and damage to several major infrastructure components.
Major	Extensive destruction and damage to more than one major infrastructure component.
Moderate	Significant damage to at least one major infrastructure component.
Minimal	Minor damage to major infrastructure components.

5.4 Time to Restore Operation to Critical or Major Infrastructure

We consider any damage to critical or major infrastructure is likely to take up to three months to restore operation. Therefore, a moderate damage level is appropriate to restore operation to critical or major infrastructure, as highlighted within Table 12.

Table 12: Time to Restore Operation to Critical or Major Infrastructure

Damage Level	Critical or Major Infrastructure
Catastrophic	More than one year
Major	Up to 12 months
Moderate	Up to three months
Minimal	Up to one week

5.5 Natural Environment

The effects of a dam breach on the natural environment downstream may include deposition of sediment and scour within the downstream watercourses, potentially impacting water quality and fish habitat.

We consider that the damage to the natural environment downstream of the dam is likely to be significant but recoverable. Therefore, we considered that a moderate damage level is appropriate for the natural environment, as highlighted within Table 13.

Table 13: Natural Environment Damage Level

Damage Level	Natural Environment
Catastrophic	Extensive and widespread damage.
Major	Heavy damage and costly restoration.
Moderate	Significant but recoverable damage.
Minimal	Short-term damage.

5.6 Community Recovery Time

We consider the community would take months to recover from a dam breach. Therefore, we consider that a Moderate damage level is appropriate for the community recovery time, as highlighted within Table 14.

Table 14: Community Recovery Time Damage Level

Damage Level	Community Recovery Time
Catastrophic	Many years
Major	Years
Moderate	Months
Minimal	Days to weeks

5.7 Damage Level Summary

Table 15 summarises the selected damage levels for each of the categories. The highest damage level from the five categories is major and therefore, the damage level for the dam is major.

Table 15: Damage Level Summary

Category	Damage Level
Residential Houses	Major
Critical or Major Infrastructure	Moderate
Time to Restore Operation to Critical or Major Infrastructure	Moderate
Natural Environment	Moderate
Community Recovery Time	Moderate

6.0 Population at Risk

6.1 General

The Population at Risk (PAR) is defined as the number of people likely to be incrementally affected by inundation greater than 0.5m if a dam breach occurs. When evaluating PAR, the potential evacuation of people is not considered. The NZSOLD Guidelines require the PAR to be determined as one of the following:

- 0
- 1 to 10
- 11 to 100
- Greater than 100

The PAR will vary with time of day, week, and year. The NZSOLD Guidelines state that the most critical situation should be used to determine the PAR. The PAR does not take into account exposure times, except for temporary populations on designated routes. We have not undertaken a site inspection as part of the PAR estimate.

The following sections provide an outline of the assessed PAR.

In general, the model results indicate that the areas to the west of SH1 within Ohaeawai Village will experience flooding greater than 0.5m depth. The area to the east and south will experience flooding but generally less than 0.5m depth. There are some notable features in this area such as:

- Ohaeawai School
- Ohaeawai Community Pre-School

- Ohaeawai Hotel
- A freedom camping location (Carpark Te Corner)

These features are not included in the PAR assessment, as the predicted flood depth does not exceed 0.5m.

6.2 Residential Houses

As presented within the residential house damage level assessment, a number of residential houses appear to be located in areas where inundation depths are predicted to exceed 0.5m (above surrounding ground levels). These houses are highlighted within the drawings. The total number of houses meeting the 0.5m threshold is predicted to be 36 to 47 (noting again that aerial imagery has been used to identify residential houses from other buildings such as sheds etc.). We note that a specific floor level survey has not been undertaken. If such a survey was undertaken, the number of houses meeting the threshold may reduce.

As per the latest census in 2018, the population of 1,140 in Ohaeawai was located within 408 occupied houses, which is approximately 2.8 people/house. Assuming an occupancy rate of 2.8 people/house, the PAR associated with the residential dwellings is 100 to 130.

We note that we have not made a specific increased allowance for a bed and breakfast (Quiet Waters) located with the 0.5m deep floodplain.

6.3 Community Facilities

We have not identified any facility that will be affected by at least 0.5m depth of water and therefore the PAR will be zero.

6.4 Business Areas

The majority of the small businesses within the village appear to be located outside the 0.5m deep floodplain. The LINZ building classification indicates that one commercial building (at 41 SH1) will be affected by flood waters greater than 0.5 depth. The large building appears to be associated with an orchard or similar. The PAR associated with commercial premises is difficult to estimate, without undertaking a site inspection. As a conservative estimate we have allowed for a workforce of 10 to 20 people at the building, and within the property as a whole.

6.5 Recreational Areas

The Ohaeawai Rugby Club fields are predicted to be inundated by depths greater than 0.5m, although the flood depth at the clubrooms appears to be less than 0.5m. If the breach occurred at the time of a rugby game, the PAR could be in the order of 30 to 50.

We have not identified any other specific public areas that will be exposed to flood depths greater than 0.5m.

6.6 Road Crossings

The dam breach floodplain exceeds 0.5m depth across both SH12 and Remuera Settlement Road.

Considering exposure times, the PAR associated with road crossings is likely to be low. We consider the PAR associated with road crossings is likely to be less than 5.

6.7 Discussion

The PAR may vary considerably depending on the time of day and day of week of a breach. We consider Table 16 provides an appropriate summary of the PAR, noting that the PAR associated with business and recreational areas will be significantly lower at times (i.e. at night for both business and recreational areas and during week days for recreational areas). The PAR is in the highest category, for the PIC assessment (greater than 100), therefore, we do not consider it critical to further refine the PAR estimates.

Table 16: Population at Risk Summary

Type	Population at Risk
Residential Houses	100 - 130
Community Facilities	0
Business Areas	10 - 20
Recreational Areas	30 - 50
Road Crossings	0 - 5
TOTAL	140 - 205

7.0 Potential Loss of Life

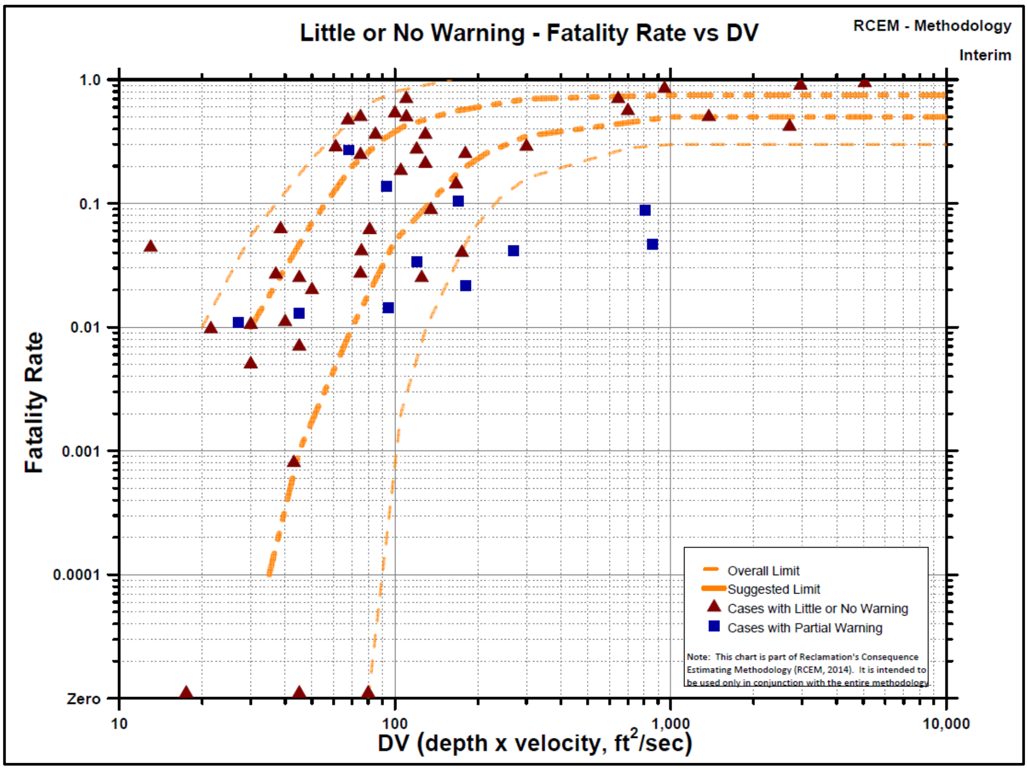
The NZSOLD Guidelines require that a high PIC is used if two or more lives are highly likely to be lost or a medium PIC if a life is highly likely to be lost. The NZSOLD Guidelines do not provide a definition of highly likely or guidance on the weighting of the different potential dam breach scenarios (unlike the PAR where the guidelines clearly state that the most critical situation should be used). The potential loss of life (PLL) takes evacuation into account.

In 2014, the United States Bureau of Reclamation developed a methodology for estimating PLL entitled Reclamation's Consequences Estimation Methodology (RCEM). RCEM provides a graphical approach giving the fatality rate as a function of the D x V and amount of warning time (based on measured fatality rates in actual dam breach events).

Model results indicate that the first residential houses downstream of the dam could be inundated by greater than 0.5m of water within 20-minutes of an instantaneous breach initiation such as a seismic event. We therefore consider that there is a possibility that the opportunity for evacuation is limited.

Figure 6 presents a figure from RCEM for little to no warning. The figure uses the empirical units of ft^2/s. The important feature of the figure, in this case, is that a D x V product of $30ft^2/s$ (or approximately $3m^2/s$) has a fatality rate of approximately 0.01 (at the upper end of the suggested limit). The flood drawings present the DxV results. The typical DxV in the vicinity of the PAR is approximately $3m^2/s$. Using a fatality rate of 0.01 and assuming PAR of 140, the statistical PLL is 1.4.

Figure 6: RCEM 2014 Fatality Rate – Little to No Warning



We consider that a detailed assessment of the residential houses and PAR would be required to further refine the PLL assessment. However, there is sufficient evidence to conclude that it is highly likely that at least one life will be lost.

8.0 Potential Impact Classification

The PIC assessment is summarised within Table 17 (as taken from the NZSOLD Guidelines). Given that the damage level is major, the PAR more than 100, the table indicates that the dam should have a high PIC.

We note that as four or more houses are highly likely to be destroyed, the assessed dam classification is not sensitive to the other damage categories or the PAR.

Table 17: Determination of Dam Classification

Assessed Damage Level	Population at Risk (PAR)			
	0	1 to 10	11 to 100	More than 100
Catastrophic	High	High	High	High
Major	Medium	Medium/High[4]	High	High
Moderate	Low	Low/Medium/High[3,4]	Medium/High[4]	Medium/High[2,4]
Minimal	Low	Low/Medium/High[1,3,4]	Low/Medium/High[1,3,4]	Low/Medium/High[2,3,4]

Notes:

1. With a PAR of five or more people, it is unlikely that the potential impact will be low.
2. With a PAR of more than 100 people, it is unlikely that the potential impact will be medium.
3. Use a medium classification if it is highly likely that a life will be lost.
4. Use a high classification if it is highly likely that two or more lives will be lost.

9.0 Flood Design Criteria

The PIC assessment classifies the dam as High PIC. The NZSOLD Guidelines recommend that a high PIC dam has an Inflow Design Flood (IDF) between the 10,000-year flood event and the Probable Maximum Flood (PMF), as outlined in Table 18.

Table 18: Recommended Minimum Inflow Design Floods (NZSOLD, 2015)

PIC	PAR	PLL	IDF
Low	0 to 10	0	100 to 1,000
Medium	0 to 10	0	1,000
	0 to 10	1	2,500
	11 to 100	0 to 1	10,000
High	No limits	0 to 1	10,000
	No limits	>10	PMF

In this instance, we consider that the most appropriate design event is the PMF.

10.0 Hydrology

10.1 Methodology

NZSOLD (2015) recommends that two or more methods are used to determine the inflow design flood. For this assessment we have:

1. Developed a rainfall-runoff model using HEC-HMS.
2. Undertaken a regional based flood frequency assessment.

We have not undertaken a flood frequency analysis on nearby flow gauges, noting that both the NIWA portal and NRC website show there are three flow gauges on Waitangi River. These are Waitangi at Waimate North Road (since 2016), Waitangi at SH10 (since 2012) and Waitangi at Wakelins (since 2001).

We note that there are large uncertainties in estimating flood events in excess of the 100-year event. We have therefore used a conservative approach as suggested by NZSOLD (2015) in determining the appropriate inflow design flood. We also note that the hydrological hazards (as well as the understanding of) can change with time, and therefore a conservative approach may also reduce the need for future upgrade works to the spillway facilities. We have not specifically allowed for climate change as recommended by NZSOLD (2015).

The NZSOLD Guidelines recommend that the PMF should be determined using Tomlinson and Thompson (1991) (and Campbell et al (1994)). However, this document was superseded by an article in the Journal of Hydrology (Volume 31 No. 2), also by Thompson and Tomlinson in 1993, for rainfall durations from 0.5-hours to 6-hours in length. The 1993 method has been used to determine total rainfall depths for a range of rainfall durations.

We have elected to undertake an assessment of the following design events:

- Mean annual flood event.
- 100-year flood event.
- 1,000-year flood event.

- 10,000-year flood event.
- Probable Maximum Flood (PMF).

10.2 Catchment Area

The catchment area was determined using the 5m DEM previously discussed. RILEY Dwgs: 200240/3-210 and -211 present the derived catchment boundary with a catchment area of 3.1km^2. We note that the NIWA GIS Portal indicates that the catchment area is between 3.42km^2 and 3.66km^2.

10.3 Infiltration

A number of methods are available to allow for soil infiltration (i.e. precipitation loss) during rainfall events. Soil infiltration is typically categorised/influenced by soil types and ground cover.

We note that NRC does not appear to have a preferred method for soil infiltration allowance. The Soil Conservation Service (SCS) method is commonly used, however, and is specified by Auckland Council within TP108. The SCS method categorises soil types into four groups (Group A, B, C or D) based on soil types. We anticipate that the soils within the catchment mainly consist of Group C soils as presented in RILEY Dwg: 200240-211. Group C soils are described as:

> *Soils that have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (1 to 4 mm/hr).*

The majority of the catchment is covered in pasture. Group C soils with pasture cover in good condition have a CN of 74, in accordance with SCS Technical Report 55 (1986).

The SCS method also requires the selection of the Initial Abstraction (Ia). Ia represents the initial precipitation loss at the start of a rainfall event. We note that TP108 recommends the use of Ia = 5mm in the Auckland Region.

For the purposes of this assessment we have used the SCS method, with a Curve Number (CN) of 74 for previous surfaces. We note that the Priority Rivers Modelling Report (URS, 2011) used a CN of 74 for the Waitangi River Catchment. We have used Ia = 5mm.

The proposed reservoir covers approximately 6.7% (0.21km^2) of the total catchment area. We have therefore adjusted the CN to 76 to obtain a weighted value for the entire catchment. We have not allowed for other impervious areas within the catchment.

10.4 Transform

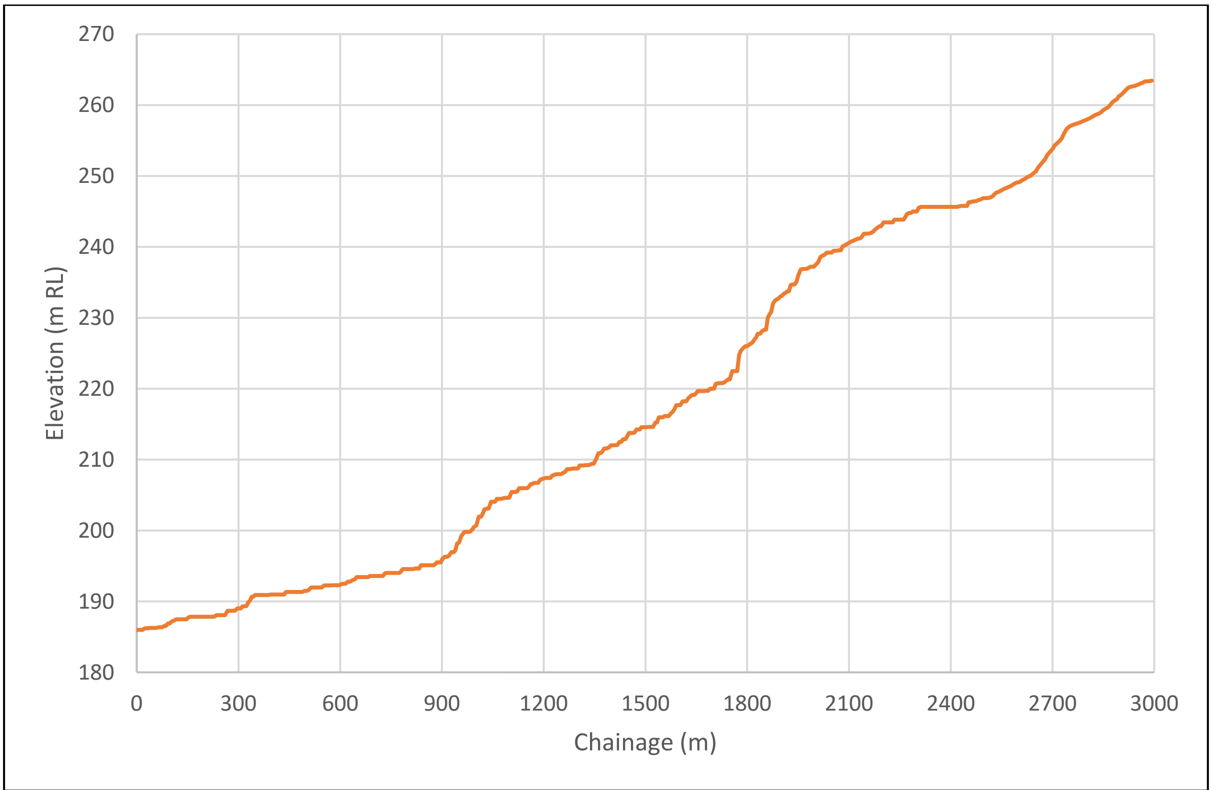
A number of methods are available to model the transformation of excess precipitation to runoff. We note that NRC does not appear to have a preferred method for the region, however, the Priority Rivers Modelling Report (URS, 2011) used the SCS Unit Hydrograph method, and it is also used with TP108.

For the purposes of this assessment we have used the SCS Unit Hydrograph method, along with a Peak Rate Factor of 484 (PRF 484). PRF 484 is the standard factor used. Other factors are available which result in peakier or flatter runoff hydrographs. Without any observed events to calibrate to for the catchment, we consider that a PRF of 484 is the most appropriate to use.

10.5 Time of Concentration

Figure 7 presents a long section along the longest flow path to the dam site from the upstream reaches of the watercourse. The maximum elevation with the catchment is RL 372m. The average gradient of the catchment was estimated to be approximately 2.6%.

Figure 7: Longest Flow Path Long Section



We have used various methods to estimate the time of concentration as presented within Table 19. The methods generally use flow path length, catchment area and elevation change as input parameters. The TP108 method was specifically derived for Auckland catchments.

Table 19: Time of Concentration (Minutes)

Method	Value
Ramser Kirpich	40
Bransby Williams	80
TP108	73

The Bransby Williams and TP108 methods provide similar results. We consider that the use of a time of concentration of 75-minutes is appropriate. The SCS unit hydrograph method uses lag time as the input parameter instead of time of concentration, where the lag time is equal to two thirds of the time of concentration. Therefore, a lag time of 50-minutes has been used within the assessment.

10.6 Rainfall Depth

10.6.1 Design Rainfall Depths

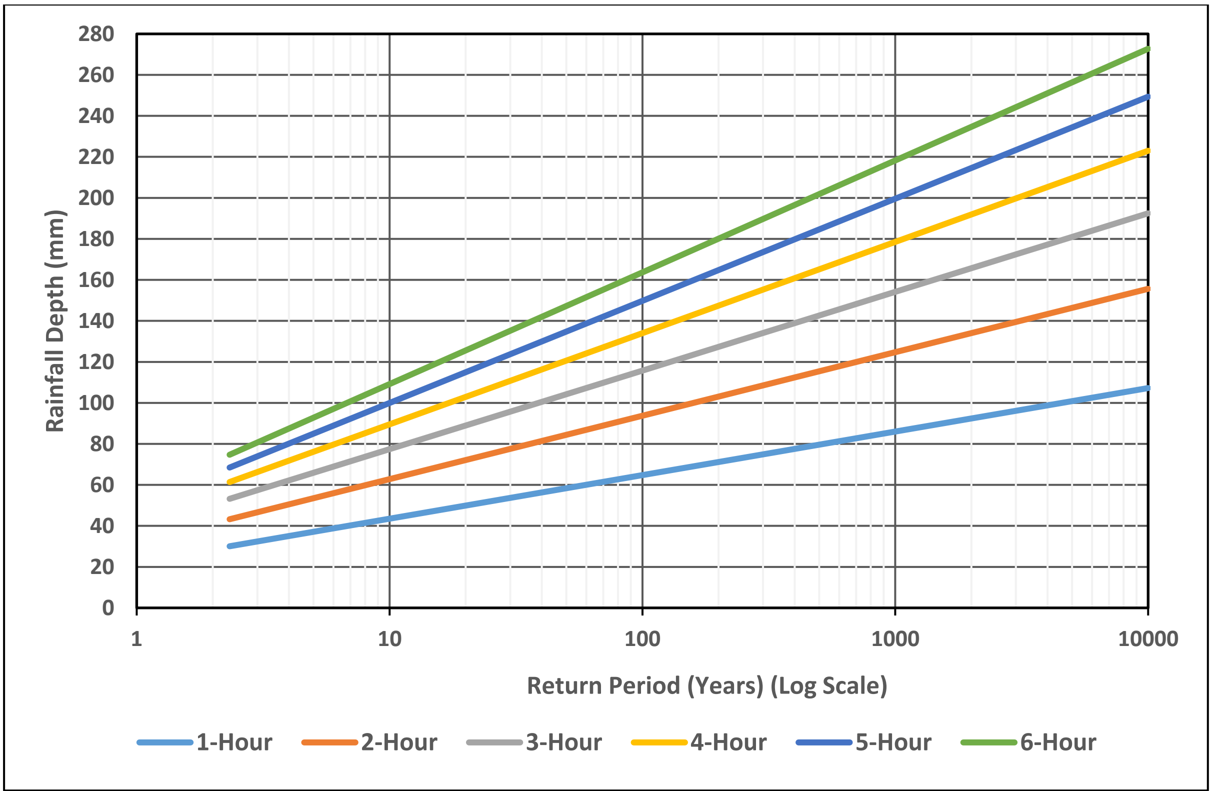
We consider extrapolations of the High Intensity Rainfall Design Systems (HIRDS) data provides the best estimate of rainfall depths up to the 10,000-year event at this time.
Table 20 provides a summary of the selected rainfall depths for the full range of rainfall durations.

Table 20: High Intensity Rainfall Design Systems Rainfall

Rainfall Event	Duration (hours)						6 to 1 Hour Ratio
	1	2	3	4	5	6	
2.33-Year	30	43	53	61	68	75	2.5
100-Year	65	94	116	134	150	164	2.5
250-Year	73	106	131	152	170	185	2.5
1,000-Year[1.]	86	125	154	179	200	218	2.5
10,000-Year[1.]	107	156	192	223	249	273	2.5

Note: [1] Extrapolated on a log scale

Figure 8: Design Rainfall Depths



10.6.2 Probable Maximum Precipitation

Thompson and Tomlinson (1993) provides a methodology for estimating Probable Maximum Precipitation (PMP) depths for rainfall durations from 0.5-hour through to 6-hours. It uses a baseline point value of 220mm for rainfall durations of 1-hour and allowances are subsequently made for catchment area, catchment elevation, moisture potential (values generally reduce from north to south in New Zealand). We determined a 1-hour PMP of 205mm, based on a catchment area of 3.1km^2 and without any adjustments for catchment elevation and moisture potential. Using the Thompson and Tomlinson (1993) methodology, the 1-hour PMP depth is factored to other durations by selecting an appropriate 6-hour to 1-hour ratio. We have conservatively selected a ratio of 2.5, noting that the HIRDS information indicates an approximate ratio of 2.5. Table 21 presents the PMP depths used within the assessment, along with the ratios used as recommended by Thompson and Tomlinson (1993).

Table 21: Probable Maximum Precipitation Depths

Duration (hour)	Ratio to 1 Hour Duration	PMP (mm)	New Zealand Record (mm)[1.]	Australian Record (mm)[2.]
1	1.00	205	134	230
2	1.42	291	-	-
3	1.75	359	-	-
4	2.03	416	-	-
5	2.27	465	-	-
6	2.50	512	-	589
12	-	-	566	-

Note:
1. Sourced from NIWA (up until 31 December 2016).
2. Sourced from Australian Government Bureau of Meteorology.

The predicted PMP rainfall depths compare favourably with the New Zealand records. We have also included some Australian records for comparison. One of the largest recorded flood events in the Northland Region is the 1981 Kerikeri flood. Approximately 450mm of rainfall occurred in approximately 8-hours. We note that this event was not included within the dataset for Thompson and Tomlinson (1993). The determined six-hour PMP rainfall depths compares favourably with this event.

10.7 Temporal Distribution

There are a number of options available for the temporal distribution of the design rainfall depths as outlined below:

1. NRC Priority Rivers Hyetograph.
2. HIRDs Standard Project Storm Hyetograph.
3. Hyetograph from locally recorded rainfall events.

Figure 9 provides a comparison of the derived temporal distributions from the Priority Rivers method and the HIRDs method. We note that Thompson and Tomlinson (1993) does not provide a method for the temporal distribution of the total rainfall depth.

The HIRDS method has been derived hyetographs shapes for different regions within New Zealand. The area of interest is located in the north of the North Island Region. Parameter values are provided for use within a formulas for different durations. The two most relevant durations for this catchment are the 1-hour and 6-hour durations. The NRC Priority Rivers hyetograph was developed in 2010/2011 and uses a 12-hour duration event as a basis. We understand that a recent draft review for NRC has recommended that the HIRDS hyetograph be used in the short term as a replacement for the Priority Rivers hyetograph.

For the purposes of this assessment we consider that the HIRDS hyetograph is the preferred approach, noting that it has been developed on a regional basis for specific durations in the order of those that will be critical for this catchment (i.e. 1-hours to 6-hours). HIRDS provides different parameters for the 1-hour and 6-hour events. Figure 10 presents the different distribution for the 1-hour and 6-hour events. The critical events for the catchment are likely to be somewhere between the 1-hour and 6-hour event, however, we have elected to use the 6-hour parameters for all assessed durations, as we consider that the critical duration events are likely to be closer to 6-hours.

Figure 9: Temporal Distribution Comparisons

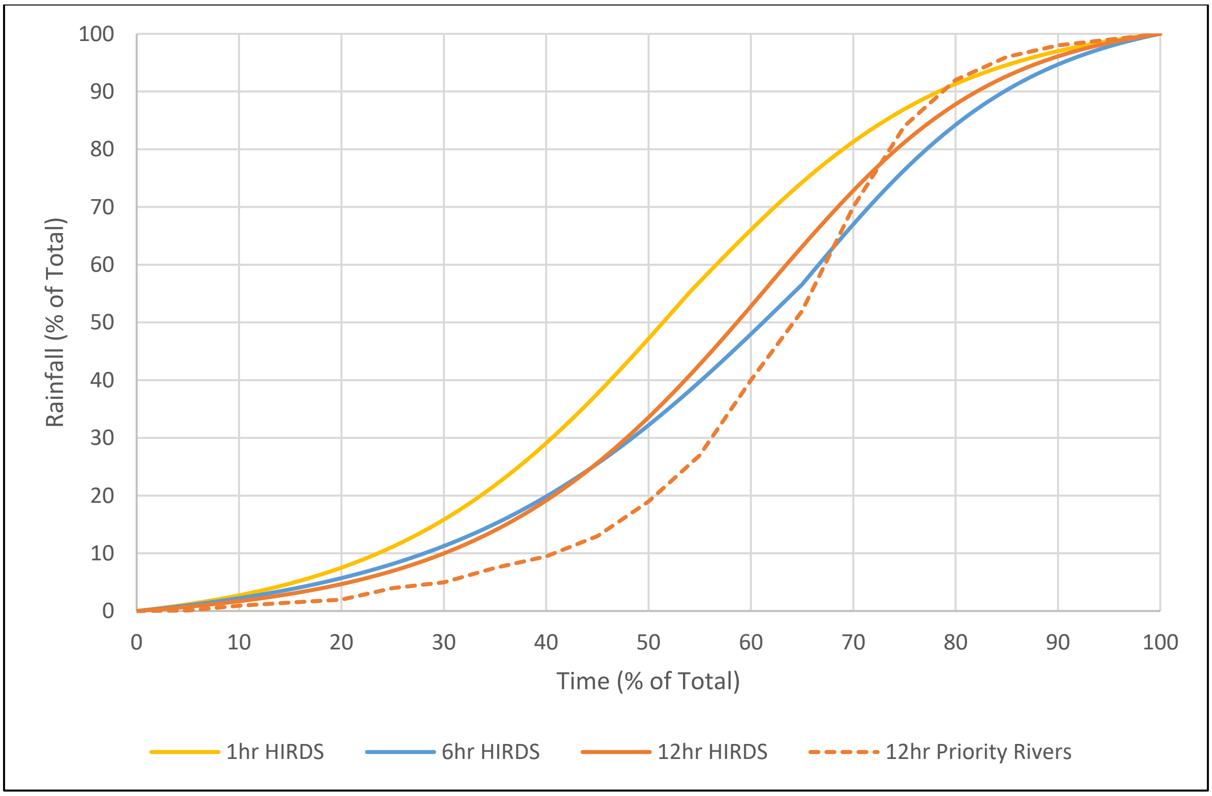
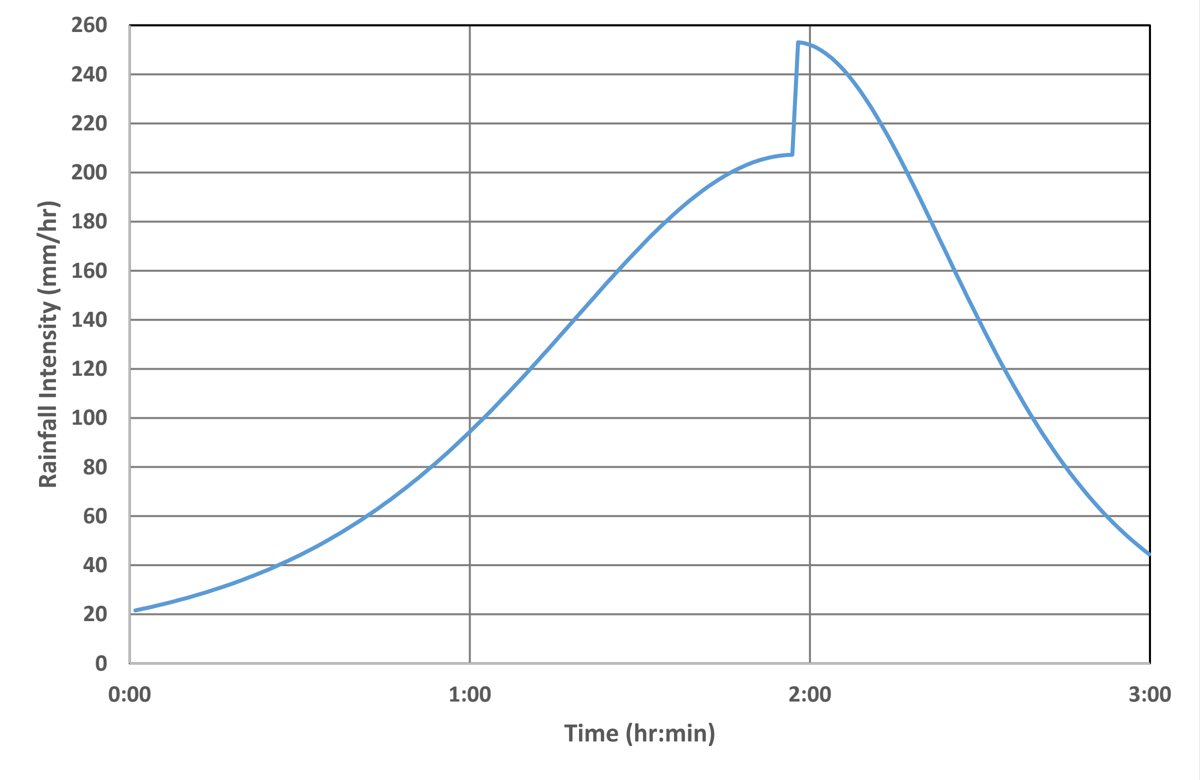


Figure 10 presents the design PMP 3-hour rainfall hyetographs.

Figure 10: PMP 3-Hour Rainfall Hyetographs



10.8 Inflow Design Hydrographs

A HEC-HMS rainfall-runoff model has been developed with a single sub-basin utilising the input parameters detailed in the previous sections. The model results are presented within Table 22, with the critical durations highlighted in red.

Table 22: Rainfall Runoff Model Peak Flow Results

Event	Duration (hr)						
	2	3	4	5	6	7	8
2.33-Year	-	-	9.6	10.0	10.2	10.1	10.0
100-Year	-	-	31.3	31.5	31.2	30.5	-
10,000-Year	-	59.4	60.8	60.1	58.6	56.8	-
PMF	118.9	126.7	126.6	123.1	119.5	-	-

10.9 Regional Methods

McKercher and Pearson (1989) presents a regional method for determining mean annual and 100-year flood magnitudes. The results of the assessment are summarised in Table 23.

Table 23: McKerchar and Person Regional Method

Value	Dam Site
$Q_{2.33}/A^{0.8}$	5.0
$Q_{2.33}$ (m^3/s)	12.4
q_{100}	2.7
Q_{100}	33.4

A revised regional method is the New Zealand River Flood Statistics GIS portal. The information indicates that the mean annual flood slightly downstream of the proposed dam site (3.66km^2 catchment) is 4.0m^3/s with a 100-year flow of 10.0m^3/s (a Q_{100}:$Q_{2.33}$ ratio of 2.5).

10.10 Observed Flood Events

We have not reviewed observed flood events specifically at the site as it is outside the scope of the assessment. However, on a regional basis, we note that during the 1981 Kerikeri event the estimated flow at the Maungaparerua gauge was 184m^3/s equating to a specific discharge of 16.5m^3/s/km^2 for a catchment area of 11.1km^2 (NIWA 2009). The flow was estimated to have a return period of close to 1,000-years i.e. the event was extreme. We note, however, that there is some uncertainty associated with the estimated peak 1981 flows (as well as the maximum rainfall). We also note that NIWA 2009 states that on the basis of rainfall records from the Kerikeri storm, the existing New Zealand PMP estimates may be too low.

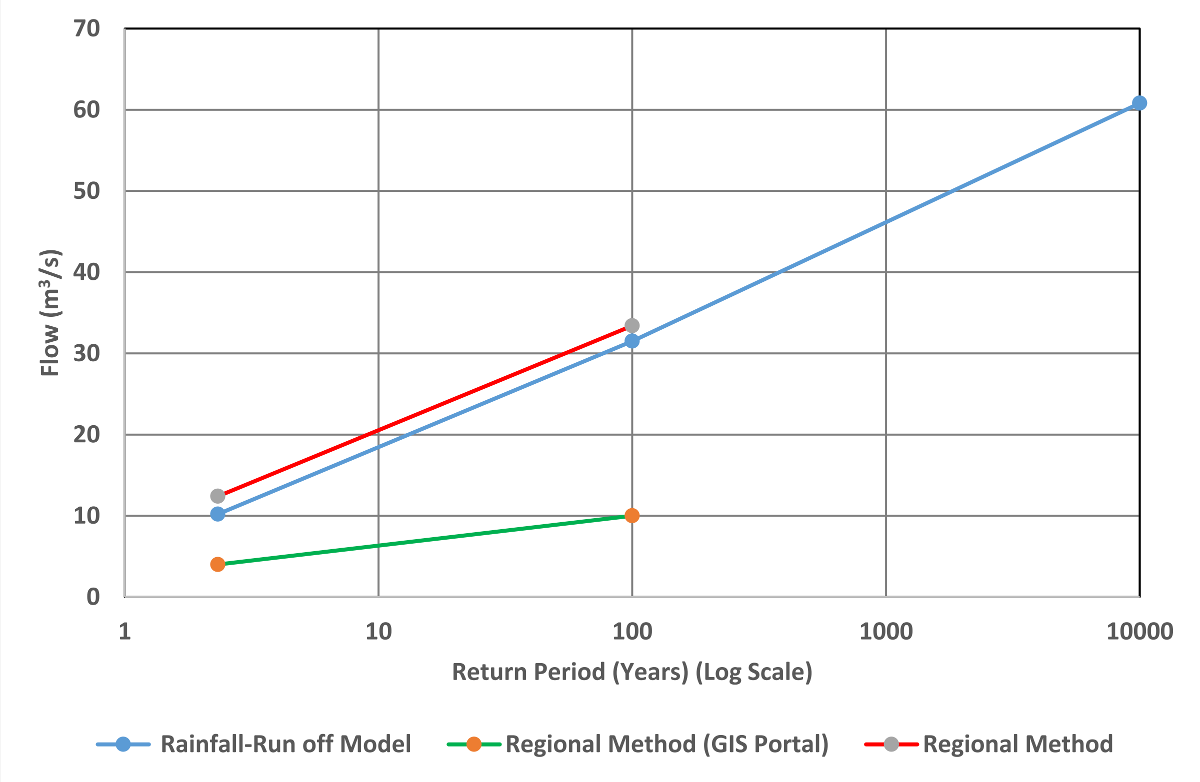
10.11 Summary

Table 24 presents a summary of the peak inflows derived using the various methods. The results are also presented in Figure 11 (with a log scale).

Table 24: Peak Flow Results (m^3/s)

Method	2.33-Year	100-Year	10,000-Year	PMF	Q_{100}:$Q_{2.33}$ Ratio
Rainfall-Runoff Model	10.2	31.5	60.8	127	3.1
Regional Method New Zealand River Flood Statistics GIS portal	4.0	10.0	-	-	2.5
Regional Method McKercher and Pearson (1989)	12.4	33.4	-	-	2.7

Figure 11: Peak Flow Results



The New Zealand River Flood Statistics GIS portal appears to underestimate the mean annual flood and therefore the 100-year flood (noting that the Q_{100}:$Q_{2.33}$ ratio of 2.5 is comparable to other methods).

For the proposes of this assessment, we consider that the rainfall-runoff model provides appropriate inflow design hydrographs.

11.0 Spillway Design

11.1 Design Criteria

The adopted design criteria is summarised in Table 25.

Table 25: Design Criteria

Element	Criteria
Service Flood	100-year flood event to be passed with very low-probability of erosion within the spillway arrangement.
Design Flood	PMF flood event to be passed with adequate freeboard to the dam or wave wall crest. Freeboard the greater of 900mm or the sum from the wind set up and wave run up from the 10% annual exceedance probability (AEP) wind.
Construction Diversion	Less than 2% probability of the partially completed dam being overtopped.

11.2 Methodology

We have used HEC-RAS (v5.07) to simulate the hydraulic performance of the reservoir and spillway.

11.3 Geometry and Spillway Design

The reservoir has been modelled as a 2D flow area. The 2D flow area is connected to a downstream 2D flow area via a connection represented by the spillway crest.

For the purposes of the preliminary design we have assumed that there is a single overflow spillway. During detailed design, a dual spillway arrangement may be considered, with a service and an auxiliary spillway. The primary spillway will be designed to have a low risk of erosion during more frequent and smaller magnitude flood events. The spillways will be located entirely within natural ground.

The preferred spillway location is on the right abutment, with the spillway discharging to the Pekapeka Stream approximately 100m downstream of the dam toe. The preliminary spillway has been designed with a sill elevation of RL 205.0m and a lower sill width of 10m and an upper sill width of 30m at RL 205.5m (total spillway width of 40m). The spillway design was incorporated into the 5m DEM described previously.

The downstream 2D flow area extends from the spillway entrance to a point approximately 800m downstream of the dam. A refinement region with a grid size of 0.5m by 0.5m has been used within the spillway chute.

We have used a Manning's 'n' value of 0.03 to reflect a grassed lined spillway. Future detailed design may consider the use of a concrete chute spillway or a combination of a concrete chute and grassed lined. Erosion protection at the downstream toe of the chute will also need to be considered. We envisage that riprap lining will be adequate.

11.4 Initial Condition

We have used an initial condition of RL 205.0m for the reservoir 2D flow area.

11.5 Upstream Boundary Condition

The results from the HEC-HMS model have been used as inflow hydrographs to the reservoir.

11.6 Downstream Boundary Condition

We have used the normal depth calculation method with a friction slope of 0.002, to correspondence with the general longitudinal gradient of the terrain in the region of the downstream boundary location.

We do not consider the downstream boundary condition is critical to the assessment as the boundary location is sufficiently downstream of the area of interest at the downstream toe of the dam.

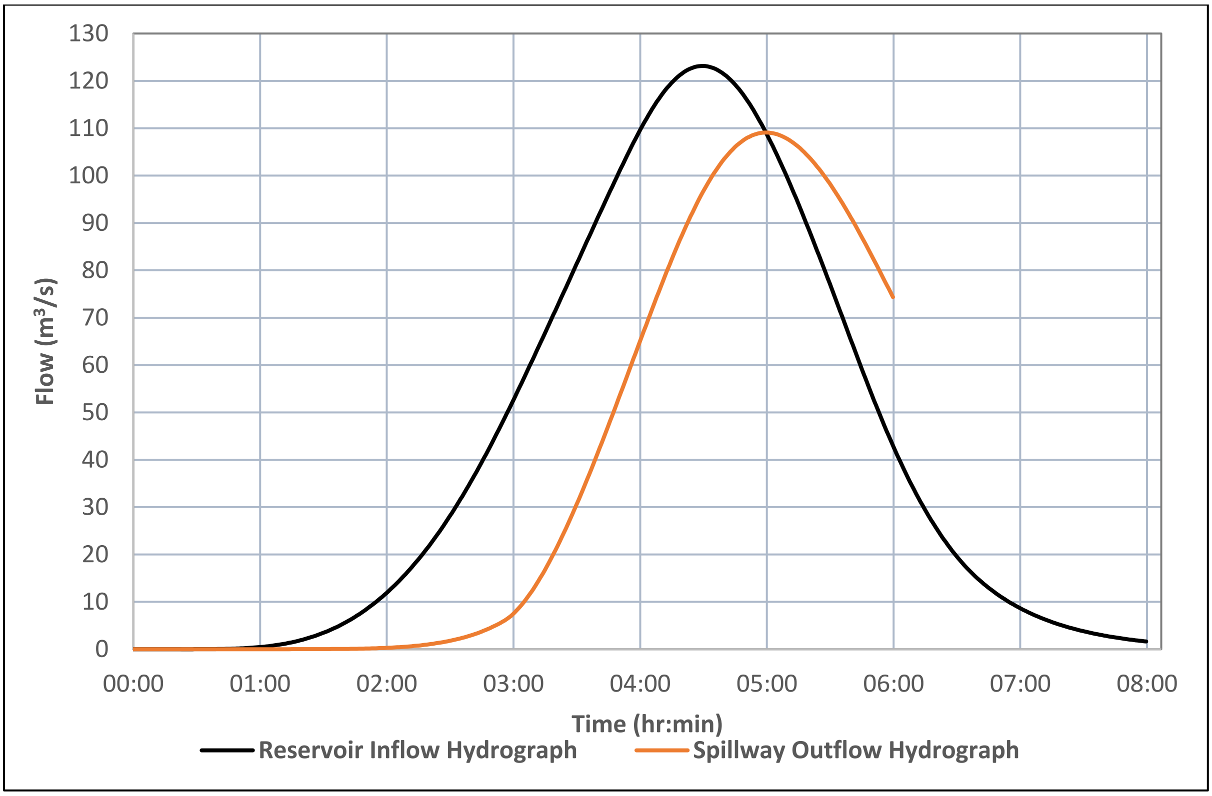
11.7 Reservoir Results

The critical duration was found to be 5 hours for the PMF event, with a peak reservoir level of RL 206.8m (rounded up to the nearest 0.1m).

The proposed dam embankment crest level is RL 207.0m. The model results indicate that a 700mm high wave wall is required to provide 900mm freeboard. During future detailed design, the spillway arrangement and dam crest level may be able to be optimised further. Sensitivity analysis should also be undertaken including assessing the available freeboard.

The reservoir inflow and outflow hydrographs are presented in Figure 12. The peak inflow of $123m^3/s$ is attenuated by the reservoir to a peak outflow of $109m^3/s$.

Figure 12: PMF Flow Hydrographs



11.8 Velocity Considerations

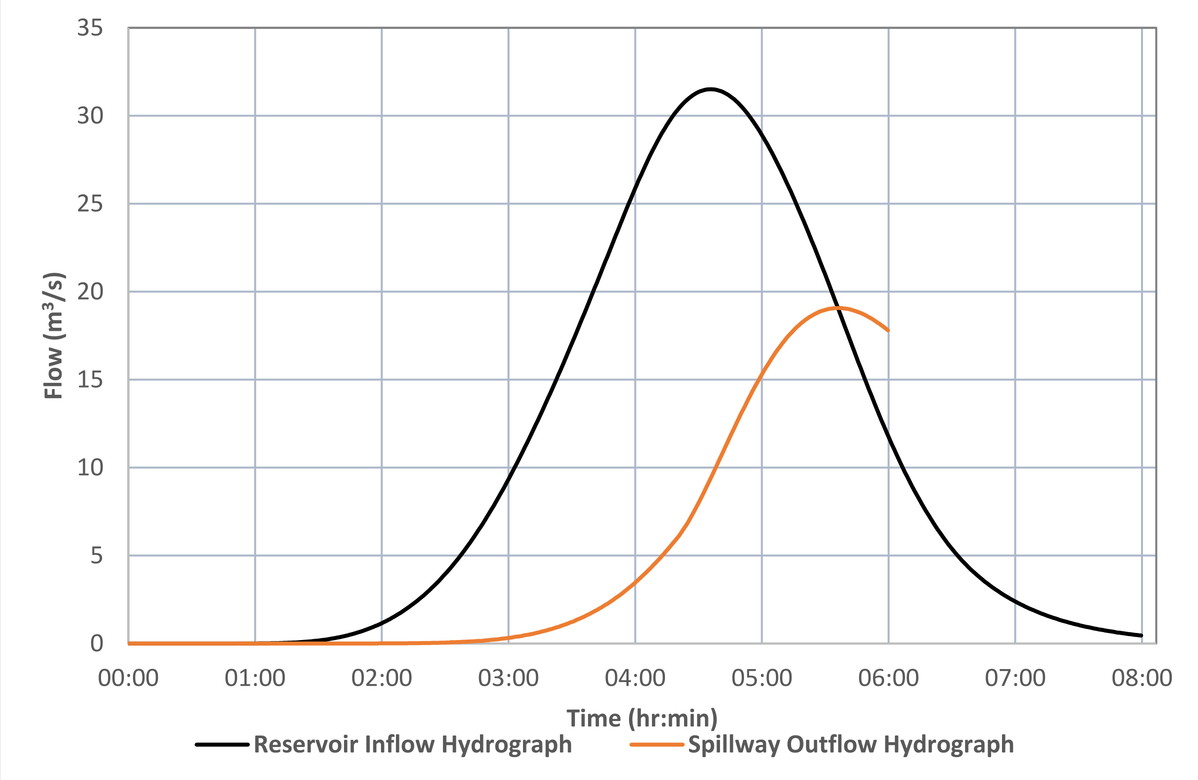
Preliminary results indicate that velocities within the spillway chute may reach 10m/s depending on the final longitudinal profile, during the PMF event. We therefore consider that at least the service spillway chute will be constructed from concrete to provide adequate erosion protection. Erosion protection will also be required at the toe of the spillway, where the transition to the stream occurs.

12.0 Flood Attenuation

A secondary objective of the proposed dam design is the capacity to attenuate peak flows from the catchment. The effect of this is a reduction in the flooding experienced by the downstream community.

The most relevant events to assess when considering flood attenuation are events in the order of the 100-year event, as larger events are less relevant to communities. The attenuation provided during the critical 5 hours duration 100-year event is presented within Figure 13.

Figure 13: 100 Year Flow Hydrographs



The peak inflow of 32m^3/s is attenuated by the reservoir to a peak outflow of 19m^3/s. The reduction of peak flow through the spillway is approximately 40% of the inflow which will significantly reduce the downstream flooding. We note that if the reservoir level is below the full supply level prior to the rainfall event, the attenuation provided by the dam would be increased, further reducing downstream flooding.

13.0 Diversion During Construction

The stream needs to be diverted during construction to provide a dry working area during construction and also to prevent the overtopping of a partially formed embankment. We have taken a risk-based approach to the diversion design as recommended by the NZSOLD Guidelines i.e. at lower dam heights the likelihood of overtopping is higher, however, the downstream consequence an embankment breach is lower. The construction cost risk has not been specifically considered, as it is intended that the contractor's construction insurance will cover the cost in this event. There is no public safety risk from a breach during foundation works.

The design intent is to construct the diversion culvert offline from the existing stream. When the culvert is completed, the creek will be diverted into the culvert, and the upstream shoulder of the dam will be preferentially constructed ahead of the downstream area, to form a cofferdam.

The NZSOLD Guidelines do not provide specific guidance on acceptable risk, however, it does state that "if the incremental consequences of a dam failure during construction include no potential for the loss of life downstream of the dam, a return period of 50-years may be appropriate for the sizing of the diversion works". Given appropriate monitoring and warning systems will be in place, we consider that the potential for loss of life is minimal.

Preliminary calculations indicate that a 1500mm to 1800mm dimeter culvert will have sufficient capacity to pass the 50-year flood. Further assessments will be required at detailed design stage, potentially including an analysis of floods with lower likelihood of occurring but with higher downstream consequences. The risk associated with the dam construction will vary throughout construction period.

14.0 Intake Details and Fish Passage

The following section includes details of the intakes and approach to fish passage.

Puhoi Stour completed a preliminary ecology assessment at Te Ruaotehauhau Water Storage Reservoir in July 2020 and has provided draft results indicating the potential effects from the proposed dam. Key issues that need to be considered in the design of the dam in regard to fish passage include:

- Migration of eels (elvers) upstream during peak migration periods (summer). Tuna (longfin eel) was the only migratory species found at the site. These elvers are <200mm in size (typically 100mm) and are good climbers even with minor flows.
- Consideration for downstream movement of migrant eels should, however, be included in spillway design to minimise the potential for injuries to occur.
- From the proposed Regional Plan water intakes will need screens with 3mm mesh and velocities into the screen of less than 0.12m/s based on Canterbury Guidelines.

We note that inanga, a native at-risk and migratory species, were found in the downstream extent of the site. They were not found in the upstream extent of the site and Puhoi Stour have assessed that any modification of access to the headwaters will not affect their lifecycle.

14.1 Upstream Migration of Elvers

The principal challenge with upstream passage is that the reservoir will have a large operating level range across the irrigation season. When the reservoir is full the barrier is 20m high for the elvers to climb to and the range from full to empty is challenging to design for. An elver pass may be feasible with a floating intake to operate in the upper few metres of the range but is not considered feasible for the entire operating level of the dam. When the reservoir water level is below the operating level of the elver pass then a trap and transfer system could be utilised to manage the upstream migration of eel.

Alternatively, a trap and transfer of elver could be undertaken without the construction of an elver pass. This involves a trap installed near the downstream toe of the embankment, within which the elvers enter via a short crawling medium into a holding tank. These are then physically transported and released over the dam. This would be located with a pass a minimal distance above the downstream water level to maximise the reservoir water level range it would operate over. The concept is the flow down the crawling medium attracts the elvers and excludes other unwanted species. If this approach were adopted the source of water could be via the dam and into the trap via the residual flow. The trap and transfer may only operate over peak migration, but adaptive management approach could be used in developing an efficient programme. This option has been used successfully on other large dam projects and therefore provides the greatest chance of success. An example of the elver trap used at Matahina Dam is shown in Photo 3.

Photo 3: Matahina Elver Trap

Another option is a trap pass system with a crawling medium all the way up to the dam crest. The system would enable the elvers to pass without intervention and a schematic is presented in Figure 14. Resting pools would be required at regular intervals up the slope, and a climbing medium would need to span the elver pass to allow elvers to attach. An open channel or frictionless chute such as a plastic pipe would then deliver the elvers to the reservoir and avoid elvers climbing back up. Figure 15 presents some indicative details. A continuous water supply would need to be pumped from the reservoir, albeit this would likely be small.

Figure 14: Schematic of trap pass

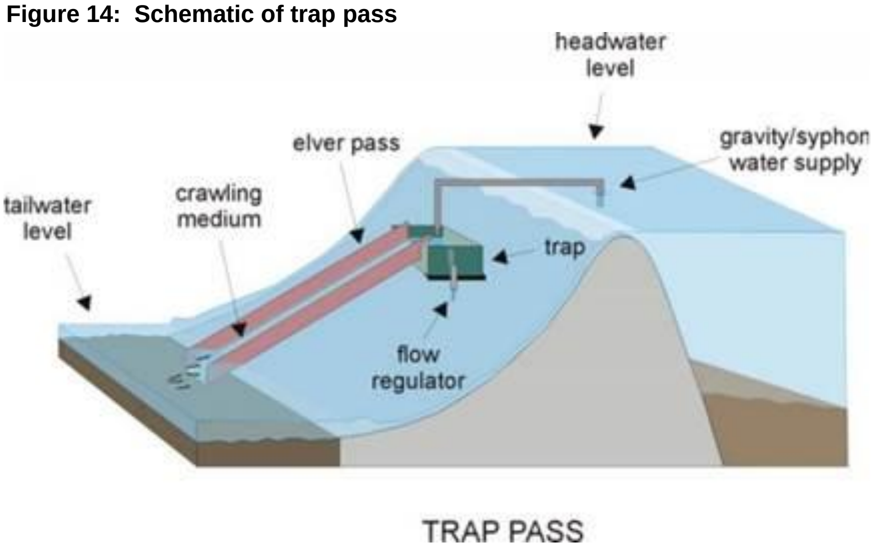
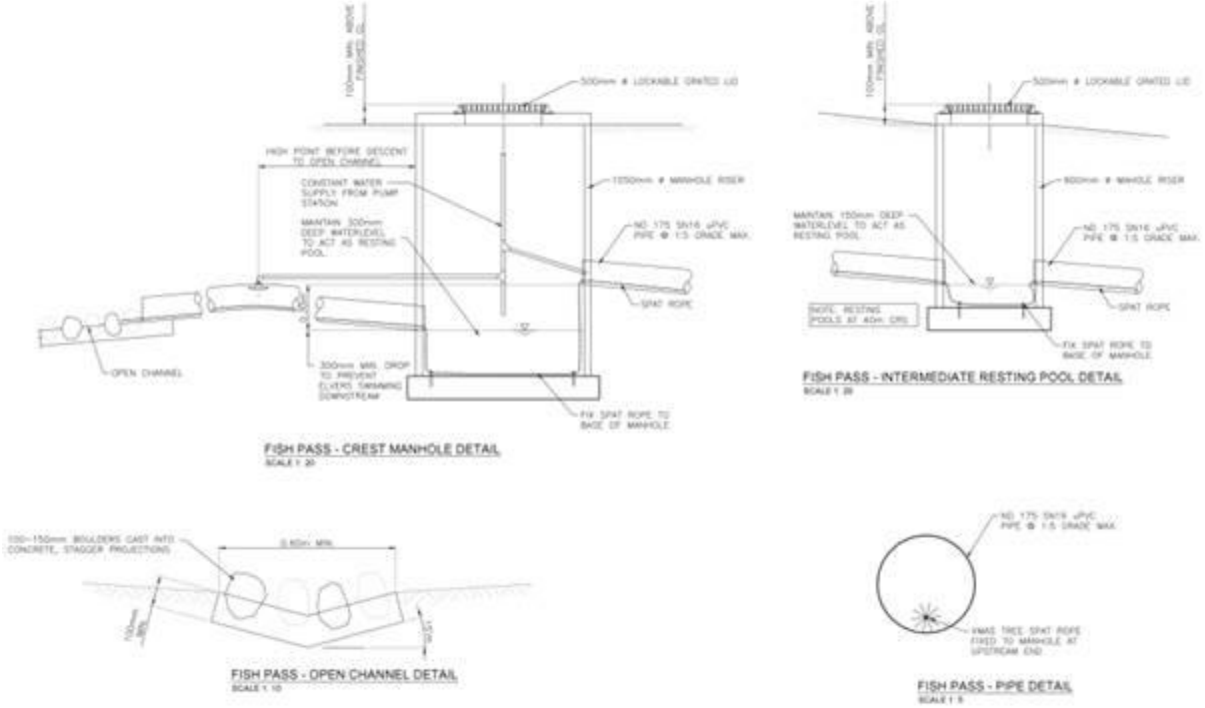


Figure 15: Typical details of trap pass



The nature of both an elver pass and trap and transfer are challenging, and it is likely that some modifications to the pass or the trap and transfer process will be required during operation. Monitoring of the effectiveness will need to be undertaken and where required modifications to resolve any issues implemented.

14.2 Spillway Design for Downstream Adult Eel Migration

The shaping of a spillway channel and downstream structures that are part of the spillway will consider what is required to minimise damage to eel. This will relate to depth of flows and any structures with the flow channel downstream and back to the river.

14.3 Intake and Screens

The dam will operate with both a residual flow requirement and an irrigation supply requirement. This will likely involve two separate smaller pipes housed within a larger pipe that also acts as temporary flood diversion during dam construction. Both smaller pipes will require a valve and flow meter to control and measure the flows released. The larger pipe will be provided with a gate to enable emergency dewatering of the reservoir. The intake will need to include a screen to comply with proposed regional plan to keep fish in the stream and also to avoid impingement onto the screen. This includes a requirement of a 3mm mesh screen. Given the small gaps in the screen there is a risk of the screen blocking and therefore, likely that a cleaning system will also be required. If the intake is a single intake located at the invert of the pond, then a rotary or retrievable screen may be used to ensure the screen is kept clean. Specific safety measures will be included that enable the reservoir level to be controlled and maintained in future.

15.0 Summary

The main findings and recommendations contained within this report are summarised as follows:

- A hydraulic model of a sunny day dam breach scenario and a subsequent PIC assessment indicates that the proposed dam has a High PIC. A rainy day scenario should be considered at detailed design stage.
- We consider that the design flood event should be the PMF.
- A preliminary spillway design has been prepared to ensure that adequate freeboard to the dam crest (including a wave wall) is maintained during the design flood event. Sensitivity analysis should be considered at detailed design stage.
- The spillway arrangement may be optimised further during detailed design. Erosion protection will also be considered further.
- The dam will provide significant flood attenuation for flood events up to and including the 100-year flood event.
- Stream diversion during construction will be managed through the preferential construction of the upstream shoulder of the dam to form a cofferdam.
- Preliminary calculations indicate that a 1500mm to 1800mm dimeter culvert will have sufficient capacity to pass the 50-year flood during construction. Further assessments will be required at detailed design stage.
- Methods to allow for fish passage upstream and downstream of the dam have been outlined. Further assessments to identify the most appropriate method will be required at detailed design stage.

16.0 Limitation

This report has been prepared solely for the benefit of Te Tai Tokerau Water Trust as our client with respect to the brief and Northland Regional Council in processing the consent(s). The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

The hydrological and hydraulic analyses and recommendations contained in this report are based on our understanding and interpretation of the available information. The recommendations are therefore subject to the accuracy and completeness of the information available at the time of the study. Should any further information become available, the analyses and findings of this report should be reviewed accordingly.

APPENDIX A

HEC-RAS Summary

HEC-RAS Model Overview: MiD North MN-06 Dam

Model Location:
T:\2020 Jobs\200240 Northland WSUP Feasibility Study\4.0 DESIGN-INVEST\4.3 Wat\Dam Break modelling\MN06\HEC-RAS

Created By:	HA
Checked by:	GL

Version	5.0.7
Projection	New Zealand Transverse Mercator 2000
Vertical Datum	NZVD 2016
Geometry Files	Sunny Day Piping
	PMF 5hr Spillway V3
	100YR 5hr Spillway
Terrain	5m 2017 LiDAR
	TOPO, DAM & Spillway Combined DEM V2
Unsteady Flow Files	Sunny Day Inflow
	PMF 5-Hour Reservoir Inflows
	100YR 5-Hour Reservoir Inflows

Plans	Plan	Unsteady Flow Files	Geometry Files
1	Sunny Day - Piping	Sunny Day -Piping	Sunny Day Piping
2	PMF 5hr Spillway V3	PMF 5-Hour Reservoir Inflows	PMF 5hr Spillway V3
3	100YR 5hr Spillway	100YR 5-Hour Reservoir Inflows	100YR 5hr Spillway

Model Input Details

Geometry Files

Geometry File:	Sunny Day Piping		PMF 5hr Spillway V3		100YR 5hr Spillway	
2D Flow Areas:						
Name:	MN-06 Reservoir	DS 2D AREA	Reservoir	Spillway 2D Area	Reservoir	Spillway 2D Area
Mannings n:	N/A	0.06	0.03	0.03	0.03	0.03
Grid Size:	E-V Curve	5m x 5m	10m x 10m; One 0.5x0.5m Refinement	5m x 5m; One 0.5x0.5m Refinement	10m x 10m; One 0.5x0.5m Refinement	5m x 5m; One 0.5x0.5 Refinement
Terrain Association:	5m 2017 LiDAR		TOPO, DAM & Spillway Combined DEM V2		TPO, DAM & Spillway Combined DEM V2	

Connections

Geometry File:	Sunny Day Piping	PMF 5hr Spillway V3	100YR 5hr Spillway
Name:	MN-06 DAM	Spillway	Spillway
Weir Width (m):	470.851	62.291	62.291
Weir Coefficient:	1.43	1.7	1.7
Weir Crest Shape:	Broad Crested	Broad Crested	Broad Crested
Overflow Computation Method:	Use Weir Equation	Use Weir Equation	Use Weir Equation
Structure Type:	Weir, Gates, Culverts, Outlet RC and Outlet TS	Weir, Gates, Culverts, Outlet RC and Outlet TS	Weir, Gates, Culverts, Outlet RC and Outlet TS
Embankment Station/Elevation Table:	Generated from DEM -Variable Elevation	Design Variable Cross Section	Design Variable Cross Section
Weir Level:	Variable with minimum RL 207m	Variable with minimum RL 205m	Variable with minimum RL 205m
Dam Breach:	Yes	N/A	N/A
Final Bottom Width:	14.7	N/A	N/A
Final Bottom Elevation:	185.3	N/A	N/A
Side Slope (V:H):	1:0.7	N/A	N/A
Breach Weir Coefficient:	1.7	N/A	N/A
Breach Formation Time (Minutes):	18	N/A	N/A
Failure Mode:	Piping	N/A	N/A
Piping Coefficient:	0.3	N/A	N/A
Initial Piping Elevation:	190.5	N/A	N/A
Trigger Failure At:	Set Time	N/A	N/A
Starting Date and Time:	05Aug2120 00:01	N/A	N/A

Unsteady Flow Files

Unsteady Flow Files:	Sunny Day Piping		PMF 5-Hour Reservoir Inflows		100YR 5-Hour Reservoir Inflows	
Storage/2D Flow Areas:	MN-06 Reservoir	DS 2D AREA	Reservoir	Spillway 2D Area	Reservoir	Spillway 2D Area
Boundary Condition:	Initial Condition	DS BC	Inflow Hydrograph	DS BC	Inflow Hydrograph	DS BC
Type:	Storage Area	Normal Depth	Timeseries Flow	Normal Depth	Timeseries Flow	Normal Depth
Enter Table:	N/A	N/A	✓	N/A	✓	N/A
Use Simulation Time:	N/A	N/A	Fixed Time	N/A	Fixed Time	N/A
Data Time Interval:	N/A	N/A	1 minute	N/A	1 minute	N/A
EG/Friction Slope:	N/A	0.00507	0.00001	0.00222	0.00001	0.00222
Data:	N/A	N/A	Generated from HEC-HMS	N/A	Generated from HEC-HMS	N/A
Initial Elevation /Value:	205.00	N/A	205.00	N/A	205.00	N/A

Plans

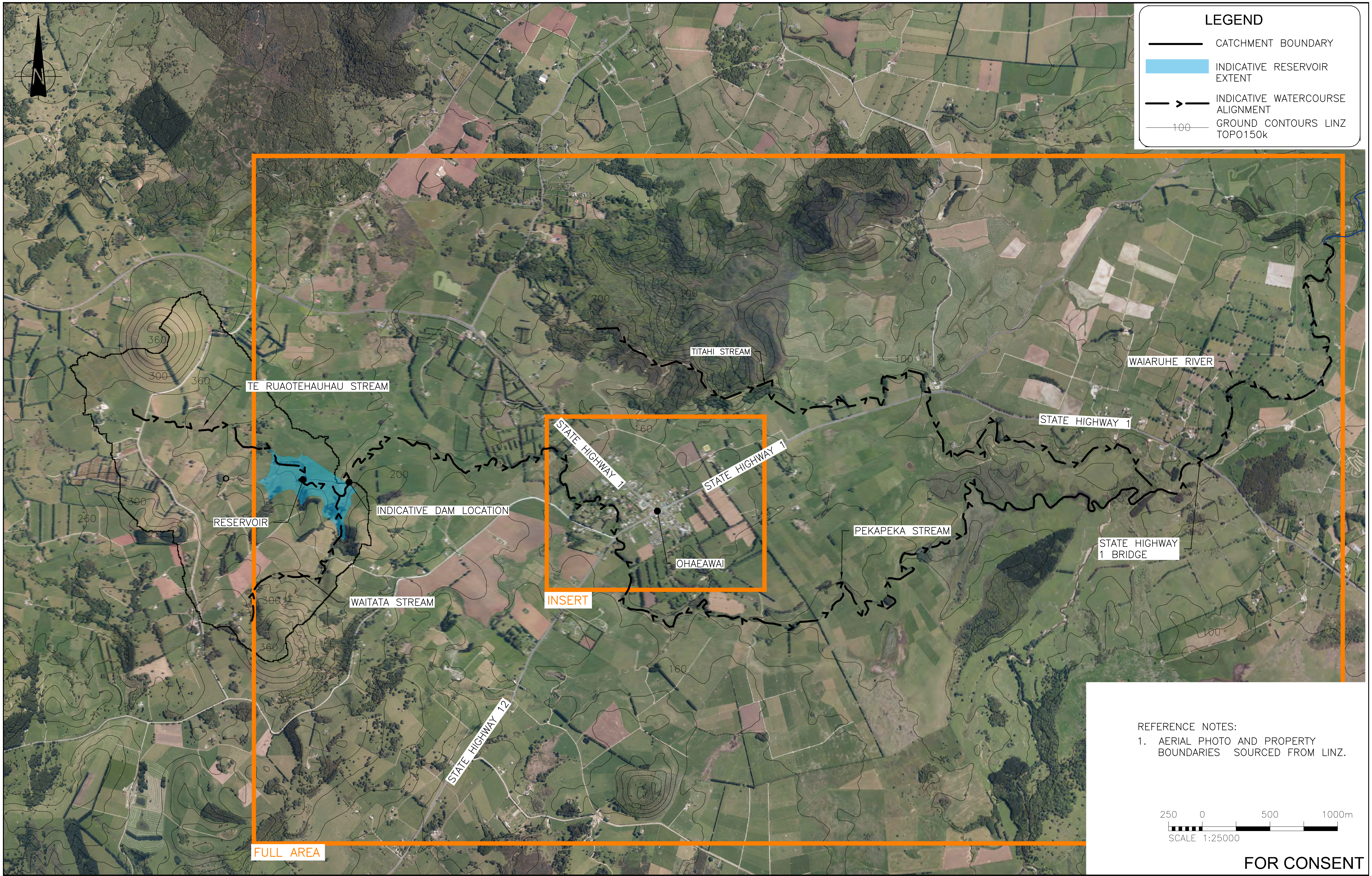
Plan:	Sunny Day - Piping	PMF 5hr Spillwqay V3	100YR 5hr Spillway
Geometry Preprocessor:	✓	✓	✓
Unsteady Flow Simulation:	✓	✓	✓
Post Processor:	✓	✓	✓
Starting Date:	5/08/2120	5/08/2120	5/08/2120
Starting Time:	0:00	0:00	0:00
Ending Date:	5/08/2120	5/08/2120	5/08/2120
Ending Time:	6:00	3:00	3:00
Computational Interval:	Vaiable time step	Vaiable time step	Vaiable time step
Mapping Output Interval:	1 minute	1 minute	1 minute
Hydrograph Output Interval:	1 minute	1 minute	1 minute
Detailed Output Interval:	1 minute	1 minute	1 minute
Equation Set:	Full Momentum	Diffusion Wave	Diffusion Wave

Model Output Details

Plan:	Sunny Day - Piping				PMF 5hr Spillway V3/100YR 5hr Spillway		
Layer Name:	WSE	Depth	Velocity	D * V	WSE	Depth	Velocity
Type:	Water Surface Elevation	Depth	Velocity	Depth * Velocity	Water Surface Elevation	Depth	Velocity
Unsteady Profile:	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum
Stored (saved to disk):	Raster Based on Terrain	Raster Based on Terrain	Raster Based on Terrain	Raster Based on Terrain	Raster Based on Terrain	Raster Based on Terrain	Raster Based on Terrain
Item:	5m 2017 LiDAR	5m 2017 LiDAR	5m 2017 LiDAR	5m 2017 LiDAR	TOPO, DAM & Spillway Combined DEM V2		

APPENDIX B

Drawings



REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN: GL
DES CHECK: GL
DRAWN: OOBG
CAD CHECK: JM
DATE DRAWN: AUG. 2020
APPROVED FOR ISSUE: D TATE
ISSUE DATE: 01/09/20

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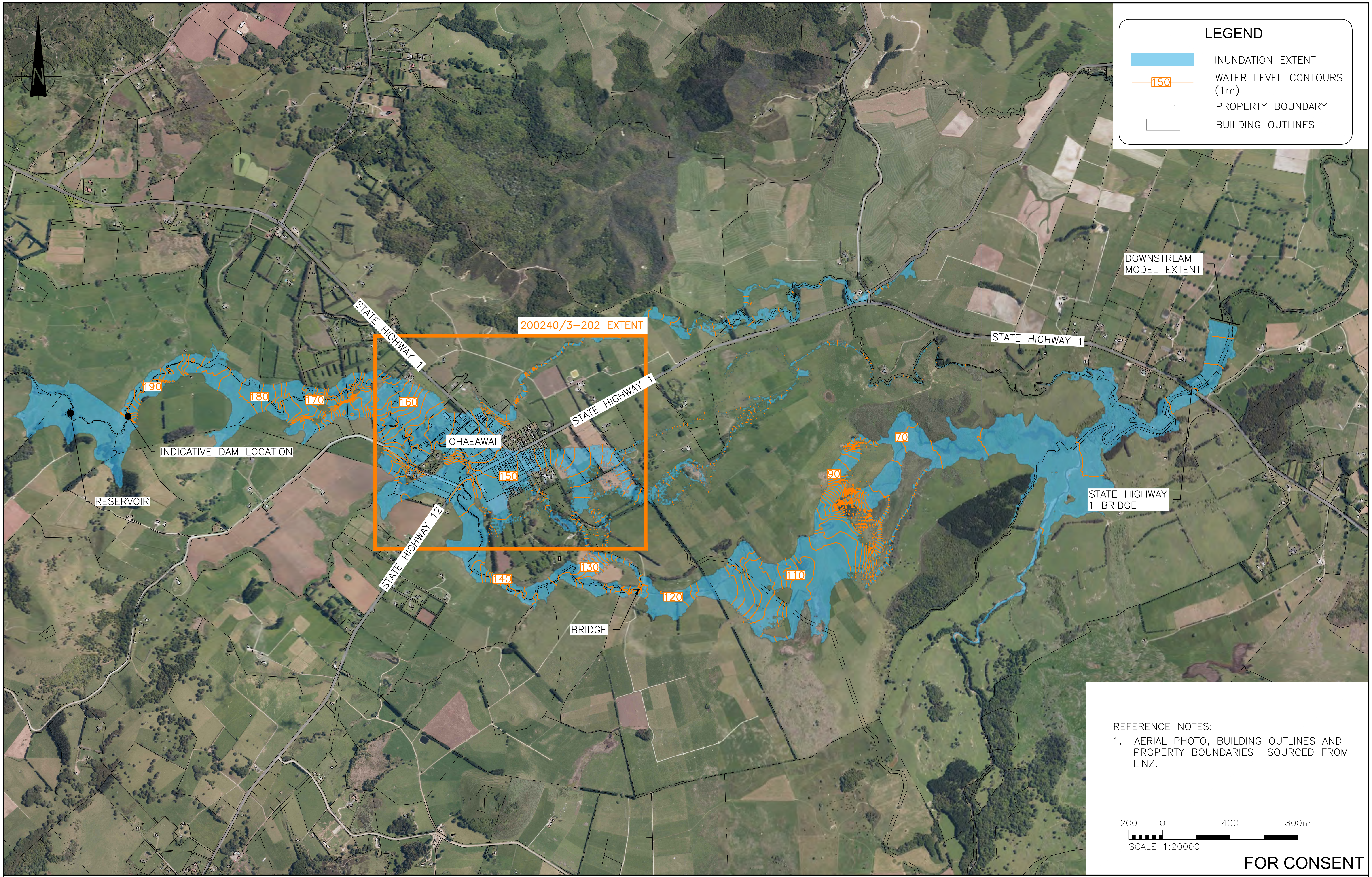
TE TAI TOKERAU WATER TRUST

CLIENT: TE TAI TOKERAU WATER TRUST
ADDRESS: NORTHLAND
PROJECT: TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE: DOWNSTREAM FLOODPLAIN OVERVIEW

CADFILE: 200240_3-200.dwg
SCALE (A3): 1:25000
ORIG. SHEET SIZE: A3
DRAWING No.: 200240/3-200
REV.: 1



FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN GL | DES CHECK GL | DRAWN OOBG | CAD CHECK JM | APPROVED FOR ISSUE D TATE | DATE DRAWN AUG. 2020 | ISSUE DATE 01 / 09 / 20

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CLIENT: TE TAI TOKERAU WATER TRUST

ADDRESS: NORTHLAND

PROJECT: TE RUAOTEHAUHAU WATER STORAGE RESERVOIR

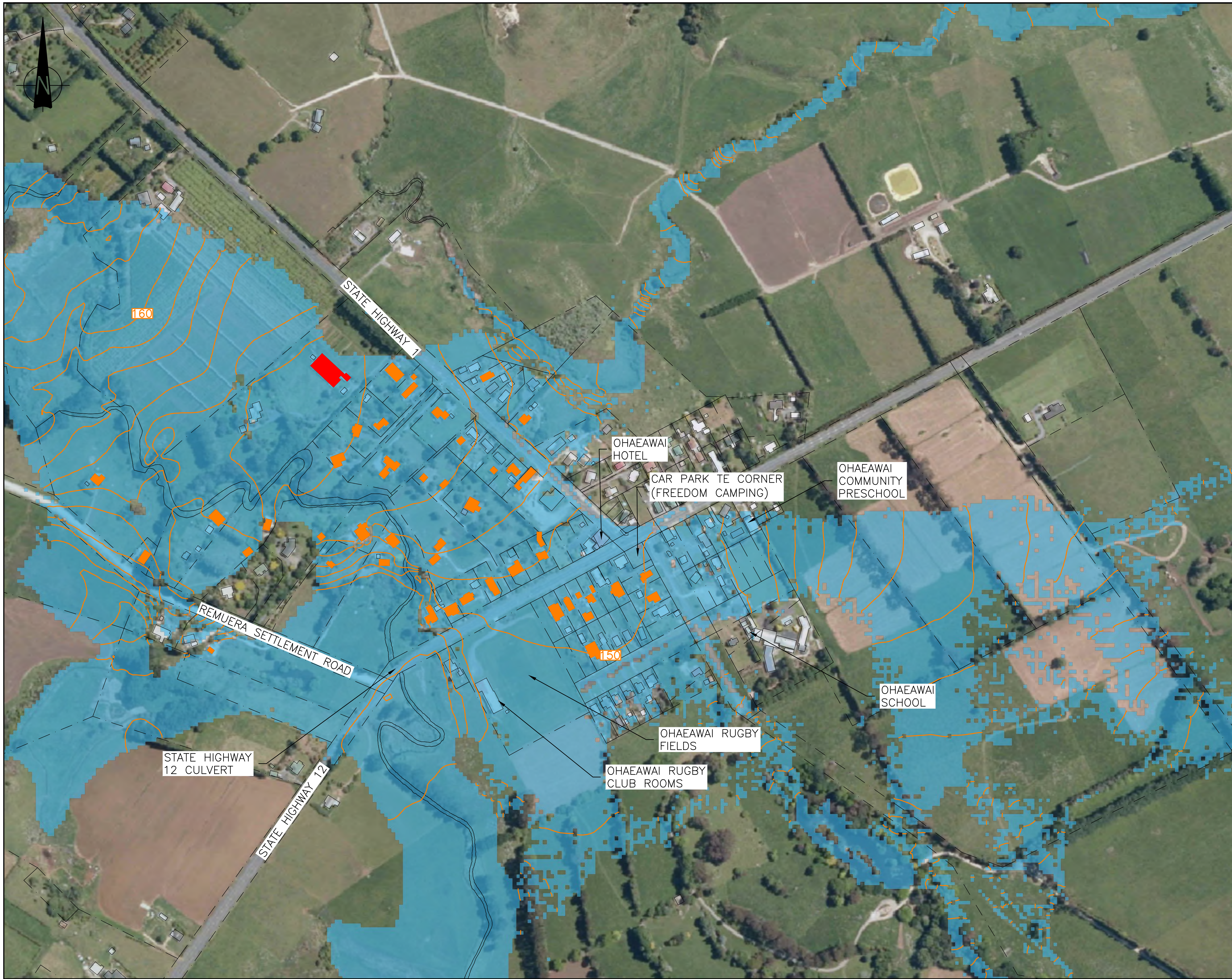
SHEET TITLE: SUNNY DAY BREACH - PEAK LEVELS (FULL AREA)

ACENZ

CADFILE: 200240_3–201.dwg

SCALE (A3): 1:20000 | ORIG. SHEET SIZE: A3

DRAWING No.: 200240/3-201 | REV.: 1



LEGEND

- INUNDATION EXTENT
- WATER LEVEL CONTOURS (1m)
- PROPERTY BOUNDARY
- BUILDING OUTLINES
- RESIDENTIAL HOUSE INUNDATED BY DEPTHS >0.5m
- COMMERCIAL BUILDING INUNDATED BY DEPTHS >0.5m

REFERENCE NOTES:

1. AERIAL PHOTO, BUILDING OUTLINES AND PROPERTY BOUNDARIES SOURCED FROM LINZ.

50 0 100 200m

SCALE 1:5000

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN	DES CHECK	APPROVED FOR ISSUE
GL	GL	D TATE
DRAWN	CAD CHECK	
OOBG	JM	
DATE DRAWN		ISSUE DATE
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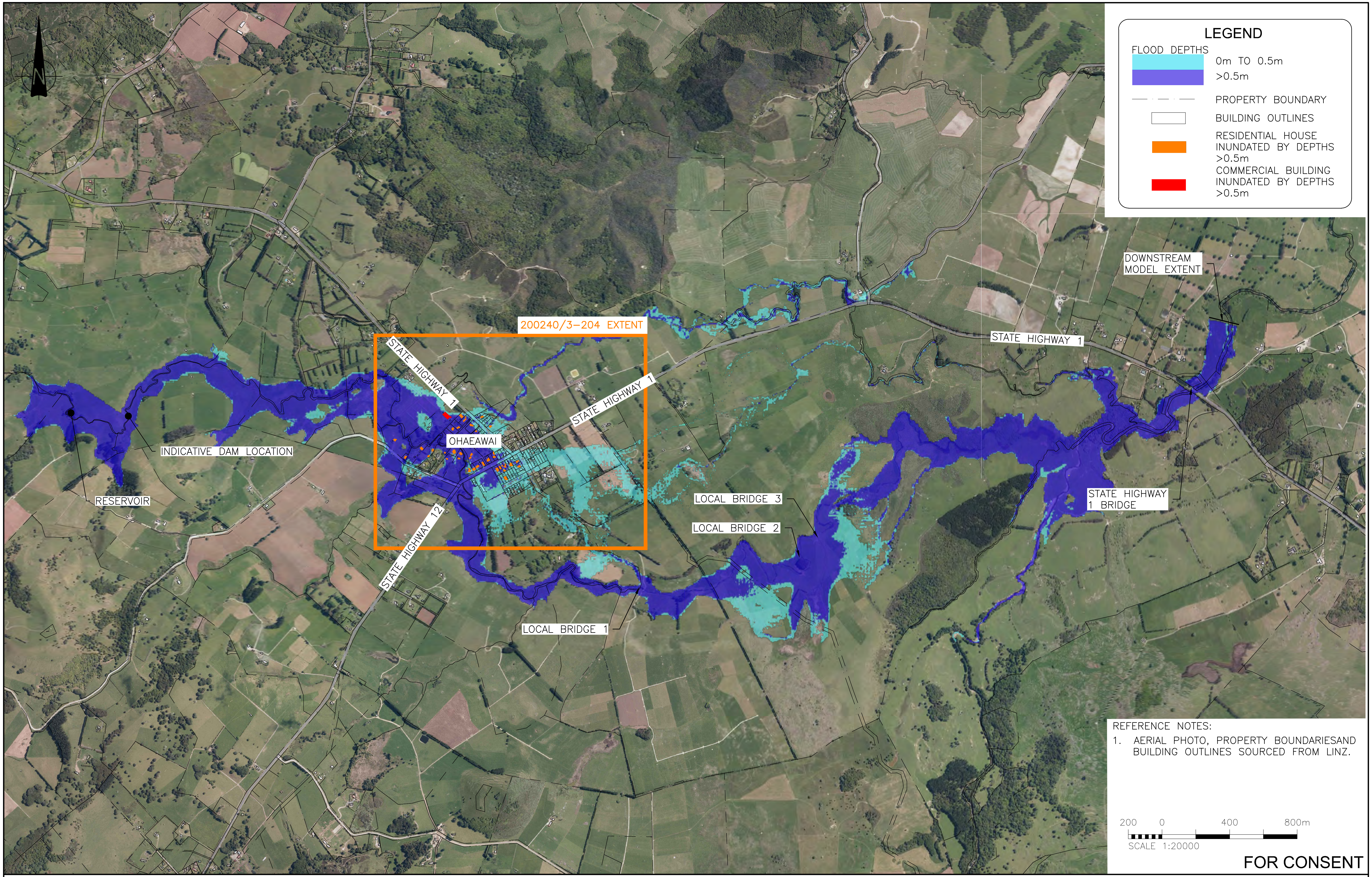
CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	SUNNY DAY BREACH - PEAK LEVELS (INSERT)

CADFILE	200240_3-202.dwg
SCALE (A3)	1:5000
ORIG. SHEET SIZE	A3
DRAWING No.	200240/3-202
REV.	1



FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN	DES CHECK	APPROVED FOR ISSUE
GL	GL	D TATE
DRAWN	CAD CHECK	
OOBG	JM	
DATE DRAWN		ISSUE DATE
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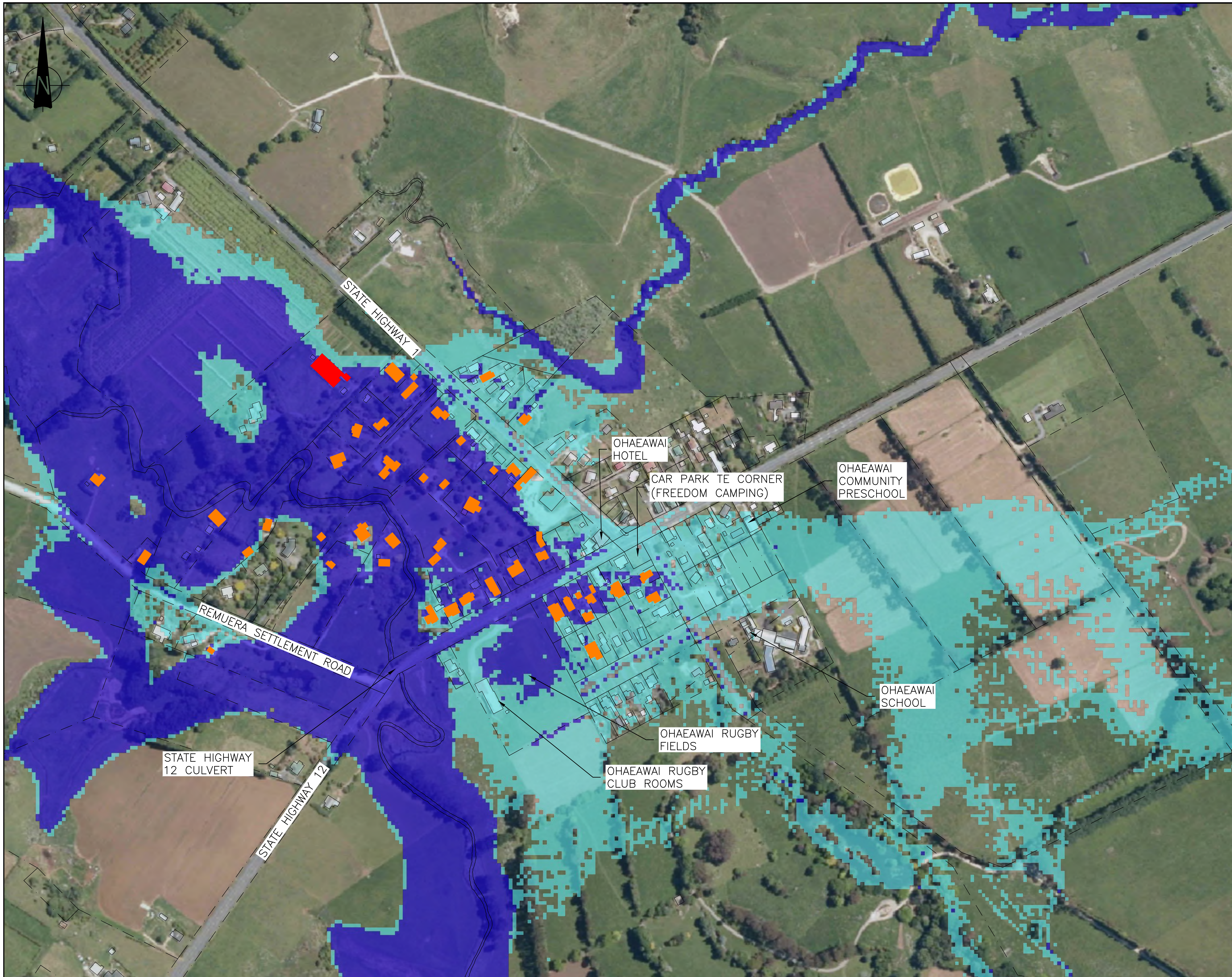
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	SUNNY DAY BREACH - PEAK DEPTHS (FULL AREA)

CADFILE	
200240_3–203.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:20000	A3
DRAWING No.	REV.
200240/3-203	1



LEGEND

FLOOD DEPTHS
- 0m TO 0.5m
- >0.5m
- PROPERTY BOUNDARY
- BUILDING OUTLINES
- RESIDENTIAL HOUSE INUNDATED BY DEPTHS >0.5m
- COMMERCIAL BUILDING INUNDATED BY DEPTHS >0.5m

REFERENCE NOTES:
1. AERIAL PHOTO, PROPERTY BOUNDARIES AND BUILDING OUTLINES SOURCED FROM LINZ.

50 0 100 200m
SCALE 1:5000

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN	DES CHECK	APPROVED FOR ISSUE
GL	GL	D TATE
DRAWN	CAD CHECK	
OOBG	JM	
DATE DRAWN		ISSUE DATE
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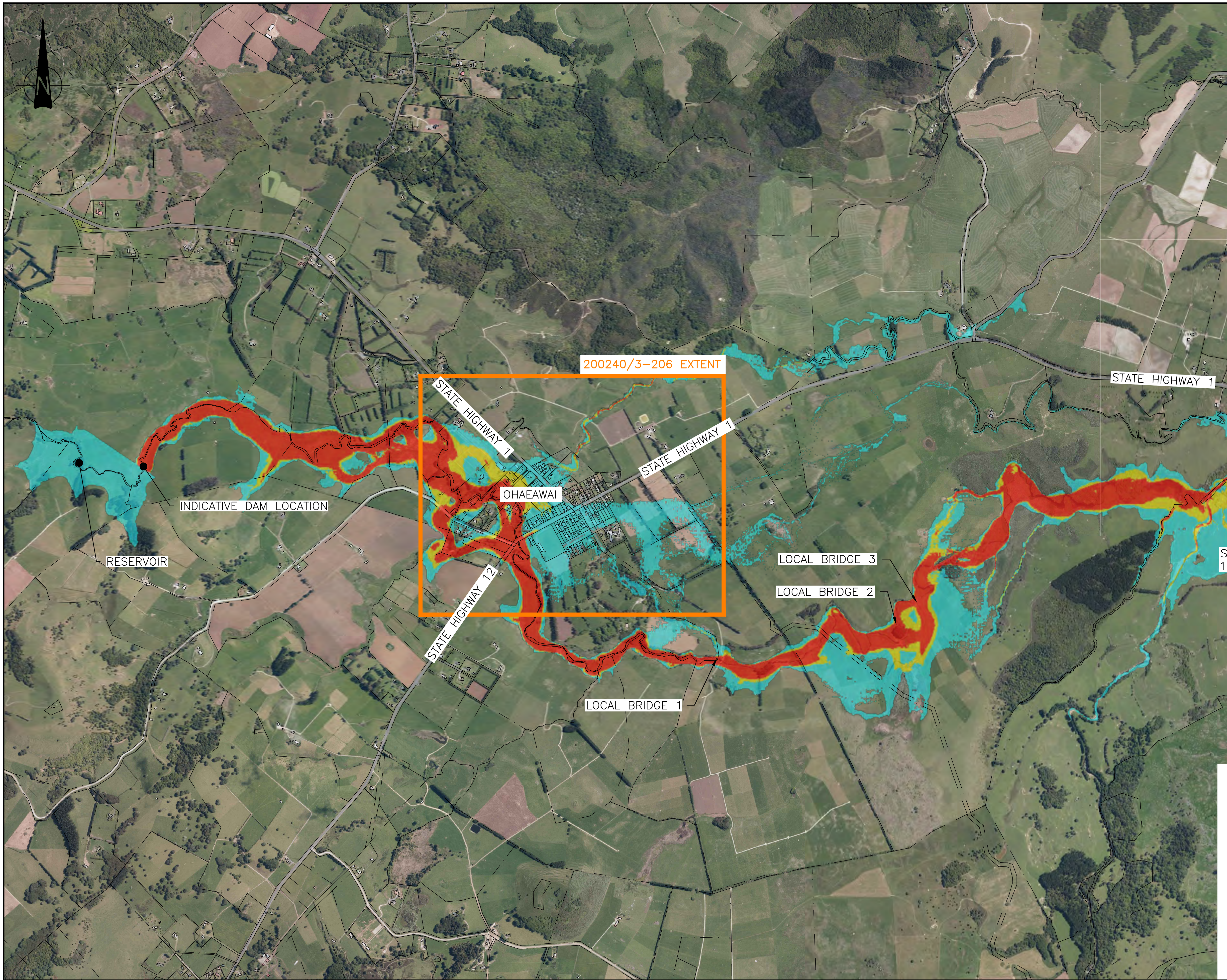
CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	SUNNY DAY BREACH - PEAK DEPTH (INSERT)

CADFILE	200240_3-204.dwg
SCALE (A3)	1:5000
ORIG. SHEET SIZE	A3
DRAWING No.	200240/3-204
REV.	1



LEGEND

FLOOD DEPTH VELOCITY PRODUCT (DxV)

- $0m^2/s$ TO $1.5m^2/s$
- $1.5m^2/s$ TO $3m^2/s$
- $>3m^2/s$
- PROPERTY BOUNDARY
- BUILDING OUTLINES
- RESIDENTIAL HOUSE AFFECTED BY $1.5m^2/s<DxV<3m^2/s$
- RESIDENTIAL HOUSE AFFECTED BY $DxV>3m^2/s$
- COMMERCIAL BUILDING WITH DxV $>1.5m^2/s$

REFERENCE NOTES:

1. AERIAL PHOTO, BUILDING OUTLINES AND PROPERTY BOUNDARIES SOURCED FROM LINZ.

200 0 400 800m

SCALE 1:20000

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	D TATE
OOBG	JM	
DATE DRAWN		ISSUE DATE
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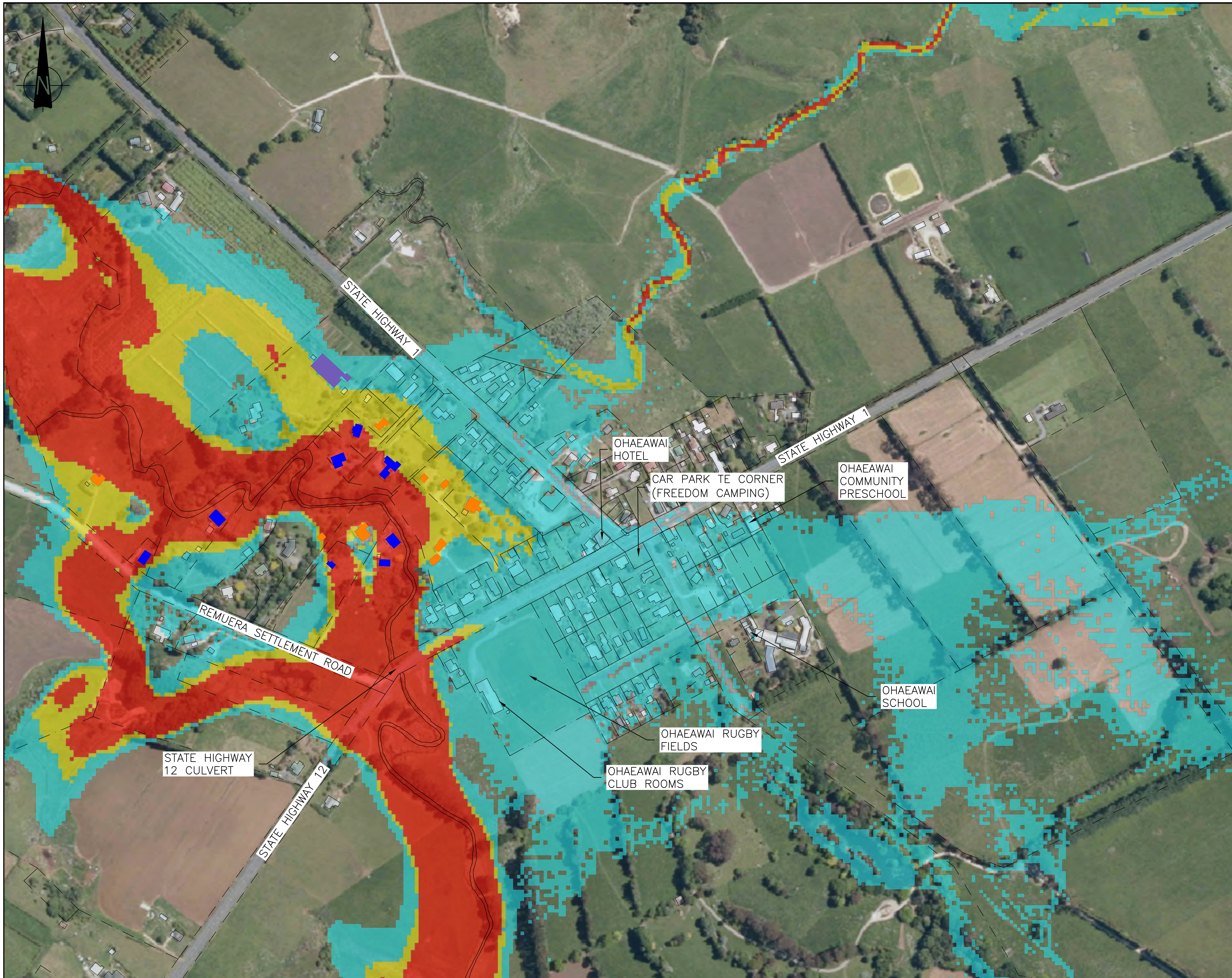
CLIENT: TE TAI TOKERAU WATER TRUST
ADDRESS: NORTHLAND
PROJECT: TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE: SUNNY DAY BREACH - PEAK DEPTH x VELOCITY (FULL AREA)

CADFILE: 200240_3–205.dwg
SCALE (A3): 1:20000
ORIG. SHEET SIZE: A3
DRAWING No.: 200240/3-205
REV.: 1



LEGEND

FLOOD DEPTH VELOCITY PRODUCT (DxV)

- $0m^2/s$ TO $1.5m^2/s$
- $1.5m^2/s$ TO $3m^2/s$
- $>3m^2/s$
- PROPERTY BOUNDARY
- BUILDING OUTLINES
- RESIDENTIAL HOUSE AFFECTED BY $1.5m^2/s<DxV<3m^2/s$
- RESIDENTIAL HOUSE AFFECTED BY $DxV>3m^2/s$
- COMMERCIAL BUILDING WITH $DxV >1.5m^2/s$

REFERENCE NOTES:

1. AERIAL PHOTO, BUILDING OUTLINES AND PROPERTY BOUNDARIES SOURCED FROM LINZ.

50 0 100 200m
SCALE 1:5000

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
OOBG	JM	
DATE DRAWN		ISSUE DATE
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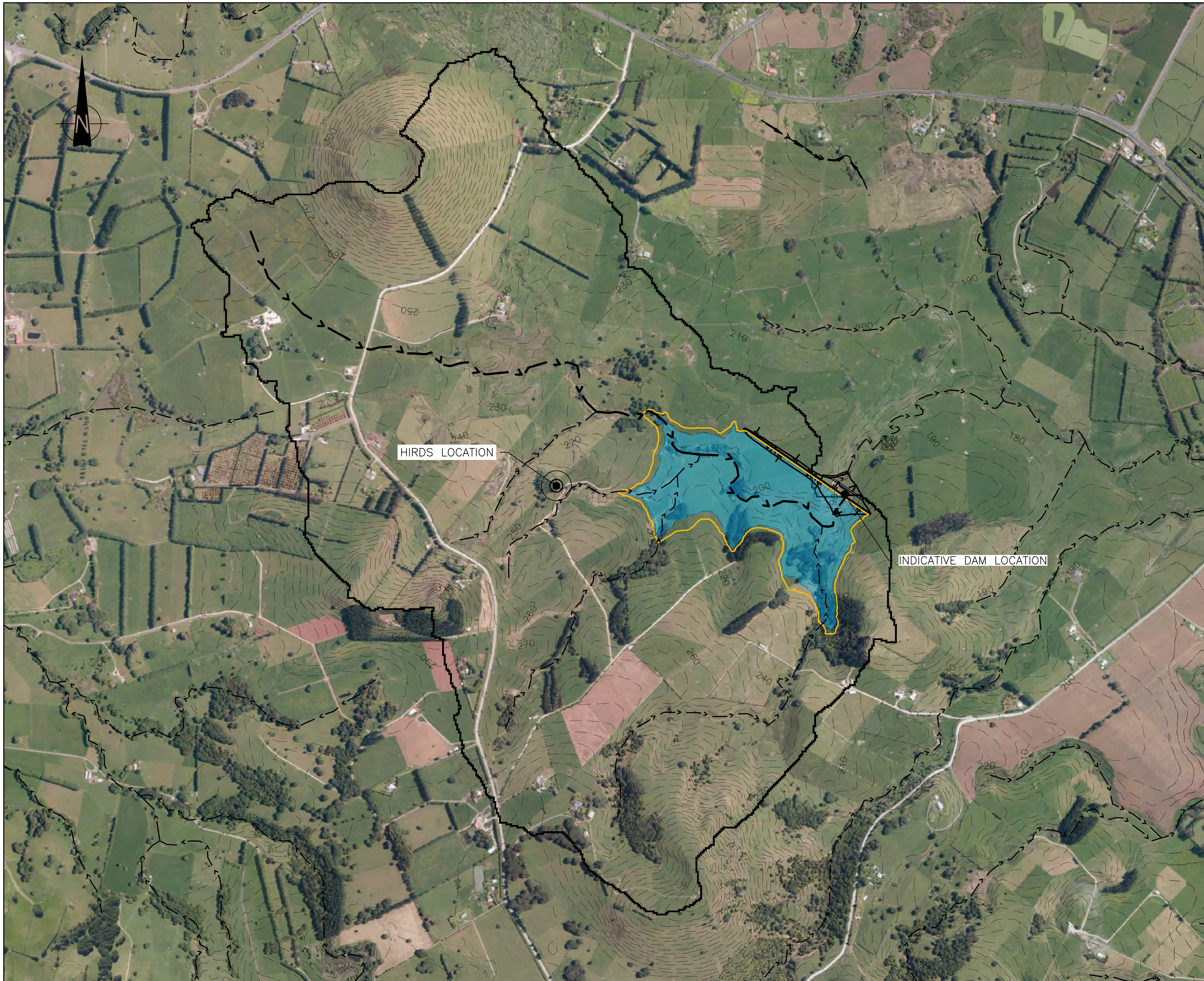
CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	SUNNY DAY BREACH - PEAK DEPTH x VELOCITY (INSERT)

ACENZ

CADFILE	200240_3-206.dwg
SCALE (A3)	1:5000
ORIG. SHEET SIZE	A3
DRAWING No.	200240/3-206
REV.	1



LEGEND

- CATCHMENT BOUNDARY
- LONGEST FLOW PATH
- INDICATIVE WATERCOURSE
- 260 5m CONTOURS
- INDICATIVE RESERVOIR EXTENT

REFERENCE NOTES:

1. AERIAL PHOTO SOURCED FROM LINZ (2014–2016)

SCALE 1: 12000

0 120 240 360 480 720 (m)

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	MP

DESIGN	DES CHECK	APPROVED FOR ISSUE
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MP	JM	
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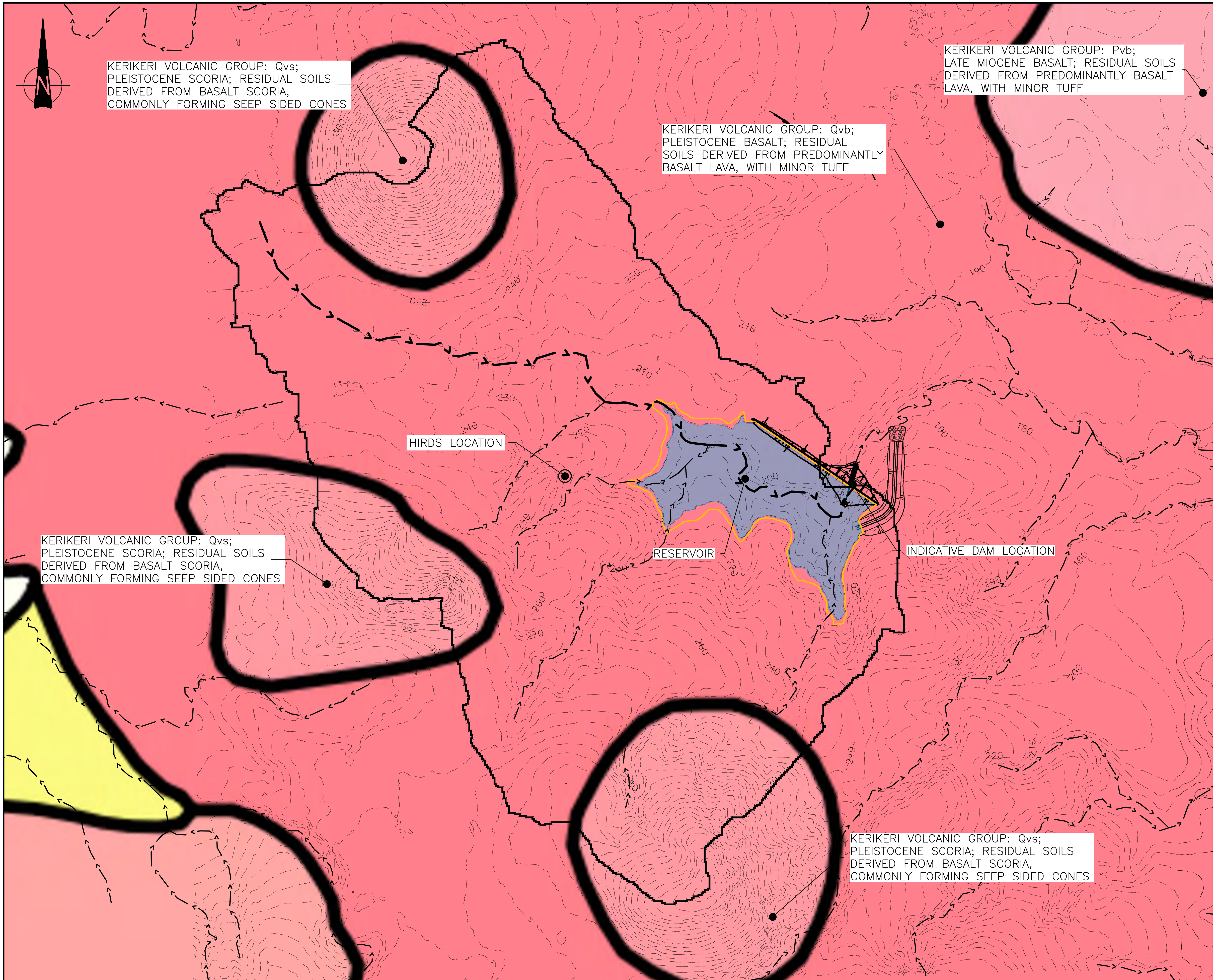
CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	CATCHMENT PLAN - AERIAL BACKGROUND

ACENZ

ISO CERTIFIED GCS

CADFILE	200240_3–210.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:12000	A3
DRAWING No.	REV.
200240/3-210	1



LEGEND

- CATCHMENT BOUNDARY
- LONGEST FLOW PATH
- INDICATIVE WATERCOURSE
- 260 — 5m CONTOURS
- INDICATIVE RESERVOIR EXTENT

REFERENCE NOTES:

1. GEOLOGICAL MAP SOURCED FROM GNS QMAP (2014).

SCALE 1: 12000

0 120 240 360 480 720 (m)

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	OBG

DESIGN	DES CHECK	APPROVED FOR ISSUE
GL	GL	D TATE
DRAWN	CAD CHECK	
OOBG	JM	
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ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	CATCHMENT PLAN - GEOLOGY BACKGROUND

CADFILE	
200240_3-211.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:12000	A3
DRAWING No.	REV.
200240/3-211	1

LEGEND

	INUNDATION EXTENT
206	WATER LEVEL CONTOURS (1m)
+206.78	WATER LEVEL SPOT HEIGHT
	PROPOSED GROUND LEVEL CONTOURS (1m)
	INDICATIVE EARTHWORKS EXTENT

REFERENCE NOTES:

1. AERIAL PHOTO SOURCED FROM LINZ.
2. GROUND CONTOURS ARE EXTRACTED FROM 2017 LIDAR AND HAVE BEEN MODIFIED WITHIN THE PROPOSED EARTHWORKS EXTENT.

20 0 40 80m

SCALE 1:2000

FOR CONSENT

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1		FOR CONSENT	GL

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
GL	JM	
DATE DRAWN		ISSUE DATE
AUG. 2020		01 / 09 / 20

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CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	PMF DESIGN FLOOD - PEAK LEVELS

ACENZ

ISO CERTIFIED 9001 GCS

CADFILE	
200240_3–220.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:2000	A3
DRAWING No.	REV.
200240/3-220	1

LEGEND

- 0m TO 0.5m
- 0.5m TO 1m
- 1m TO 1.5m
- 1.5 TO 2m
- >2m
- PROPOSED GROUND LEVEL CONTOURS (1m)
- INDICATIVE EARTHWORKS EXTENT

REFERENCE NOTES:

1. AERIAL PHOTO SOURCED FROM LINZ.
2. GROUND CONTOURS ARE EXTRACTED FROM 2017 LIDAR AND HAVE BEEN MODIFIED WITHIN THE PROPOSED EARTHWORKS EXTENT.

20 0 40 80m

SCALE 1:2000

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	GL

DESIGN	DES CHECK	APPROVED FOR ISSUE
HA	GL	D TATE
DRAWN	CAD CHECK	
GL	JM	
DATE DRAWN		ISSUE DATE
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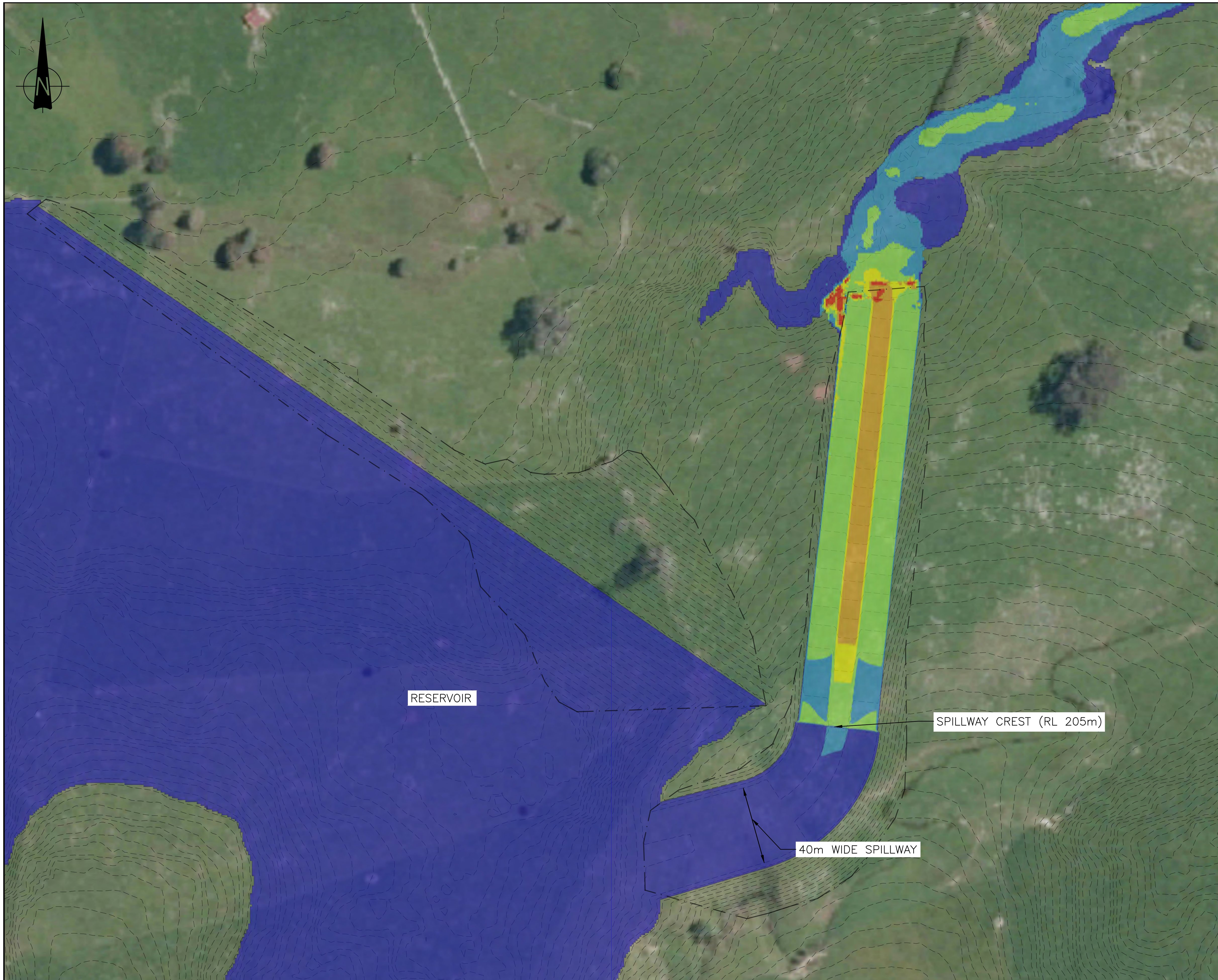
CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	PMF DESIGN FLOOD - PEAK DEPTHS

ACENZ

ISO CERTIFIED GCS

CADFILE	200240_3-221.dwg
SCALE (A3)	1:2000
ORIG. SHEET SIZE	A3
DRAWING No.	200240/3-221
REV.	1



LEGEND

- 0m/s TO 2m/s
- 2m/s TO 4m/s
- 4m/s TO 6m/s
- 6m/s TO 8m/s
- 8m/s TO 10m/s
- 10m/s TO 12m/s
- PROPOSED & EXISTING GROUND LEVEL CONTOURS (1m)
- INDICATIVE EARTHWORKS EXTENT

REFERENCE NOTES:

1. AERIAL PHOTO SOURCED FROM LINZ.
2. GROUND CONTOURS ARE EXTRACTED FROM 2017 LIDAR AND HAVE BEEN MODIFIED WITHIN THE PROPOSED EARTHWORKS EXTENT.

20 0 40 80m

SCALE 1:2000

FOR CONSENT

REV	DATE	ISSUE	BY
1		FOR CONSENT	GL

DESIGN	DES CHECK	APPROVED FOR ISSUE
HA	GL	D TATE
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DATE DRAWN		ISSUE DATE
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CLIENT	TE TAI TOKERAU WATER TRUST
ADDRESS	NORTHLAND
PROJECT	TE RUAOTEHAUHAU WATER STORAGE RESERVOIR
SHEET TITLE	PMF DESIGN FLOOD - PEAK VELOCITIES

ACENZ

ISO CERTIFIED GCS

CADFILE	
200240_3-222.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:2000	A3
DRAWING No.	REV.
200240/3-222	1