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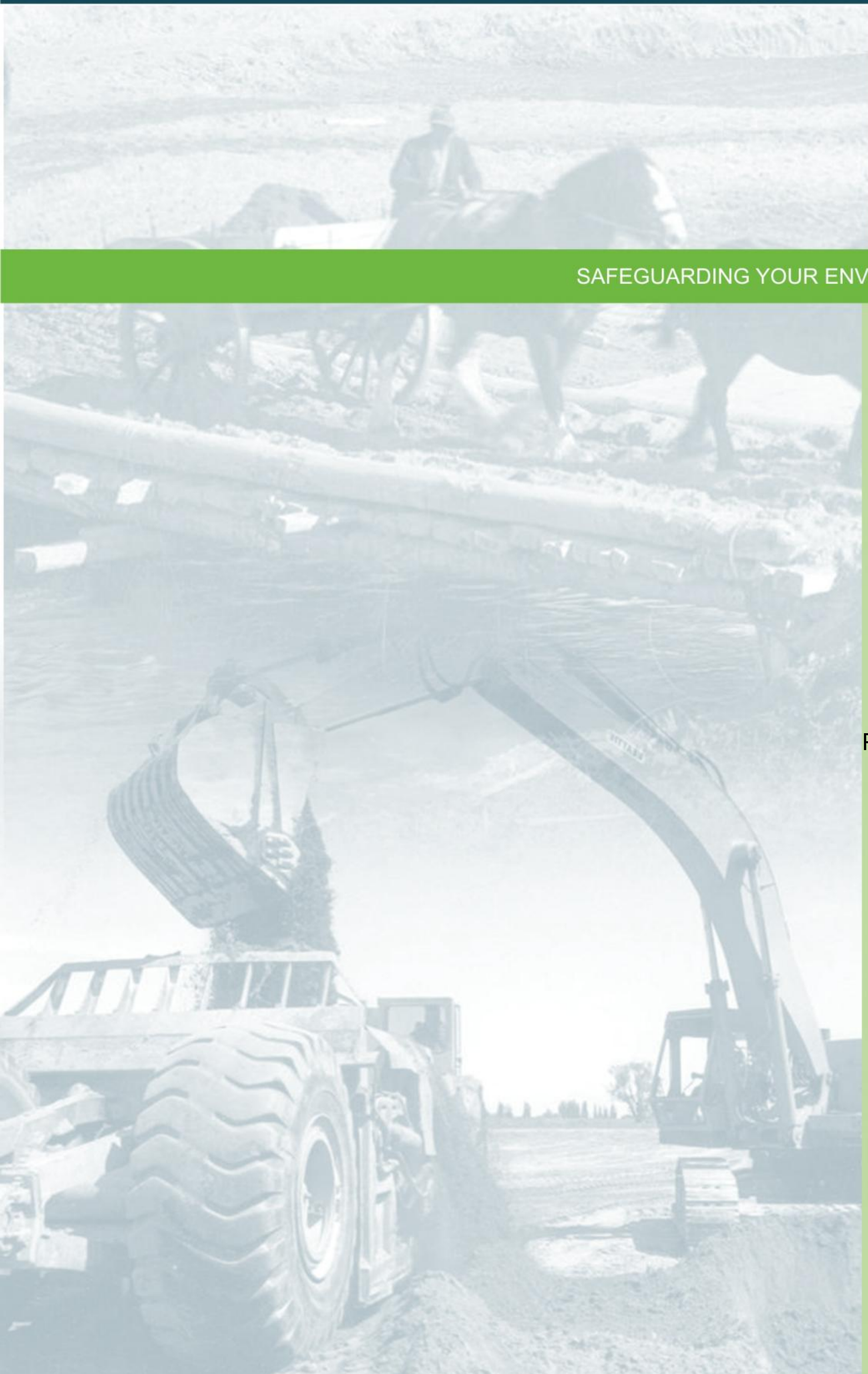

Ruataniwha Water Storage Scheme: Dam Break Analysis

Prepared for Hawke's Bay Regional Investment Company Ltd.

Final: Ref. A13
May, 2013

May 2013
WI 13/03
HBRC Plan Number 4484
Supersedes HBRC Plan 4472

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SAFEGUARDING YOUR ENVIRONMENT + KAITIAKI TUKU IHO

Asset Management Group
Engineering Section
Technical Report

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Prepared for: Hawke's Bay Regional Investment Company Ltd. (HBRIC Ltd.)

Ruataniwha Water Storage Scheme

Dam Break Analysis

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Ruataniwha Water Storage Scheme
Dam Break Analysis

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Ruataniwha Water Storage Scheme
Dam Break Analysis

1 Executive Summary

This report summarises the results from a Dam Break Analysis for the proposed Ruataniwha Water Storage Scheme, prepared for the Hawke's Bay Regional Investment Company Ltd. Background information on the Scheme can be found in the Project Description by Tonkin & Taylor (*Ruataniwha Water Storage Scheme: Project Description,* May 2013*)*.

The Ruataniwha Water Storage Scheme consists of a large water storage reservoir created by a proposed dam on the Makaroro River, as well as associated works used to distribute the water to downstream locations.

Potential Project Effects

This report provides analysis and results showing the consequences of the failure of the Dam during its operational phase. It is emphasised here that the dam break analysis is entirely hypothetical and divorced from the actual probability of a dam failure occurring, and is not instigated by any particular concern with the conditions at the dam site or the proposed concept in the construction of the dam.

The dam break analysis is used to assist in determining the Potential Impact Category (PIC) of the dam, based on an assessment of the potential downstream effects in terms of potential loss of life, as well as damage to infrastructure in the event of a dam failure. The results of this analysis indicate the proposed dam will be a **HIGH** PIC dam.

Failure of the main dam after completion of construction, and assuming the reservoir is full, would likely result in significant damage to infrastructure (bridges, roads, stopbanks, and sewage treatment plants), environmental damage to the river corridor and surrounding floodplain, and involve a population-at-risk of approximately 1000 people.

Assessments Undertaken

The assessment of the potential downstream effects of a dam break consists of three parts:

1. Determination of the dam breach discharge hydrograph,
2. Determination of the extent and timing of the flood wave,
3. Assessment of potential impact category (PIC).

This report outlines the method used in the analysis, and then presents the results with maps showing the timing and extent of the flood wave as it travels down the river system.

Results of Assessments

Results indicate the flood wave would be contained in the incised river sections of the Makaroro and Waipawa Rivers downstream of the dam until around SH50.

Downstream of SH50 there would be significant overflows on the left and right banks of the Waipawa River.

On the left bank of the Waipawa River, downstream of SH50, the overland flow spreads out over a wide area and travels towards the Mangaonuku Stream, at which point it is confined and flows back into the Waipawa River.

On the right bank of the Waipawa River, downstream of SH50, the overland flow travels to the Kahahakuri Stream and the Tukituki River, and then overtops the stopbanks on the Tukituki River around Waipukurau.

Downstream of the Waipawa/Mangaonuku confluence, the Waipawa River narrows again, forcing all the water through a confined section, then the flood wave overtops the stopbanks at the town of Waipawa. Water depths in the area of Bibby Street near the Waipawa sewage treatment works (oxidation pond) would likely be in the order of 3 m to 5 m deep.

Another overflow occurs downstream of the town of Waipawa, just after the confluence of the Waipawa and Tukituki Rivers, down an old course of the Waipawa River to the Papanui Stream. The Pukehou and Te Aute swamp areas become inundated in this scenario, due to their low lying nature.

Downstream of the confluence with the Papanui Stream, the flood wave is fully contained within the Middle Tukituki River channel.

At the mouth of the Lower Tukituki River, there would likely be high water levels in Grange Creek near Haumoana, with similar levels to those from a 50 year return period event in the Tukituki River.

The scenario analysed for the PIC determination produced a peak discharge of around 45,000 m^3/s at the dam site. Due to the topography of the river channel, the flow is fairly quickly attenuated. However, the results indicate a peak flow of around 10,400 m^3/s is still likely in the Waipawa River near the town of Waipawa. This is an area with stopbank protection up to the 100 year return period event, which has an estimated design discharge of 1350 m^3/s, i.e. the flood wave has a peak discharge that is roughly 8 times the 100 year discharge at this location.

The peak of the flood wave takes approximately 13 hours to travel from the dam site to the coast, a distance of about 116 km. There would likely be a minimum of 2 to 3 hours warning time between the initiation of failure and the time when the population and infrastructure of Waipawa and Waipukurau were at risk.

Suggested Approach for Potential Effects Identified

Due to these potential risks, along with the size of the main dam, the PIC of the main dam is determined to be HIGH. .The primary mitigation of the potential effects from an unlikely dam break event is the adoption of the highest standards for design, construction and operation to ensure that the probability of failure is extremely small related to the degree to which the potential impact is high. In addition to minimum standards for design, a High PIC dam will require an appropriate Dam Safety Assurance Programme (DSAP) under New Zealand‘s Dam Safety Regulations. Part of the DSAP will be an Emergency Action Plan (EAP) that will detail the actions that the owner, operations personnel and relevant Government and Local Authorities should take if an incident or emergency develops that threatens the safety of the dam. Both the DSAP and EAP will be required prior to commissioning of the dam.

2 Scope of Study

The hypothetical nature of a dam break analysis tends to present a skewed view of the risks associated with the dam. The dam is required to be designed, constructed and operated to ensure that the probability of failure is extremely small related to the degree to which the potential impact is high. In this particular case, the dam has been designed to survive the Probable Maximum Flood (PMF), as well as the Maximum Credible Earthquake. Significant investigation and analysis have been completed to ensure that these design standards have been achieved. Nevertheless, a dam break analysis is included in order to assess the downstream hazard potential, which in turn confirms the setting of standards to adopt for dam design, construction and operation.

Information generated from the dam break study, such as inundation maps and flood wave travel times is typically incorporated in an Emergency Action Plan for the dam.

For this report, the assessment of the effects of a dam break consists of three parts:

4. Determination of the dam breach discharge hydrograph,
5. Determination of the extent and timing of the flood wave,
6. Assessment of potential impact category (PIC).

Items 1 and 2 are determined with the use of hydrodynamic computer models, while item 3 is best determined with the aid of geographical information systems, which include information such as population and location of infrastructure.

2.1 Dam Break Scenarios

Dam break analyses usually involve two classes of failure scenarios, commonly referred to as ‘sunny day failure’ and ‘rainy day failure’. Both of these types of failures are examined in this study. The failure scenarios generally show the worst case scenarios when extremely catastrophic events occur.

2.1.1 Sunny Day Failure

The sunny day failure examines a scenario when the reservoir is full up to the normal spillway level, and there is no significant inflow into the reservoir, or in any parts of the adjoining catchments. While the actual cause of failure is not examined in this report, the breach formation time and the consequences of the failure are examined. The breach development time can be of significance to the peak flow, however, in the range of severe cases examined in this study, the consequences of all failures are generally similar. The focus is on the plausible worst case flood wave that may travel downstream, and the damage caused by such a wave.

The sunny day failure is important to consider, as the incremental damage caused by the dam failure is likely to be significant, since there were no other flood related processes occurring which would have already been causing damage, or forewarning to enable evacuation out of harm’s way. The sunny day failure is normally used to determine the Potential Impact Category (PIC) rating for a dam, since the incremental losses from a sunny day failure are normally greater than the incremental losses associated with a rainy day failure.

2.1.2 Rainy Day Failure

The rainy day failure examines a scenario when the reservoir is full, and there is an extreme inflow into the reservoir, as well as in the adjoining downstream catchments. For the proposed dam, there are two spillways (lower level and higher level), which have been designed to accommodate the probable maximum flood that may occur in the Makaroro catchment. These spillways provide a very large factor of safety for the dam, which results in a very low probability of failure due to overtopping. Despite this factor of safety being incorporated in the dam design, there may be a possibility that some unforeseen event leads to the dam failure occurring at a time when there is an extreme inflow to the dam reservoir area. The assessment is a hypothetical one which focuses on the potential downstream consequences, and not on the actual likelihood of failure.

The rainy day failure quite often leads to lesser incremental damage than the sunny day failure, since there may already be flood damage occurring from the extreme event which caused the dam failure, as well as considerably more forewarning and opportunity for evacuation. The rainy day failure results are quite often used in an Emergency Action Plan, since this failure scenario provides an indication of the largest extent of inundation likely from a dam failure for the purposes of warning and evacuation in a flood situation. A sunny day failure inundation map may also be included in the Emergency Action Plan for emergency response planning in an imminent sunny day failure.

3 Background Details

The proposed main dam is located on the Makaroro River in Central Hawke’s Bay (see Figure 1).

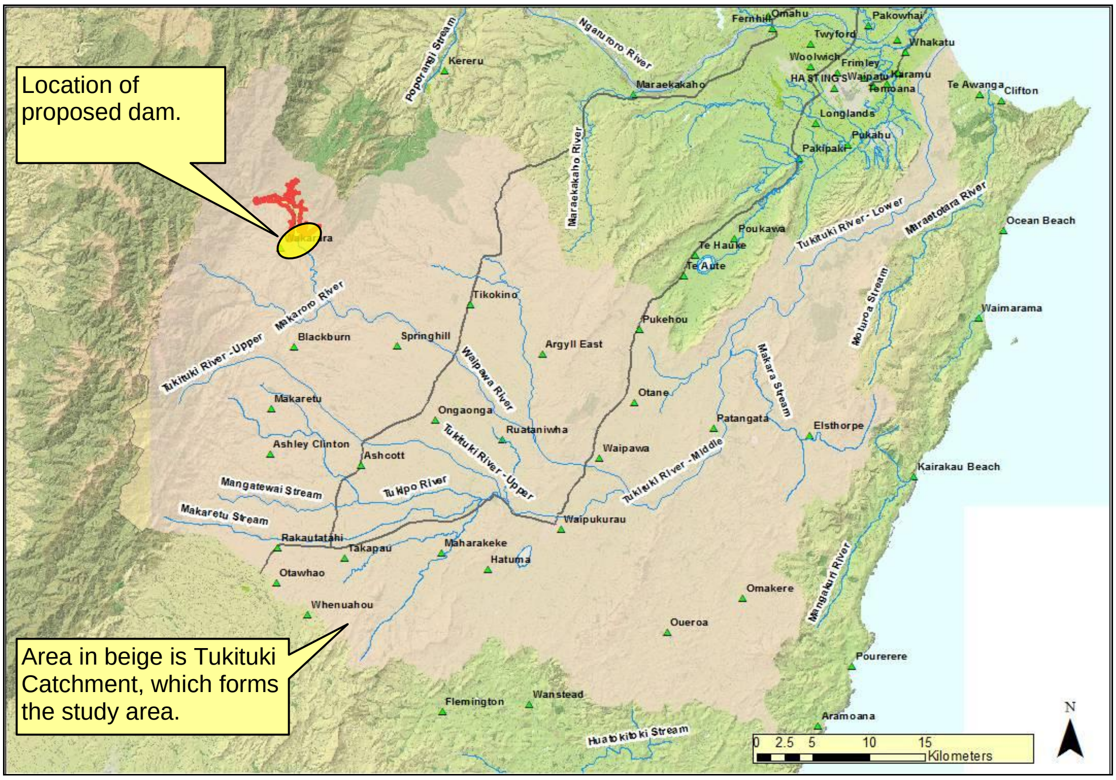

Figure 1: Location of proposed dam and outline of study area

A set of hydrologic and hydrodynamic computer models have been created for the catchment, the dam area, and the affected downstream area from the dam location to the coast, in order to assess possible flooding extents in the event of a dam-break failure. The main focus rivers are the Makaroro River, Waipawa River, and Tukituki River. Other rivers which flow into these main rivers are also included in the modelling, to allow scenarios to be examined where the flood wave from a failure coincides with high flows from surrounding catchments.

The proposed dam is approximately 83 m high at the river's deepest point, approximately 500 m long, and has a crest width of 8 m. The dam is proposed to be a Concrete Faced Rockfill Dam (CFRD), which consists of bulk coarse rockfill material, with a concrete face slab on the upstream side.

The general arrangements and a cross section of the dam are shown in Figure 2 and Figure 3 respectively (Tonkin & Taylor, 2013a).

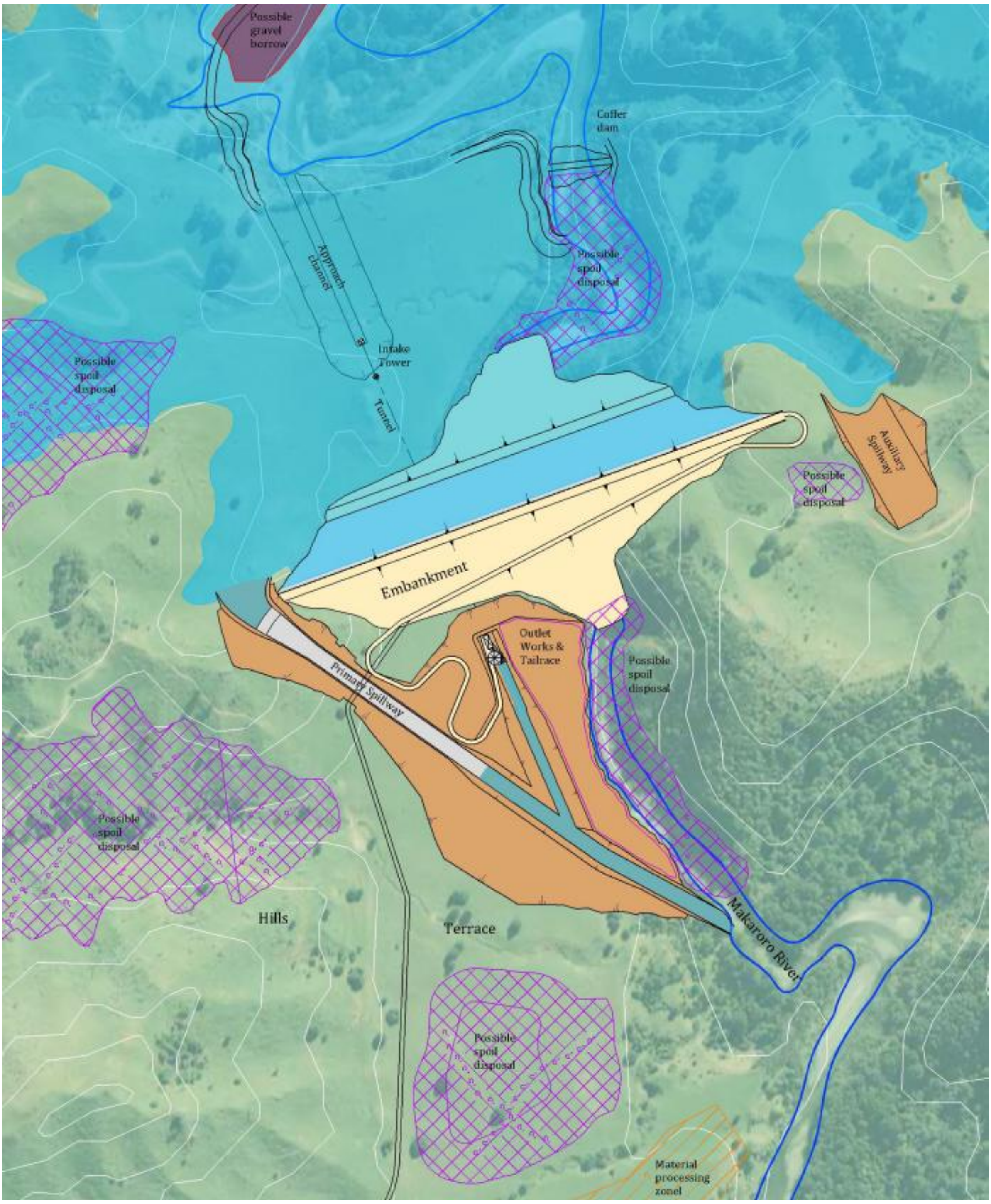

Figure 2: Layout of proposed dam

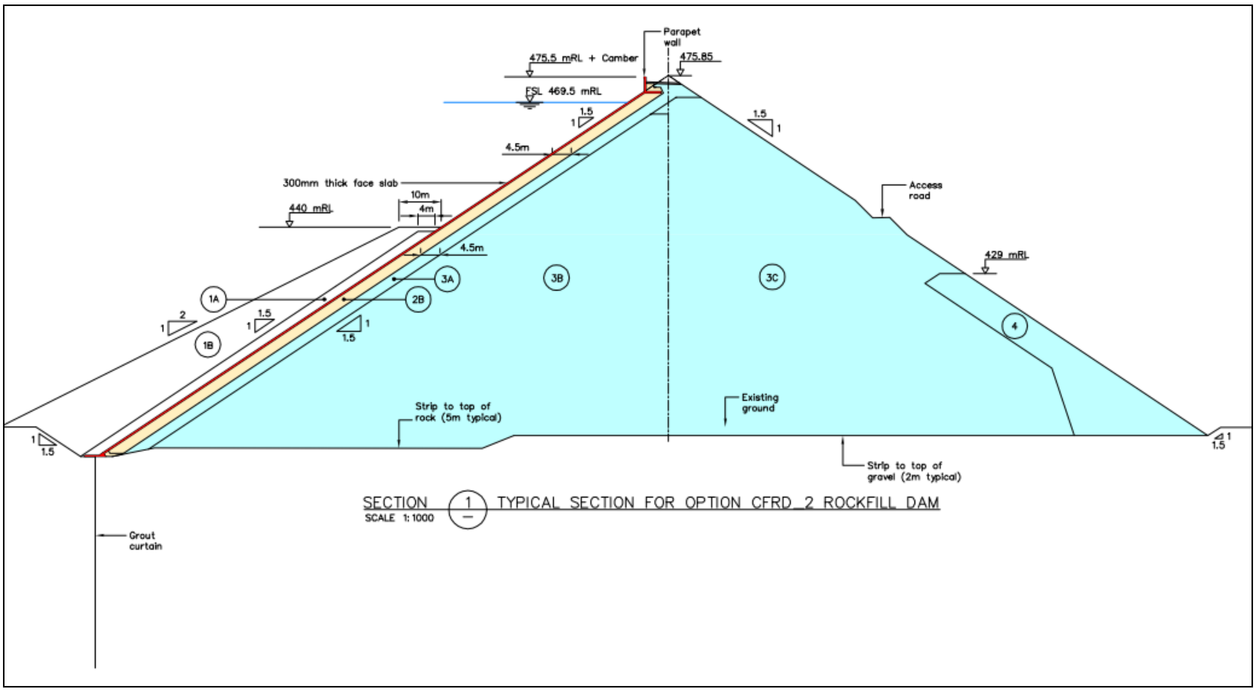

Figure 3: Cross section of proposed dam

The impoundment area behind the dam will store approximately 90,000,000 m^3 at the spillway level, and approximately 115,000,000 m^3 at the design flood level. The reservoir behind the dam is approximately 7 km long. The dam break analysis is concerned with the rate at which this water in the reservoir may escape, in the event of a failure of the dam. The reservoir area for the dam is shown in Figure 4, and the storage elevation curve in Figure 5.

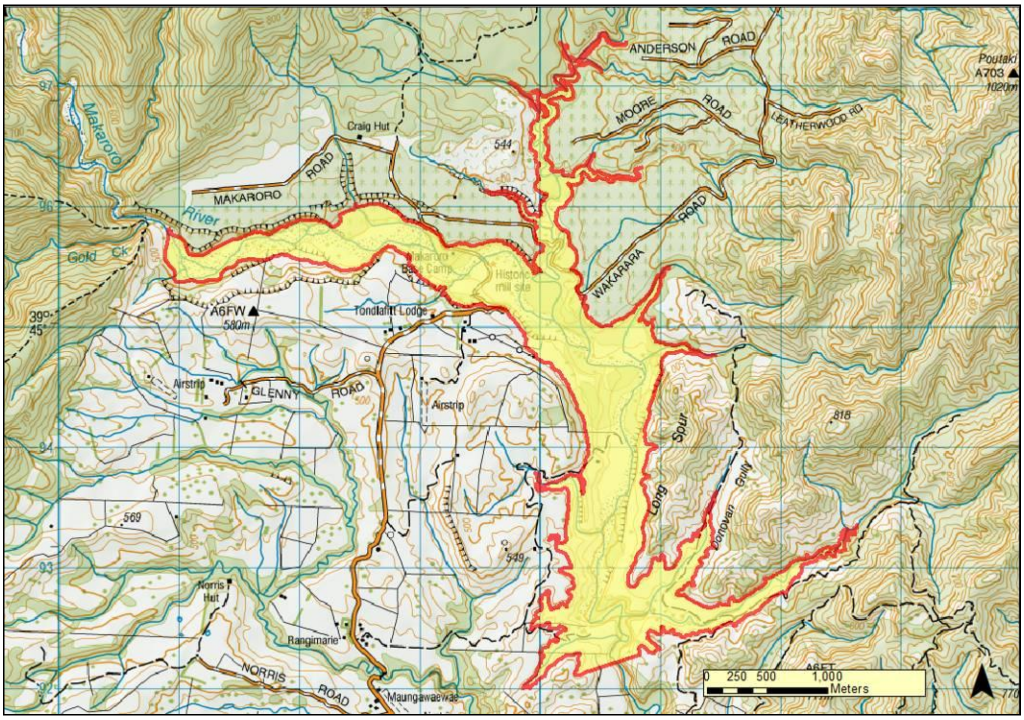

Figure 4: Reservoir for proposed dam

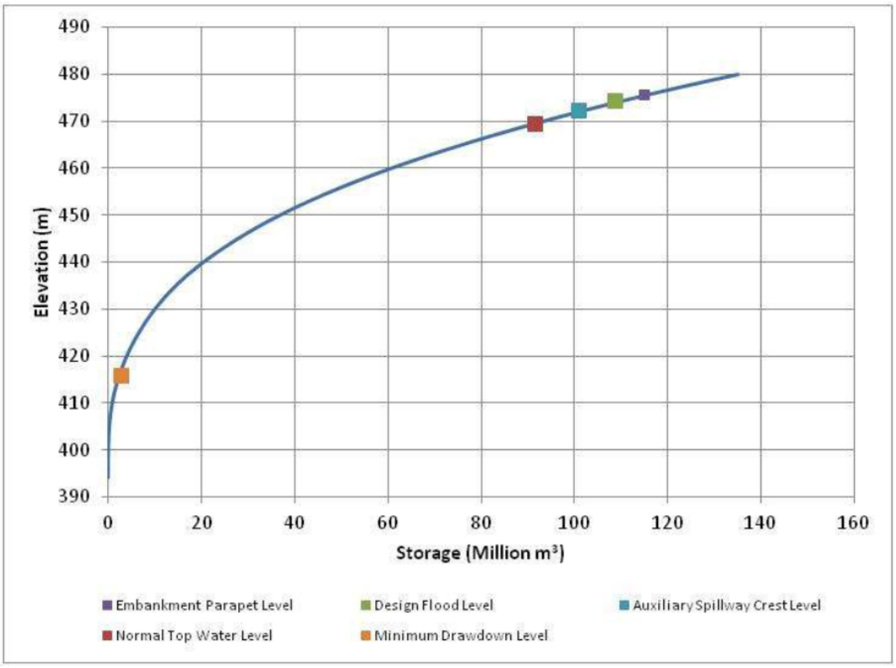

Figure 5: Storage elevation curve for proposed dam

4 Determination of Dam Breach Parameters

There is a significant amount of analysis done on actual historic dam break failures which provides a basis to compare the possible discharges from a breach of the proposed dam. A comprehensive analysis done by Wahl (*Prediction of Embankment Dam Breach Parameters*, 1998) provides several guidelines and empirical methods to assist in determining various breach parameters, which in turn are used to develop a realistic breach discharge. These empirical methods were used in conjunction with the topographical features at the dam site to estimate the possible breach discharge.

A cross section of the river at the dam location is shown in Figure 6, along with several parameters used in the dam breach scenario.

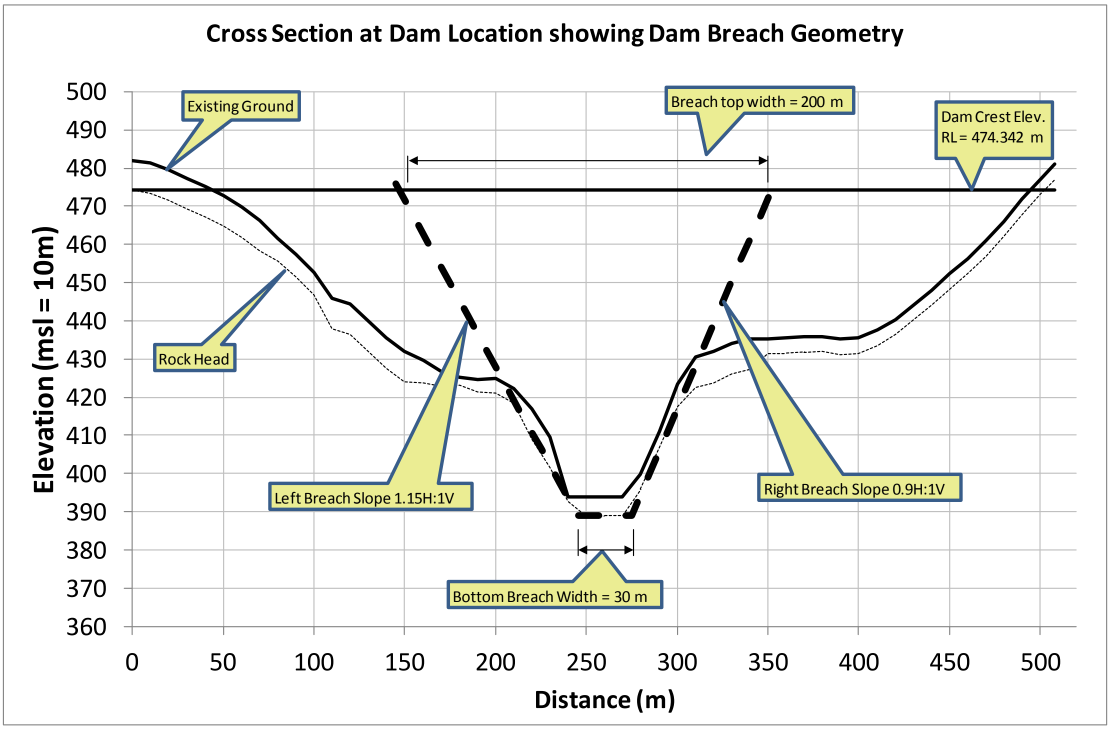

Figure 6: Cross-Section at dam location showing dam breach geometry

Based on these parameters and empirical guidelines, the failure development time was determined to range from approximately 1 hour to 2.8 hours. From this range, two development times were chosen, being 1 hour for a fast case, and 2 hours for a slower case.

The empirical guidelines summarised in Wahl (1998) provide a range of formulae and estimation methods which result in a range of theoretical estimates for each component breach parameter (breach bottom width, side slopes and development time). However, in the case of the Makaroro dam, the incised gorge dominates the topography at the dam's footprint and strongly controls the maximum width at the base of the breach viz. while empirical estimates suggest a much greater bottom width, in reality this parameter is limited by the physical width of the river bed. Similarly, and to compensate (i.e. to maximise the breach width within the gorge), the side slopes selected for the dam breach are based on approximately matching the actual side slopes of the gorge rockhead.

The peak breach outflow is clearly dependent on the suite of selected dam breach parameters. Of the breach parameters, the failure development time typically has the widest range between the highest and lowest credible estimates. Furthermore, the shape of the outflow hydrograph and the peak flow is typically highly sensitive to this parameter. Therefore, to address this uncertainty, two breach development times have been modelled, i.e. a 1-hour and a 2-hour scenario.

The HEC-HMS v3.5 computer model (US Army Corps of Engineers, Hydrologic Engineering Centre) was used to determine the discharge hydrograph from the dam failure, based on the physical parameters of the dam such as height, width, side-slopes, volume-elevation relationship for the reservoir, as well as the breach development time, which for this case was chosen to be 1 hour or 2 hours.

The discharge hydrographs based on these scenarios are shown in Figure 7. Note that the rainy day failure has a slightly higher set of discharges which take into account the slightly higher starting water elevation, and the slightly larger volume of water stored behind the dam at the time of failure.

The reservoir elevation at failure initiation for the sunny day failure was RL 469.5 m (full supply level), and RL 474.3 m (maximum flood level) for the rainy day failure. The rainy day failure included inflow into the reservoir of approximately 450 m^3/s to simulate an extreme inflow, while the sunny day failure had a nominal 10 m^3/s inflow.

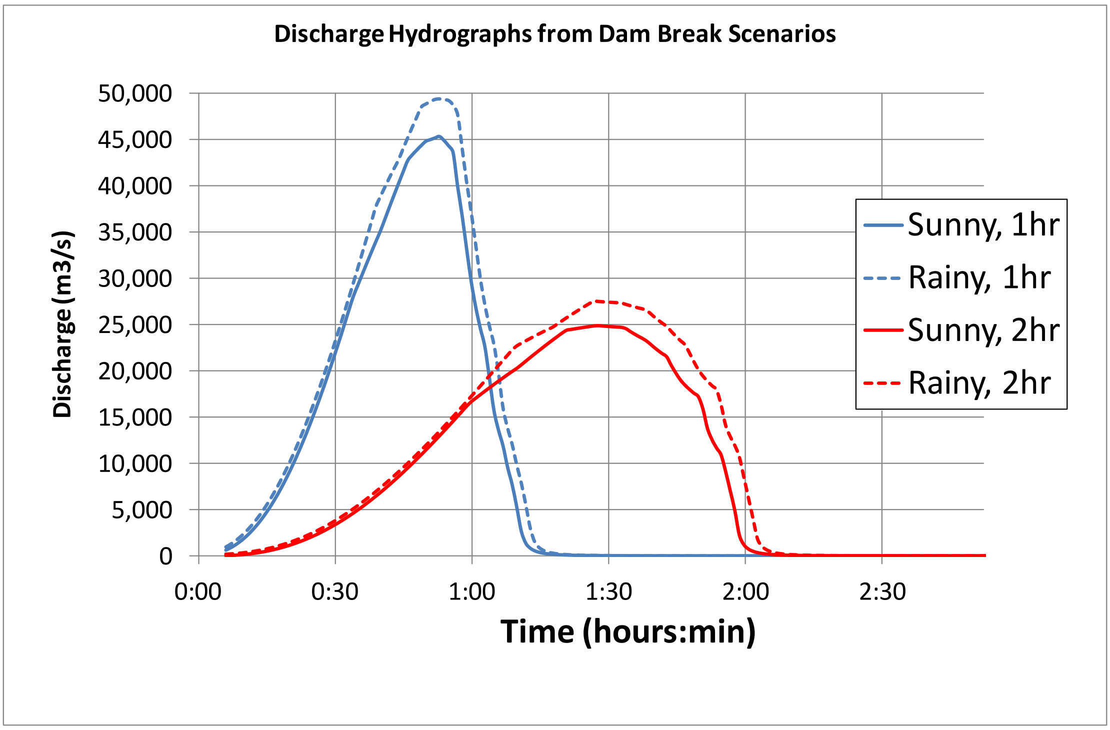

Figure 7: Discharge hydrographs from dam break model.

The results from the dam break scenarios show peak discharges for the dam as follows:

Breach Development Time	Peak Discharge (m^3/s) (Sunny day)	Peak Discharge (m^3/s) (Rainy day)
2 hours	25,000	27,500
1 hour	45,000	49,000

The discharge hydrographs shown in Figure 7 were applied to the 2-dimensional hydrodynamic model (described in section 5) in order to determine the downstream effects from the dam failure.

4.1 Comparison of Dam Break Discharge to Empirical Guidelines

The values for peak discharge from the HEC-HMS dambreak model were compared to empirical charts presented in Wahl (1998).

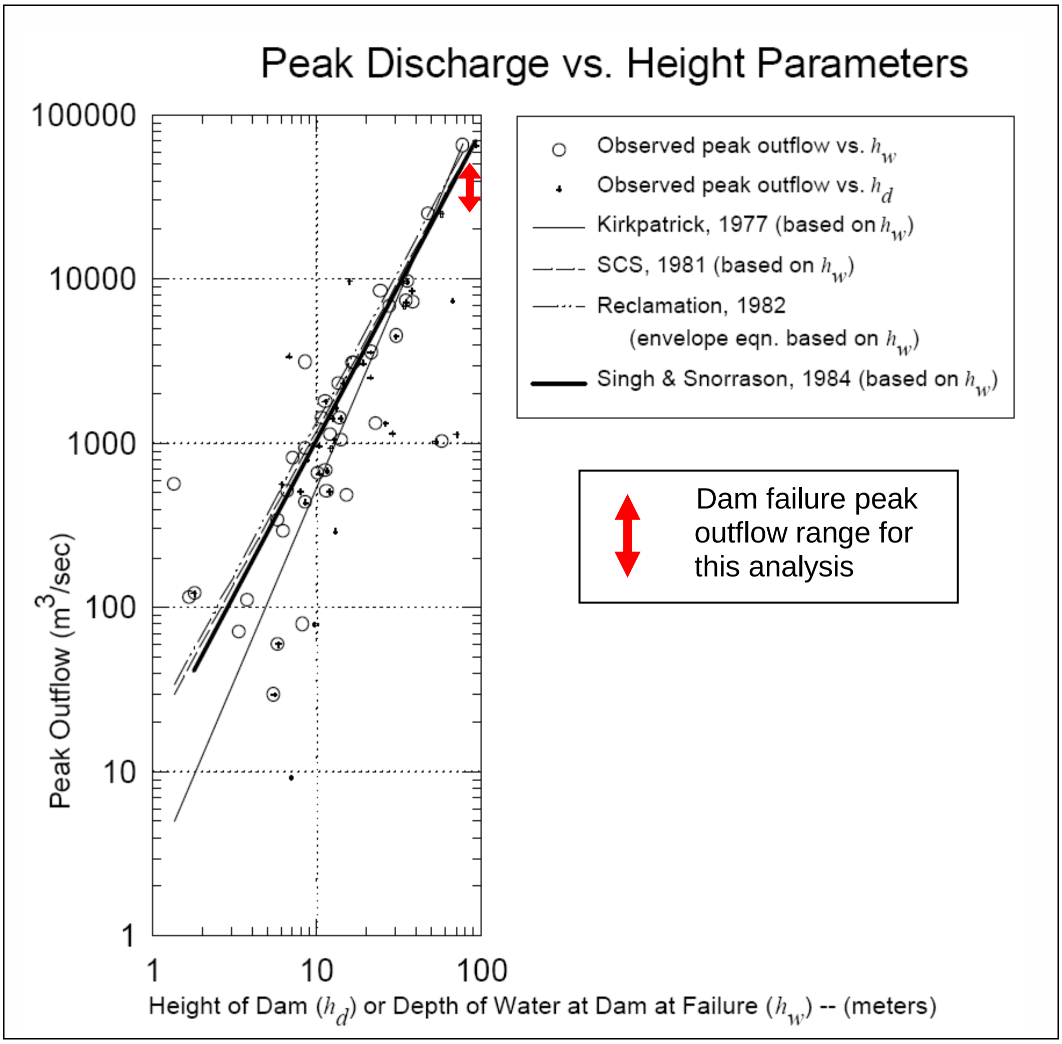

Figure 8: Peak Discharge vs. Height for observed dam failures (Wahl, 1998)

The peak discharge versus height shows that for an 85 m high dam, the range of peak outflows determined for the dam breach of between 25,000 m^3/s and 49,000 m^3/s plots slightly low compared with case study data. However it should be noted that the dams that make up the observed failures generally have much larger storage volumes for a given dam height compared with the proposed dam. Therefore, the observed dam failures have produced significantly higher discharges since the stored volume is also a determinant of the peak discharge. Since Figure 8 is only concerned with dam height, the results may appear skewed. The next figures incorporate storage volume as well, to overcome this problem.

Figure 9 uses the reservoir storage of 90 x 10^6 m^3, with the range of 25,000 m^3/s to 49,000 m^3/s being slightly higher than the graph indicates, for the main dam.

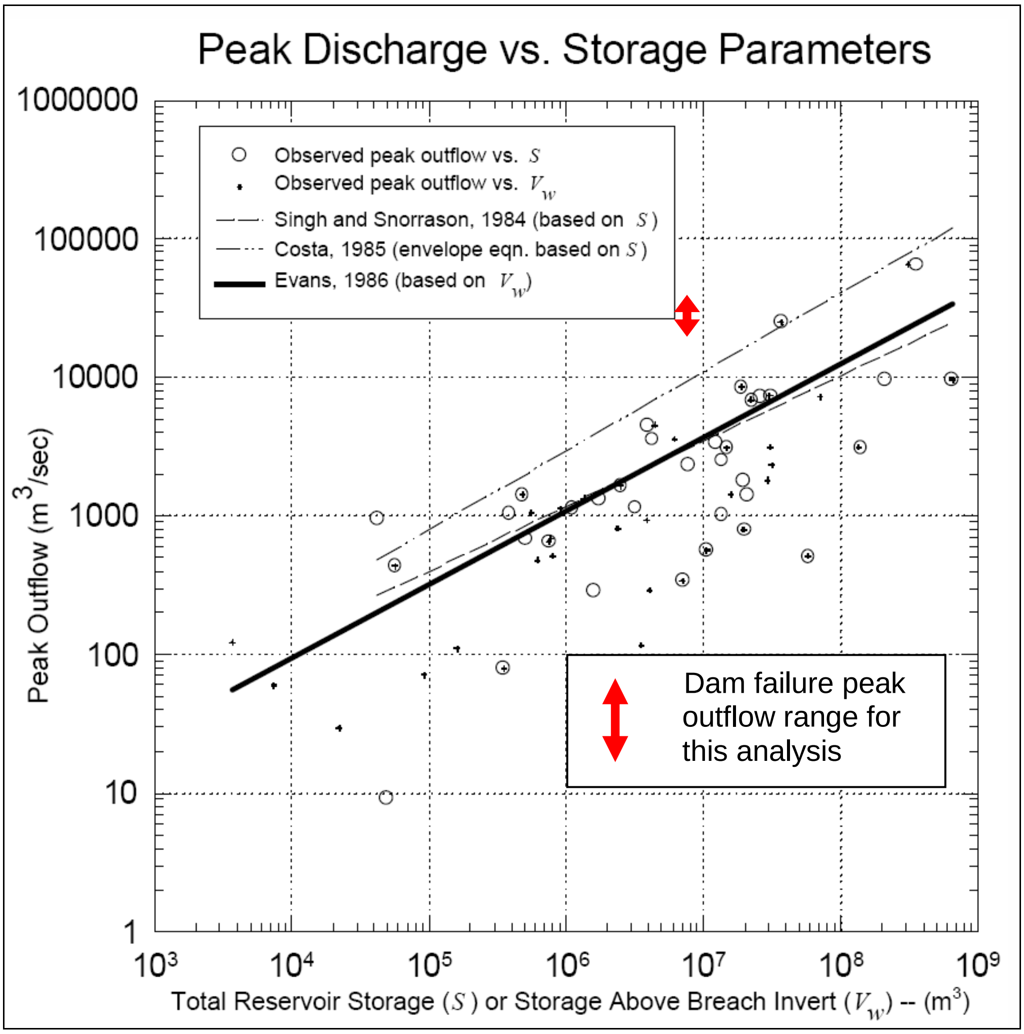

Figure 9: Peak Discharge vs. Storage Parameters for dam failures (Wahl, 1998)

Again, the reason for this apparent over-prediction is that the database of observed dam failures generally comprise dams with significantly lower heights for a given reservoir storage (i.e. other dams appear to be more 'topographically' efficient) compared with the proposed dam.

Figure 10 uses the volume x height (= 90 x 10^6 x 85 = 7.65 x 10^9) parameter versus peak discharge. The range of 25,000 m^3/s to 49,000 m^3/s is within the bounds of the values shown on the graph.

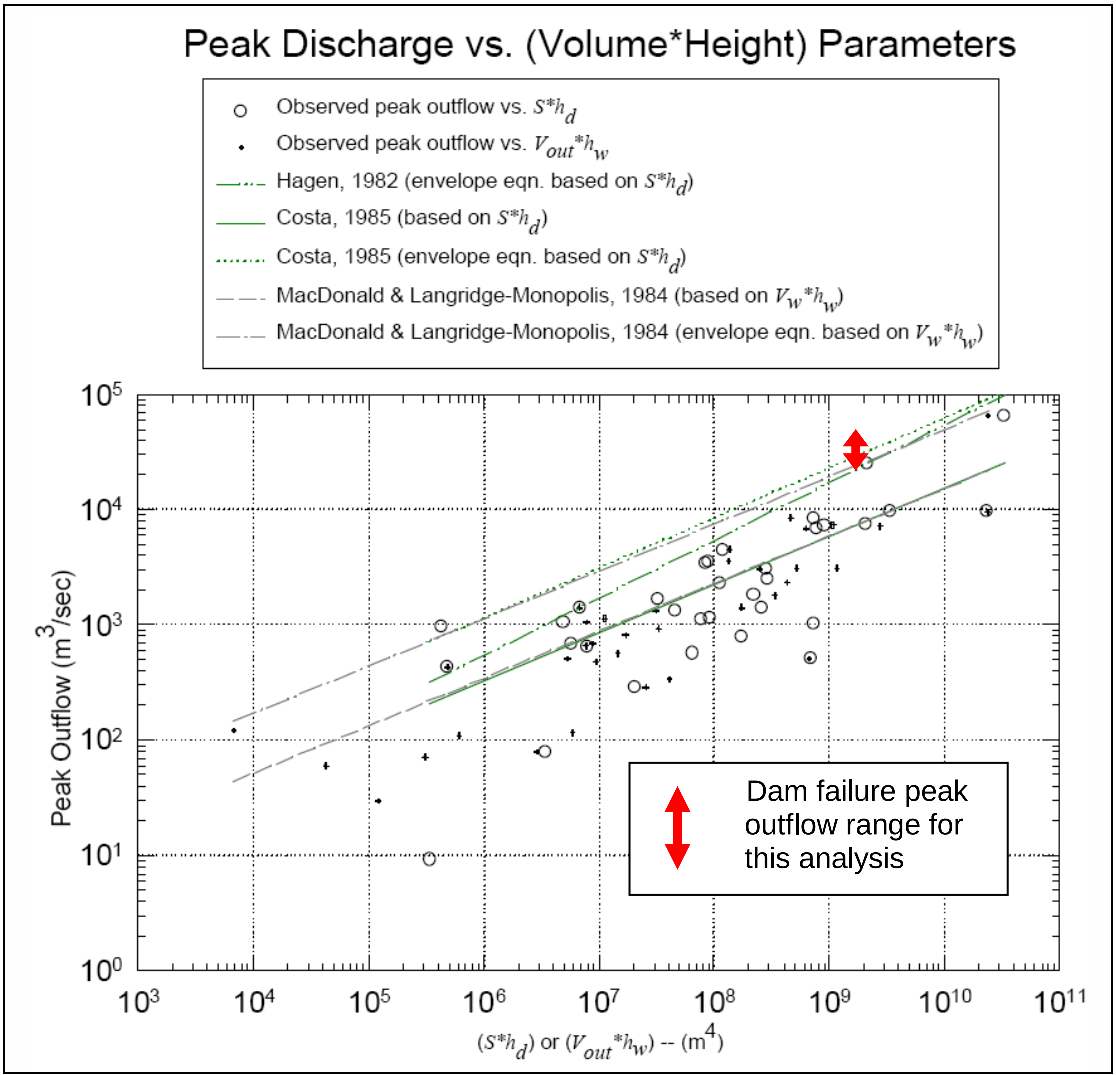

Figure 10: Peak Discharge vs. (Volume x Height) for dam failures (Wahl, 1998)

Based on the comparison of the empirical guidelines to the values obtained from the HEC-HMS dambreak simulation, the values from the simulation appear to be within reasonable bounds.

In a subsequent article by Wahl (2004), Wahl noted that a predictive equation developed by Froehlich (1995) had both the lowest prediction error and smallest uncertainty of all the peak flow prediction equations assessed. The equation is as follows: $Q_p = 0.607(V_w^{0.295} h_w^{1.24})$. By applying this equation, the peak dam break flow is estimated to be approximately 31,000 m^3/s, which is closer to the peak outflow from the modelled 2-hour failure scenario (25,000 m^3/s) than that from the 1-hour failure scenario (45,000 m^3/s). Thus, the latter may be considered to be on the conservatively high side.

5 2-Dimensional Hydrodynamic Model

In order to examine the dam failure flood wave as it travels downstream, a 2 dimensional (2-D) computer model of the system was created using the Danish Hydraulic Institute's Mike21-HD software. A key component of the model is the ground elevation, which is based on LiDAR data collected between 2003 and 2011, as well as surveyed levels of stopbanks and river cross sections. A grid size of 20 m x 20 m was chosen, which appears to adequately represent the rivers, which in most cases are several hundred metres wide during high flows, as well as the floodplains, which can be many kilometres wide. The rivers and floodplains were both modelled using the same grid. In cases where features such as stopbanks needed to be represented by a specific elevation, as opposed to average elevation of the LiDAR points within the cell, the grid elevations were set to the correct elevation, based on the stopbank elevation measured from survey cross sections.

The digital elevation model (DEM) is shown in Figure 11. This also shows thin grey lines where the locations of stopbanks have been added to the DEM. Large areas where there is no risk of flooding are eliminated from the model in order to reduce computation time.

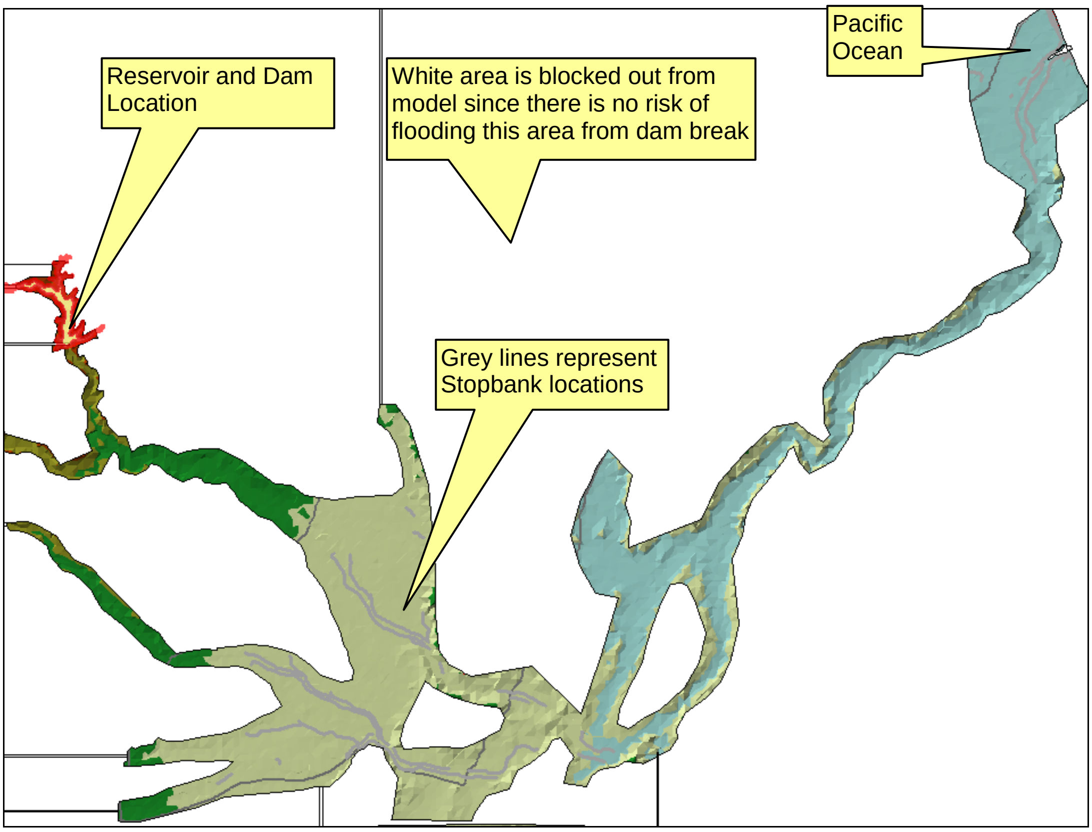

Figure 11: Digital Elevation Model used in 2D Hydrodynamic Model.

5.1 Boundary Conditions

For the rainy day scenarios, boundary conditions were set such that the background flows in the downstream rivers were equivalent to a 20 year return period design discharge.

For the sunny day scenarios, boundary conditions were set to a very low nominal flow, less than the mean annual flow.

The dam failure discharge hydrograph was applied on the Makaroro River at the dam site.

The downstream boundary condition was set to a constant sea level of RL 10.0 m (Hawke's Bay Local datum, mean sea level = 10 m.)

5.2 Roughness Values

A roughness value (Manning's n) value of 0.04 was used for all river sections throughout the model. Any area outside the river corridors was set a roughness value of 0.06. Varying roughness values for the floodplains were trialled, and rougher floodplain values result in slightly greater depths of flooding. For example, a roughness of n = 0.1 on the floodplain produced depths approximately 0.3 m deeper, as compared to n = 0.06 on the floodplain. A value of n = 0.06 was chosen as a representative value for the current level of study. If more detail is required from future analysis, the roughness values are one component that may be examined in more detail to produce results that reflect the existing ground conditions at the time. It is noted as well that regardless of the roughness values chosen for the analysis, there is no guarantee that land use changes will not alter the roughness of the floodplain in the future.

5.3 Calibration

Calibration of the 2-D model for the scale of discharge (45,000 m^3/s) is generally not possible since there are no recorded events which have discharges of such magnitude. For model scenarios with discharges less than the 100 year return period values, the results are more robust, such that there is little out of channel flow, since the stopbanking system has generally been constructed to contain the 100 year discharge.

Model results were also compared to other flood models created by the HBRC, with the conclusion that the current dam break model provides results that appear consistent with other models, at least for the range of natural floods.

6 2-Dimensional Hydrodynamic Model Results

The results from the computer model with the dam break hydrographs indicate a very severe flood wave which impacts the Makaroro, Waipawa and Tukituki Rivers from the dam site to the coast. In order to determine the potential impact category (PIC) of the dam, the sunny day 1-hour failure scenario, where the dam break results in a peak discharge of around 45,000 m^3/s was used. This scenario is most likely to produce the highest incremental damage, since the dam failure occurs at a time when there are no other flood related damages occurring in the catchment. All downstream damage in this case would be attributed to the dam failure.

6.1 Sunny Day 1-Hour Failure

An overview of the flood extents, along with discharges and timings is shown in Figure 12. In this figure, three times are shown at specific locations, being the start, peak and end of the flood wave. While the start and peak times are quite distinctive, the end time shown relates to the time when the flood wave has substantially decreased to the point where there is no out-of-channel flow. At this point, there may be significant flow in the river corridor, and there will likely be ponding in the floodplain areas, however the bulk of the flood wave will have passed by this time.

Details of specific areas are included later in this section.

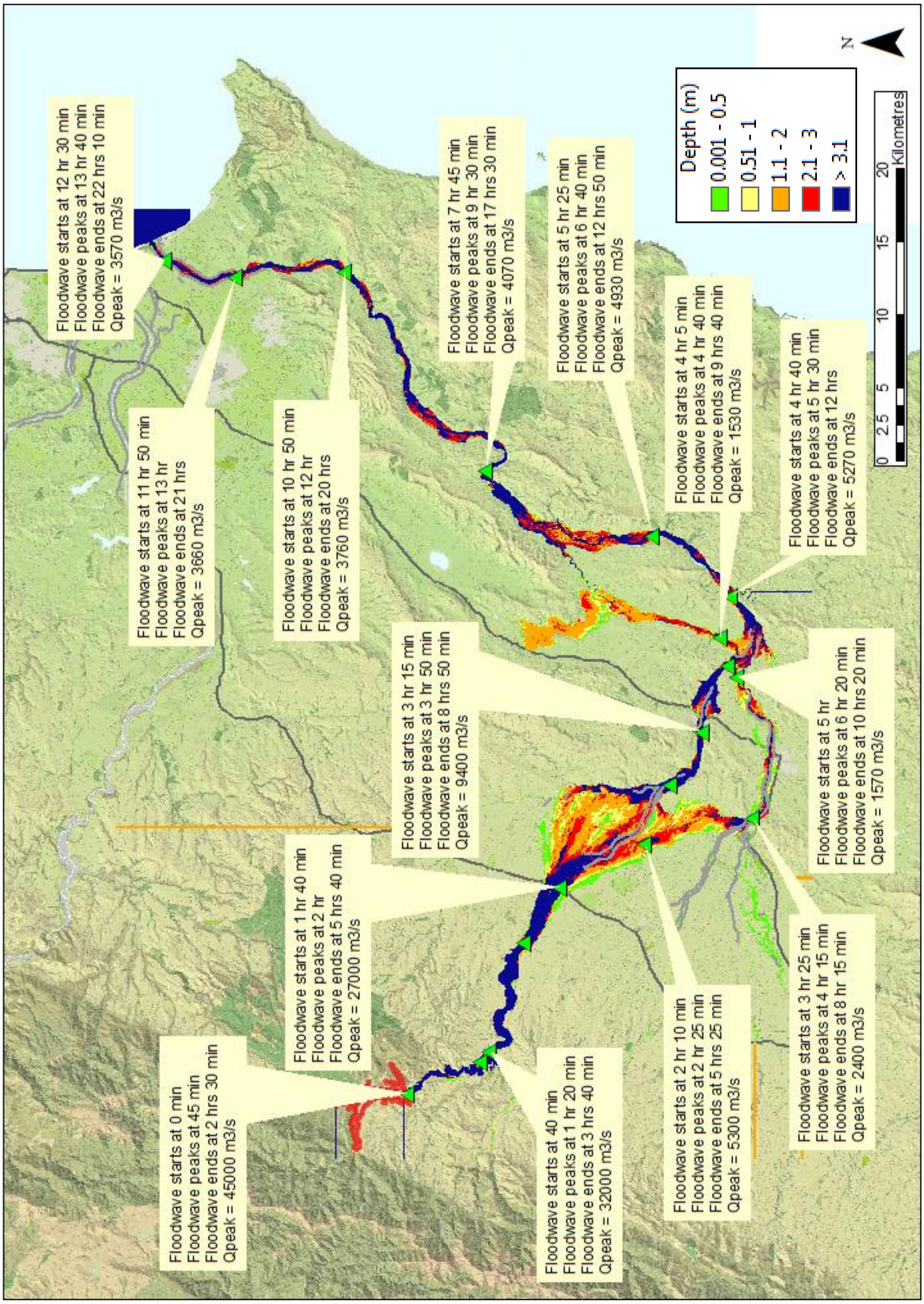

Figure 12: Timing and discharge results for sunny day failure, 45,000 m^3/s scenario

Figure 13 shows the discharge hydrographs for various locations along the flood wave path, on the Makaroro, Waipawa and Tukituki Rivers.

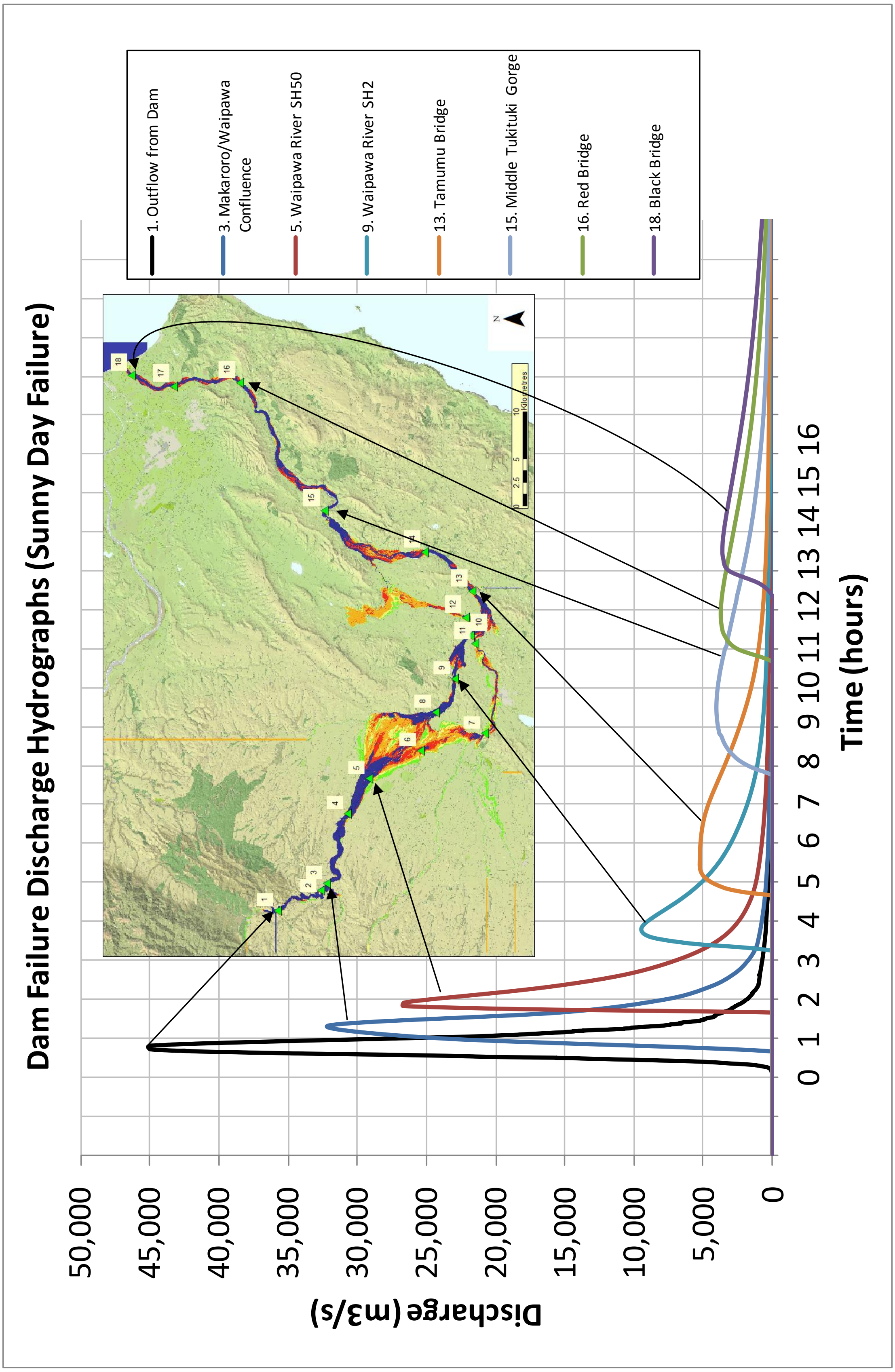

Figure 13: Discharge hydrographs sunny day failure, 45,000 m^3/s scenario.

Details of the inundation extents for particular sections of the rivers are provided in the following figures.

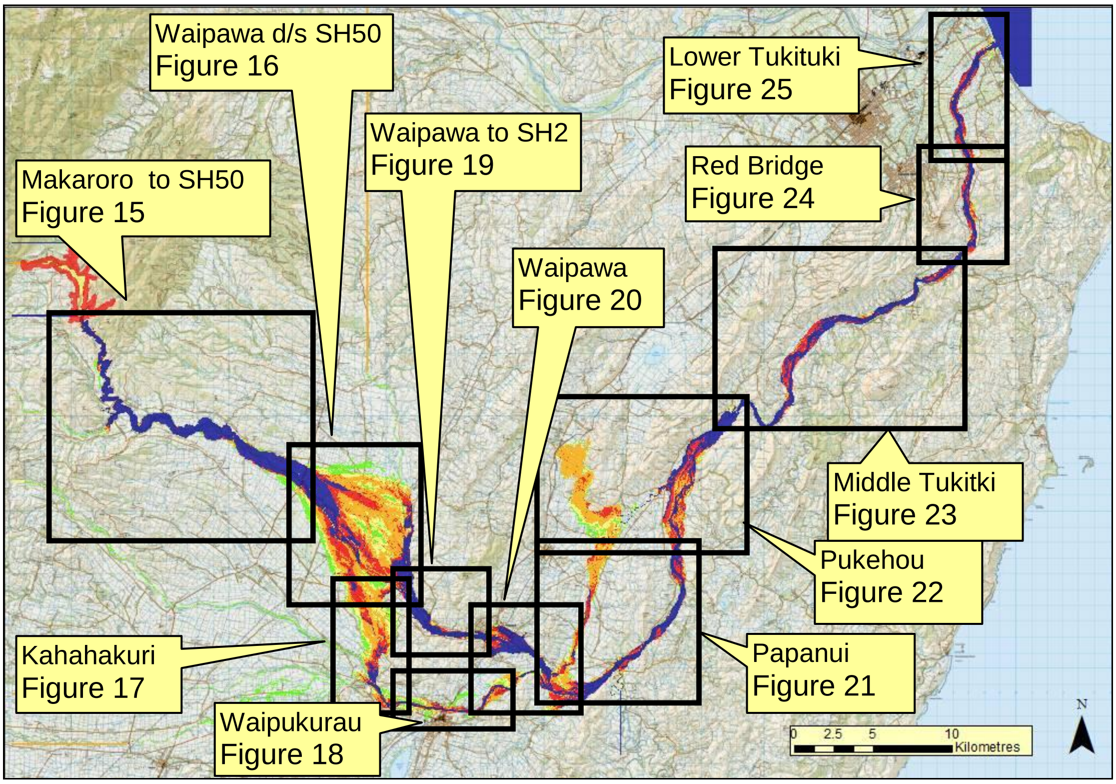

Figure 14: Map showing locations and coverage of detailed maps.

Figure 15 shows the flood wave in the deeply incised section of the Makaroro and Waipawa Rivers. In this section there is virtually no out-of-channel flow. The water depth tapers down after exiting the incised section to about 5 m deep near SH50.

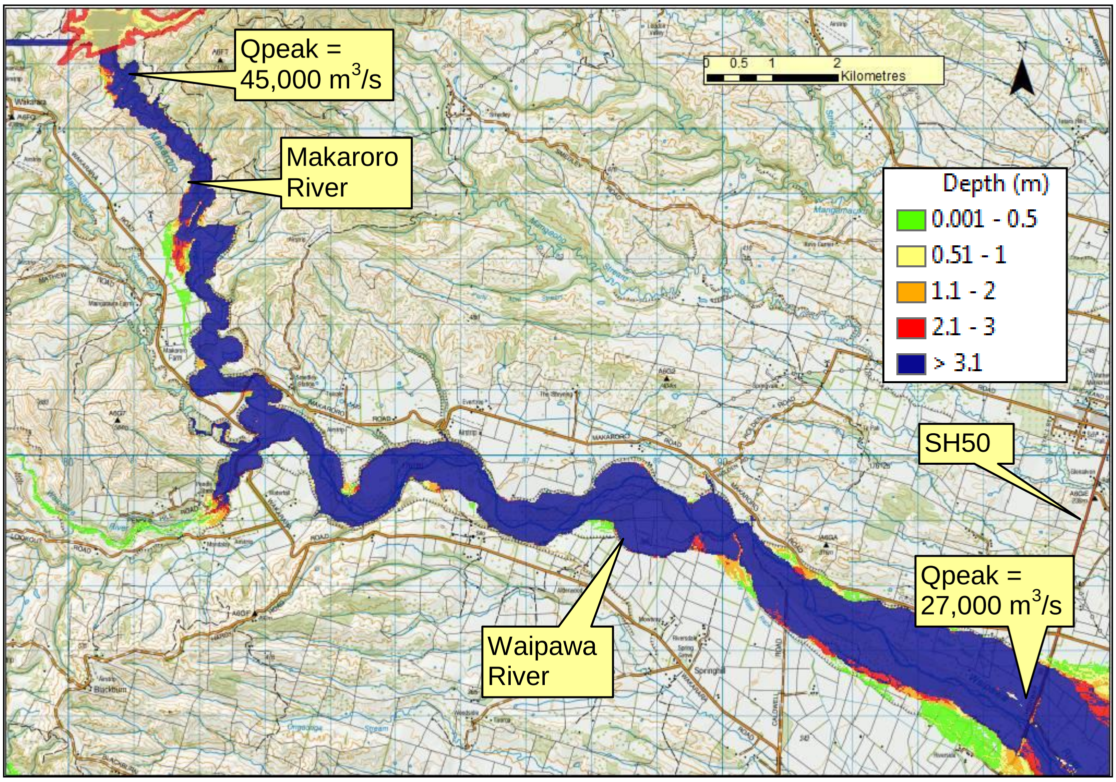

Figure 15: Makaroro River and Waipawa River to SH50, flood depths for sunny day failure, 45,000 m^3/s

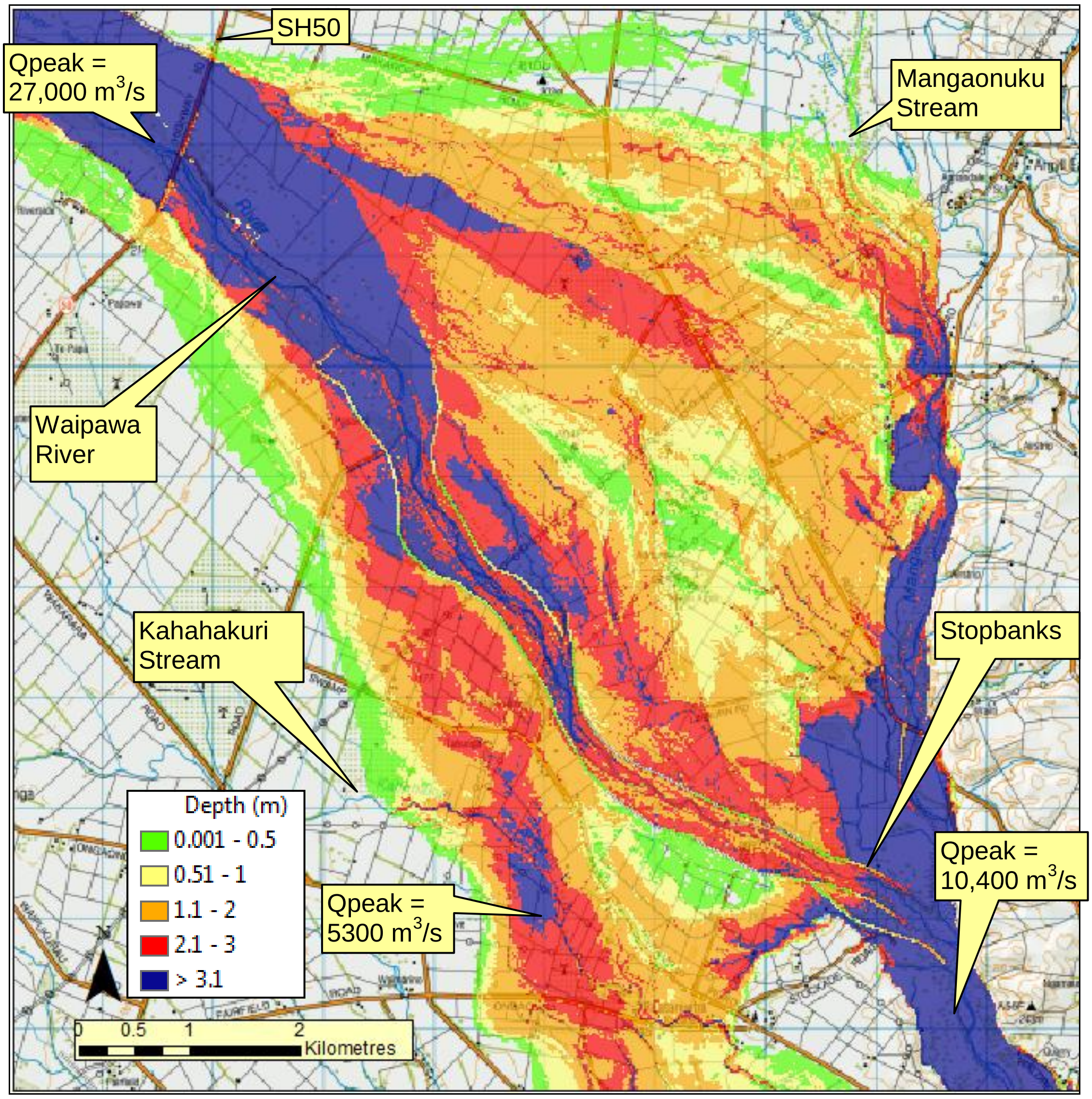

Figure 16: Waipawa River downstream SH50 for sunny day failure, 45,000 m^3/s

Figure 16 shows the extensive out-of-channel flows from the Waipawa River downstream of SH50. On the left bank the overland flow travels towards the Mangaonuku Stream, then back into the Waipawa.

On the right bank the overland flow travels to the Upper Tukituki River via the several small streams joining into the Kahahakuri Stream, as shown in Figure 17.

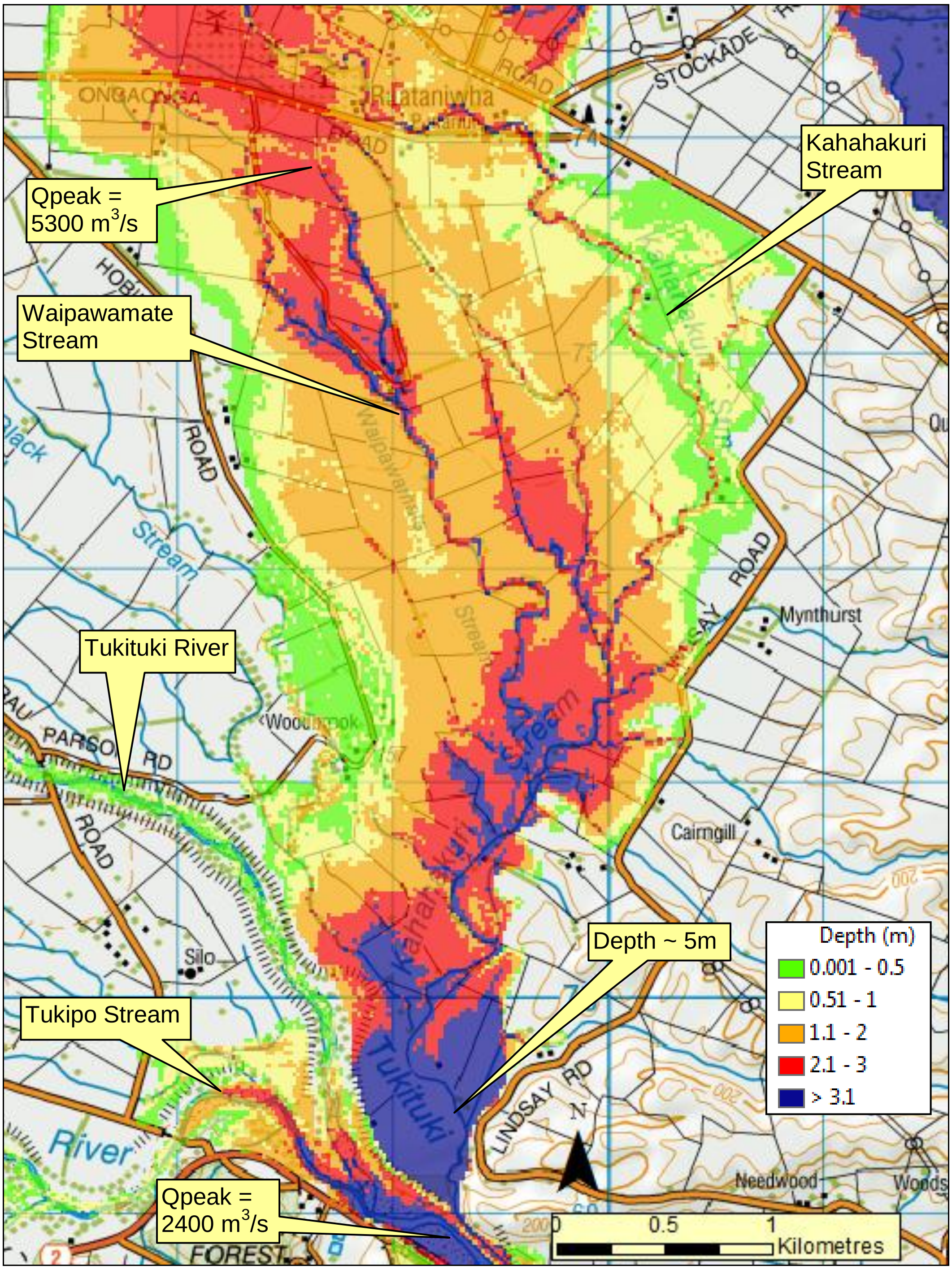

Figure 17: Kahahakuri and Waipawamate flood depths, sunny day failure, 45,000 m^3/s

Figure 17 shows the Kahahakuri and Waipawamate Streams as they flow into the Tukituki River, with depths reaching about 5 m in the lowest areas.

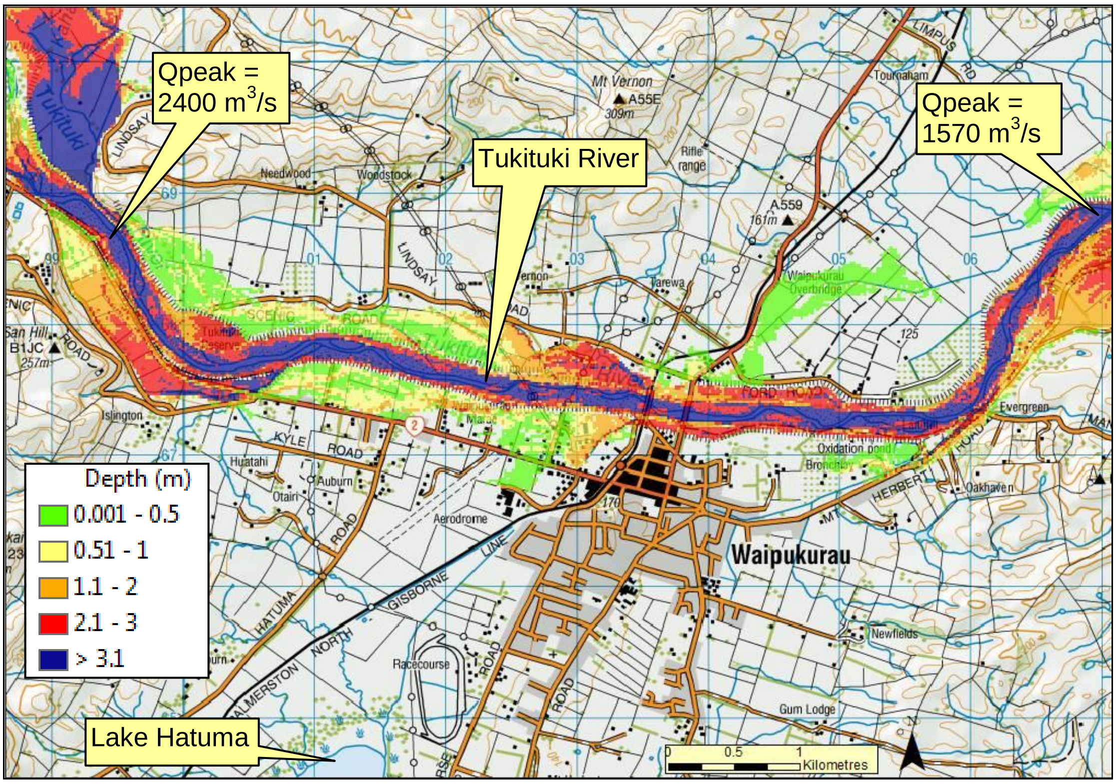

Figure 18: Tukituki River at Waipukurau flood depths, sunny day failure, 45,000 m^3/s

Figure 18 shows the Tukituki River at Waipukurau, with minor overflows over the stopbanking system on both sides of the river. In this area, the 100 year design flow for the stopbanks is 1200 m^3/s.

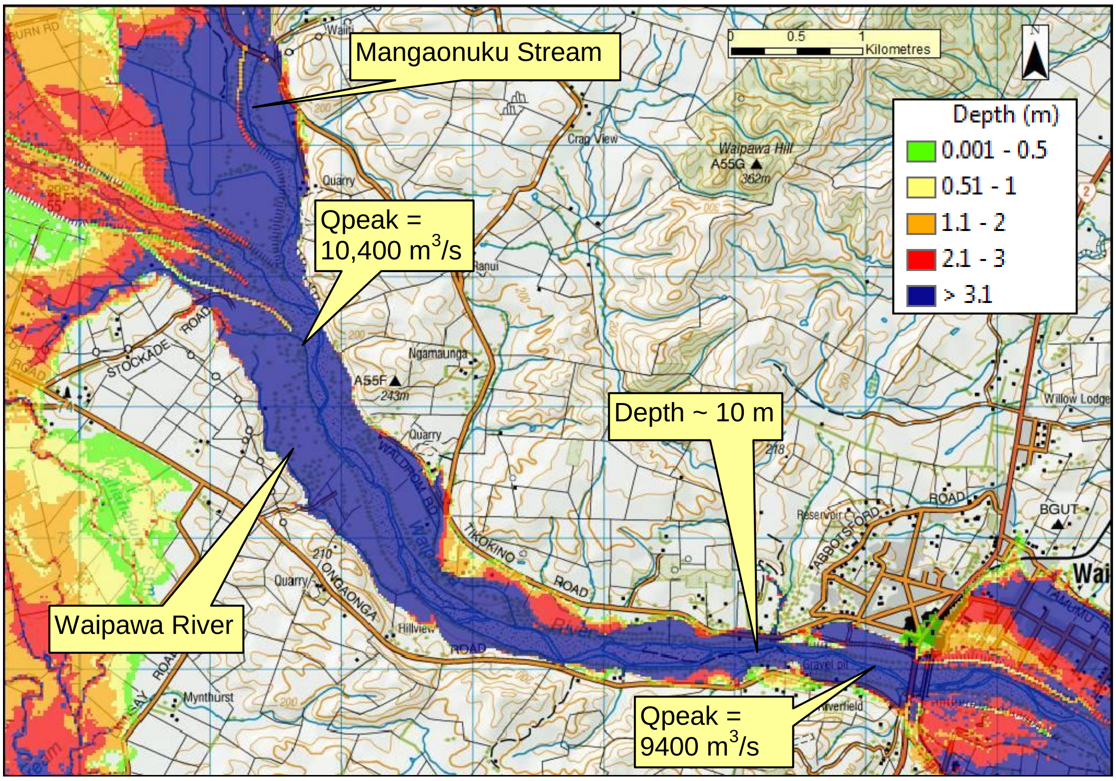

Figure 19: Waipawa River, Mangaonuku Stream to SH2, flood depths, sunny day failure, 45,000 m^3/s

Figure 19 shows the confined reach of the Waipawa River upstream of the town of Waipawa. The flood depths in this section reach about 10 m deep.

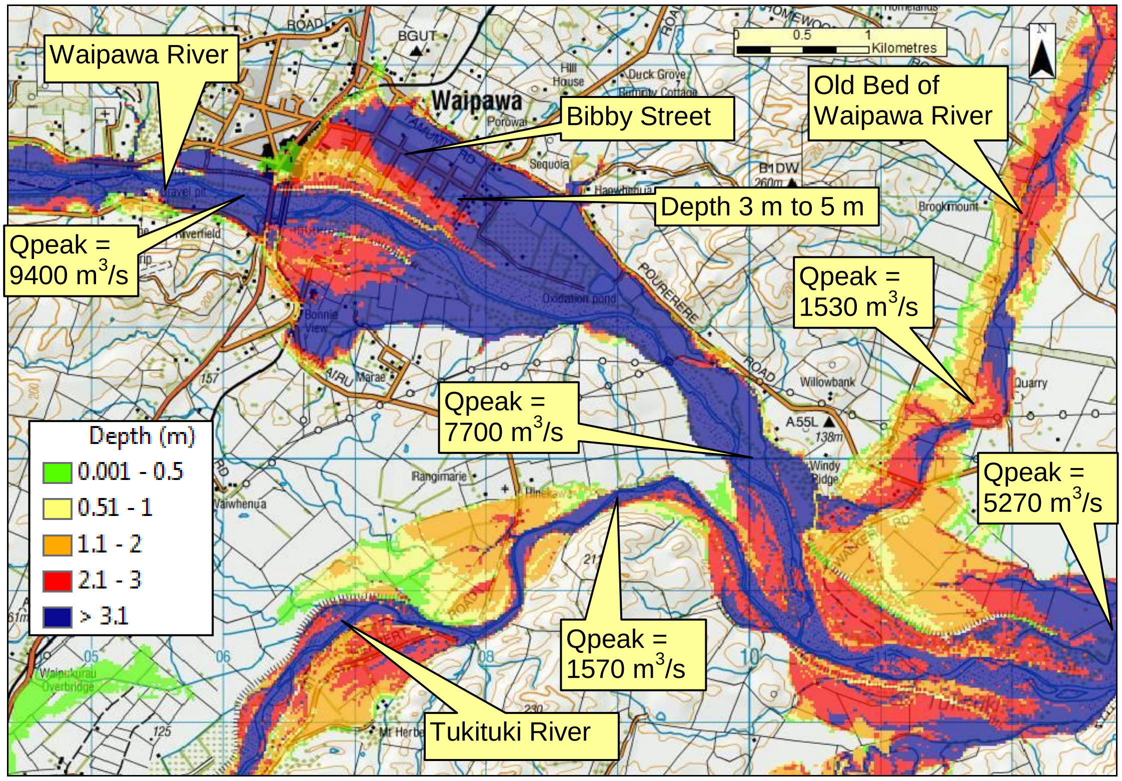

Figure 20: Waipawa Town to Waipawa/Tukituki confluence flood depths, sunny day failure, 45,000 m^3/s

Figure 20 shows the flooding around the town of Waipawa, with depths in the Bibby Street area reaching between 3 m and 5 m. Downstream of the town of Waipawa, near the confluence of the Waipawa and Tukituki Rivers, a substantial flow of about 1530 m^3/s flows down the old bed of the Waipawa River, and eventually joins the Papanui Stream.

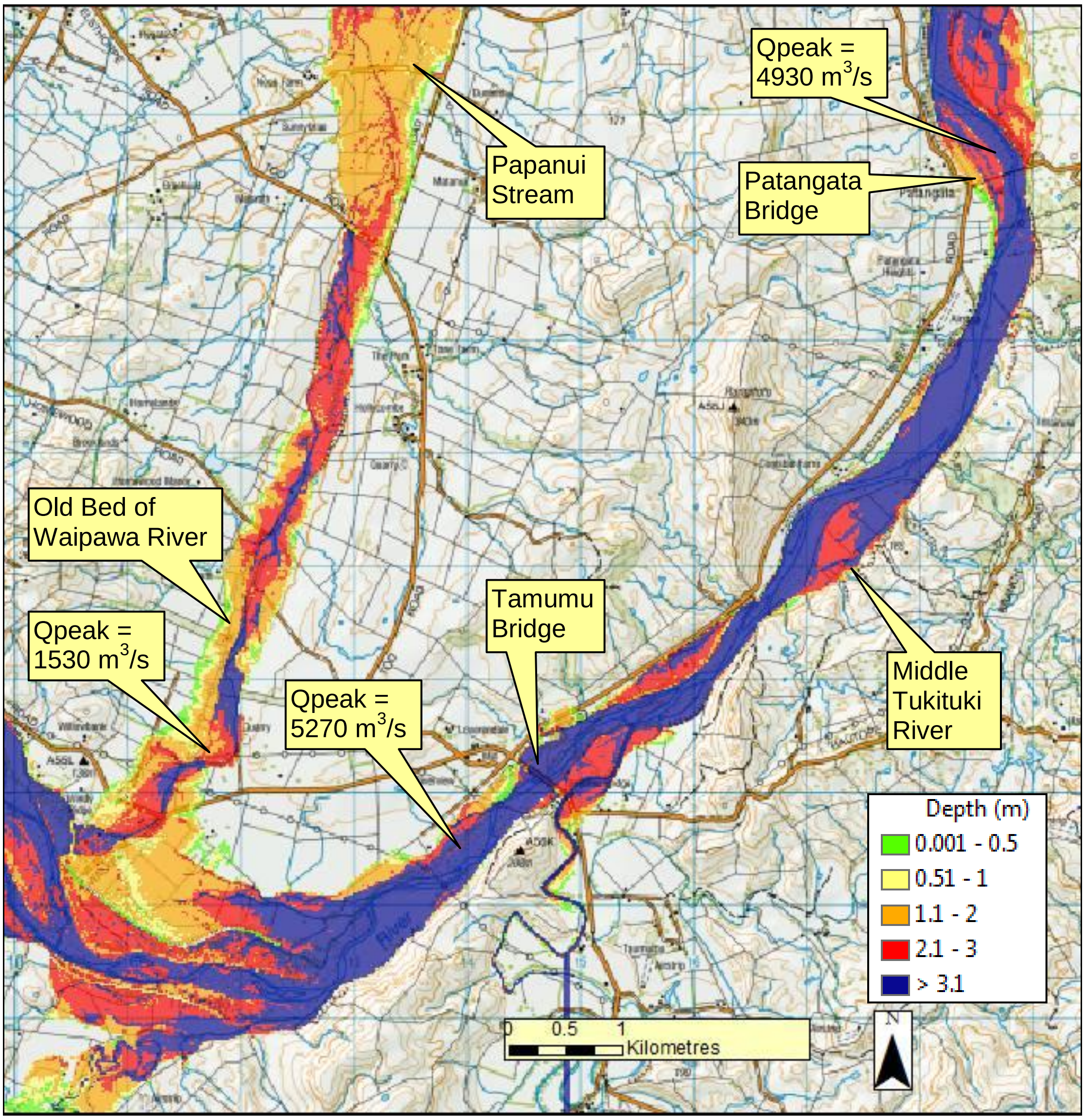

Figure 21: Papanui, Tukituki River flood depths, sunny day failure, 45,000 m^3/s

Figure 21 shows the overflow down the old bed of the Waipawa River, into the Papanui Stream. The flood depths at the Tamumu and Patangata Bridges indicate the bridges are likely to be inundated or severely damaged. The 100 year design flow in this section of the Middle Tukituki River is approximately 2350 m^3/s, indicating the flood wave from the dam break is still substantially greater than the 100 year flood at this location.

Figure 22 shows the inundation in the area towards Pukehou which is very low lying.

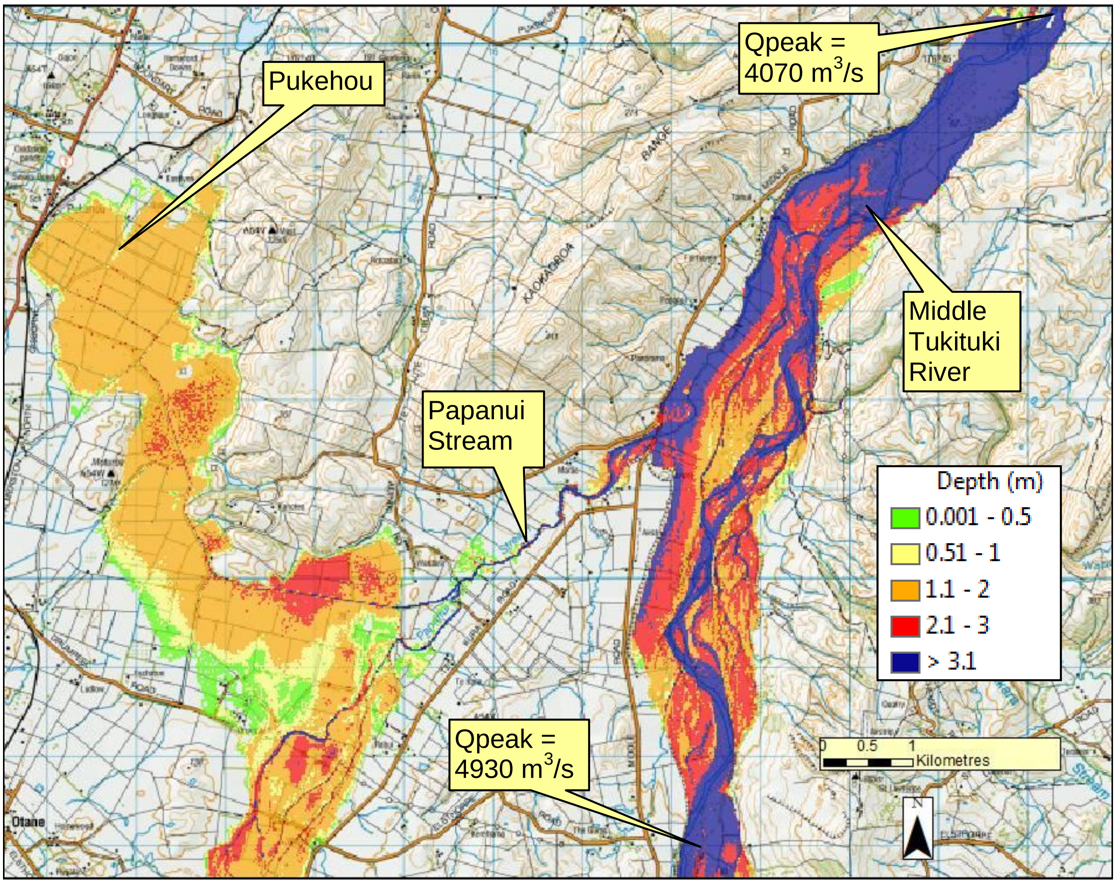

Figure 22: Papanui/Pukehou flood depths, sunny day failure, 45,000 m^3/s

Figure 23 shows the incised section of the Middle Tukituki River, where very little out-of-channel flow occurs.

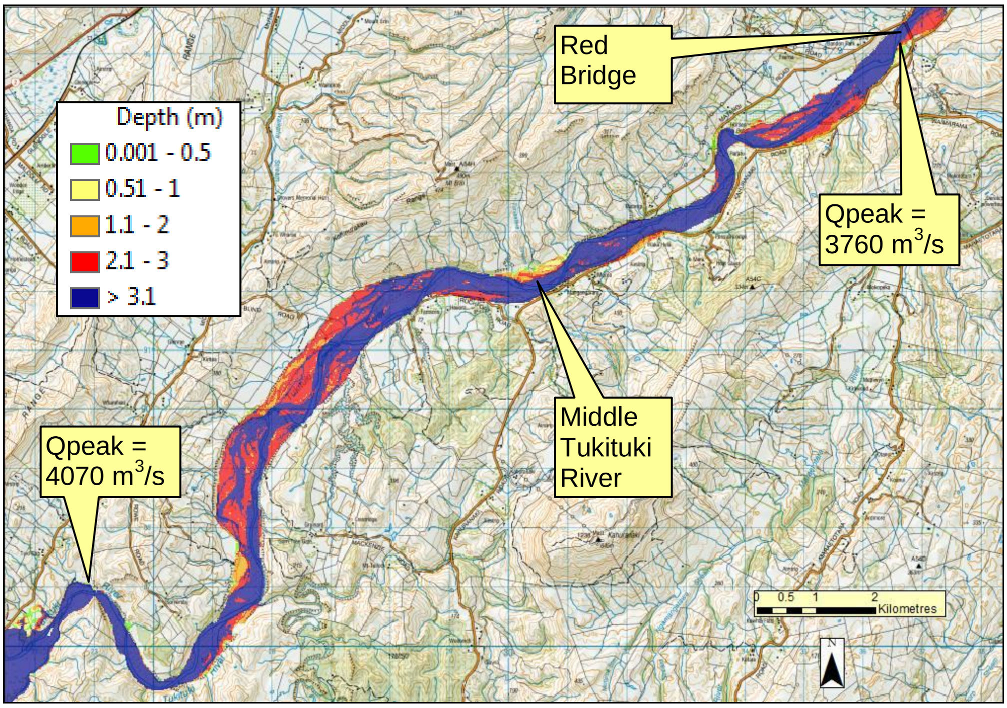

Figure 23: Middle Tukituki River flood depths, sunny day failure, 45,000 m^3/s

Figure 24 shows the section of Lower Tukituki River from Red Bridge downstream.

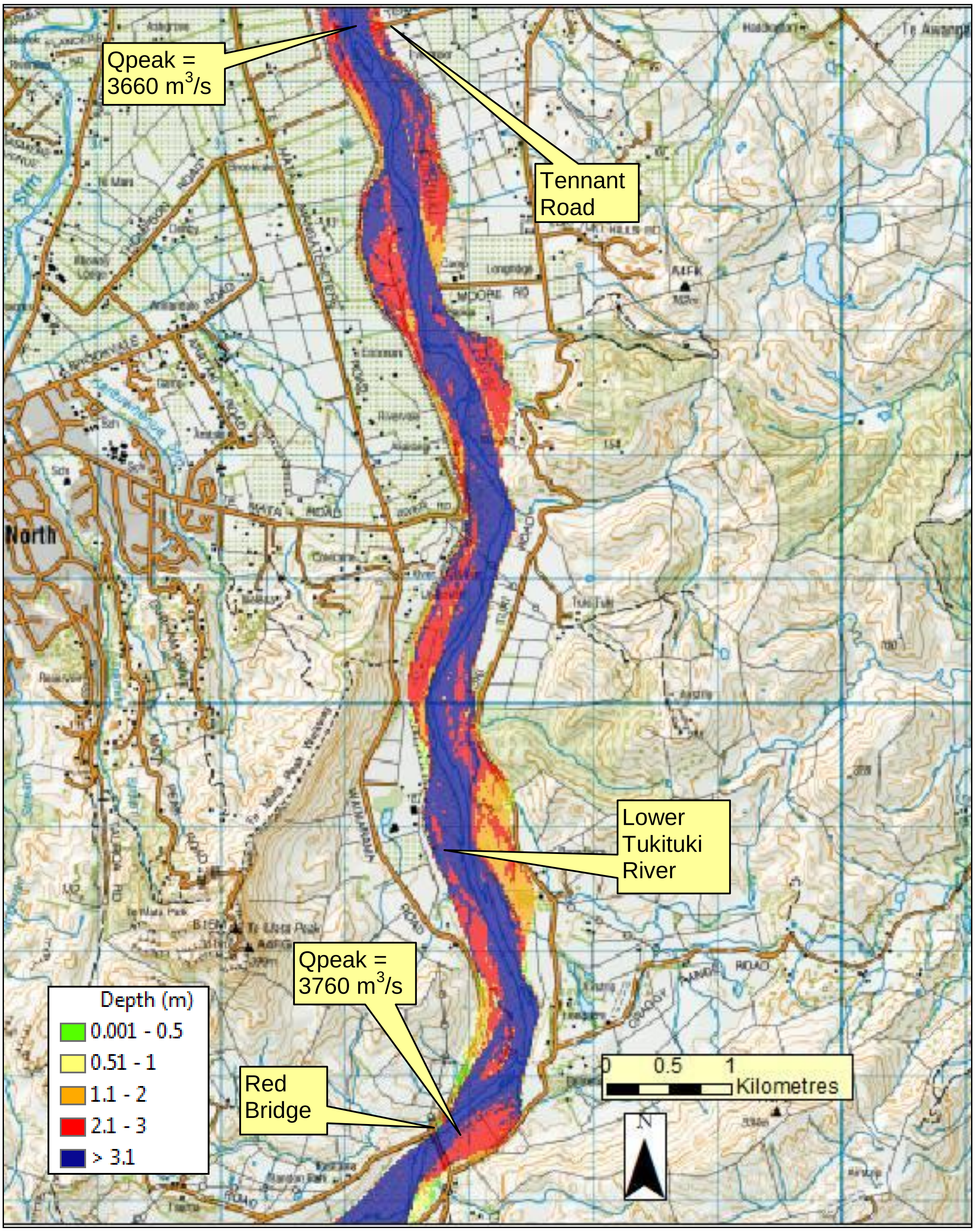

Figure 24: Tukituki River near Red Bridge, flood depths, sunny day failure, 45,000 m^3/s

In the section of the Lower Tukituki River downstream of Red Bridge, the 100 year design discharge is 4800 m^3/s, indicating that for the sunny day failure scenario starting with 45,000 m^3/s, the peak discharge in this section after attenuation of the flood wave, is less than the 100 year design flow.

Figure 25 shows the final section of Lower Tukituki River, with minor backwater flows into Grange Creek, which would normally occur for high discharges in the Tukituki River.

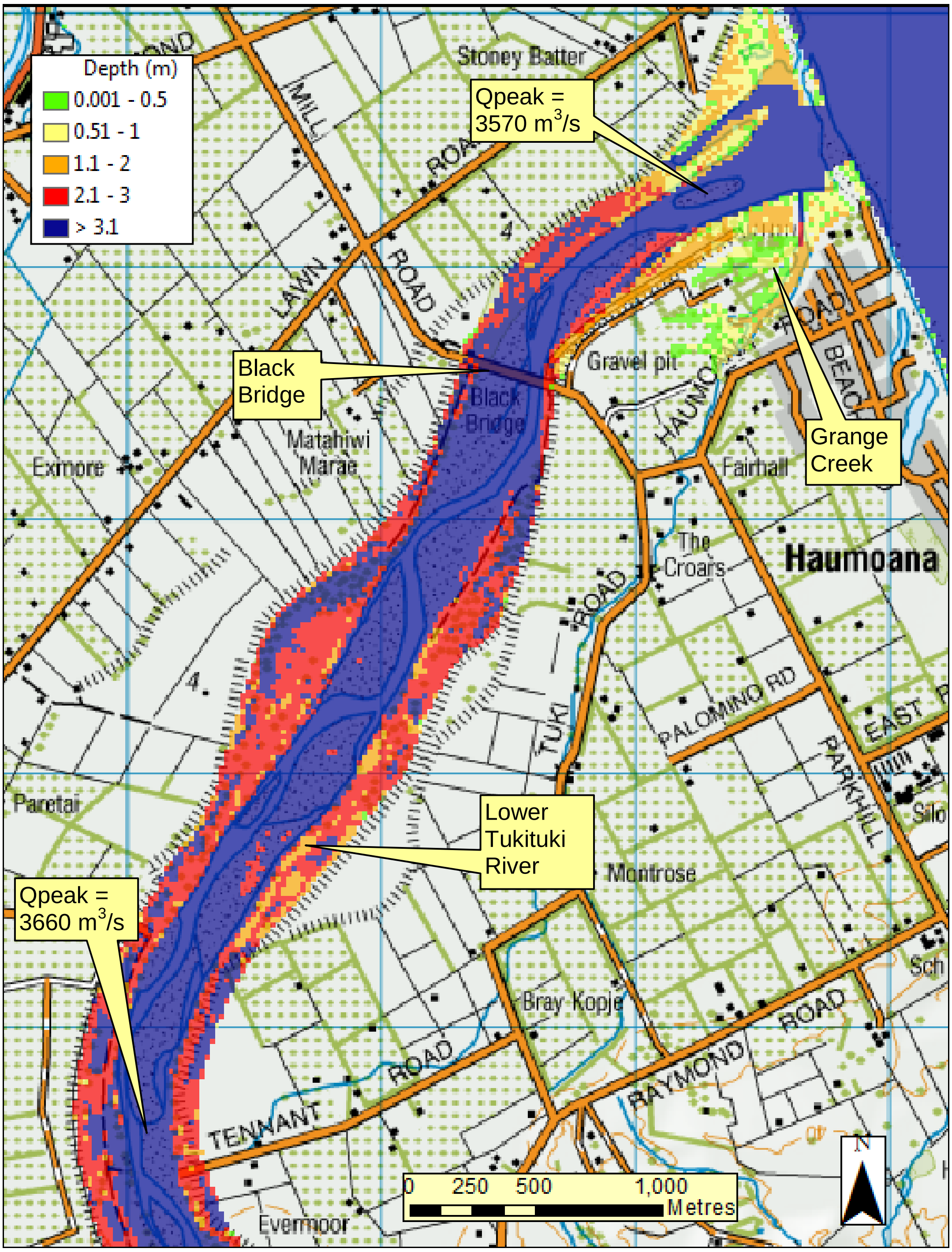

Figure 25: Lower Tukituki River flood depths, sunny day failure, 45,000 m^3/s

The results presented above from the sunny day 1-hour failure scenario, which produces a 45,000 m^3/s peak discharge at the dam, show areas where there is extensive inundation on the Ruataniwha floodplains, as well as significant flows in the river system.

Figure 26 shows the inundated areas outside of the normal river corridor that are considered to be at risk from flooding from this dam failure scenario.

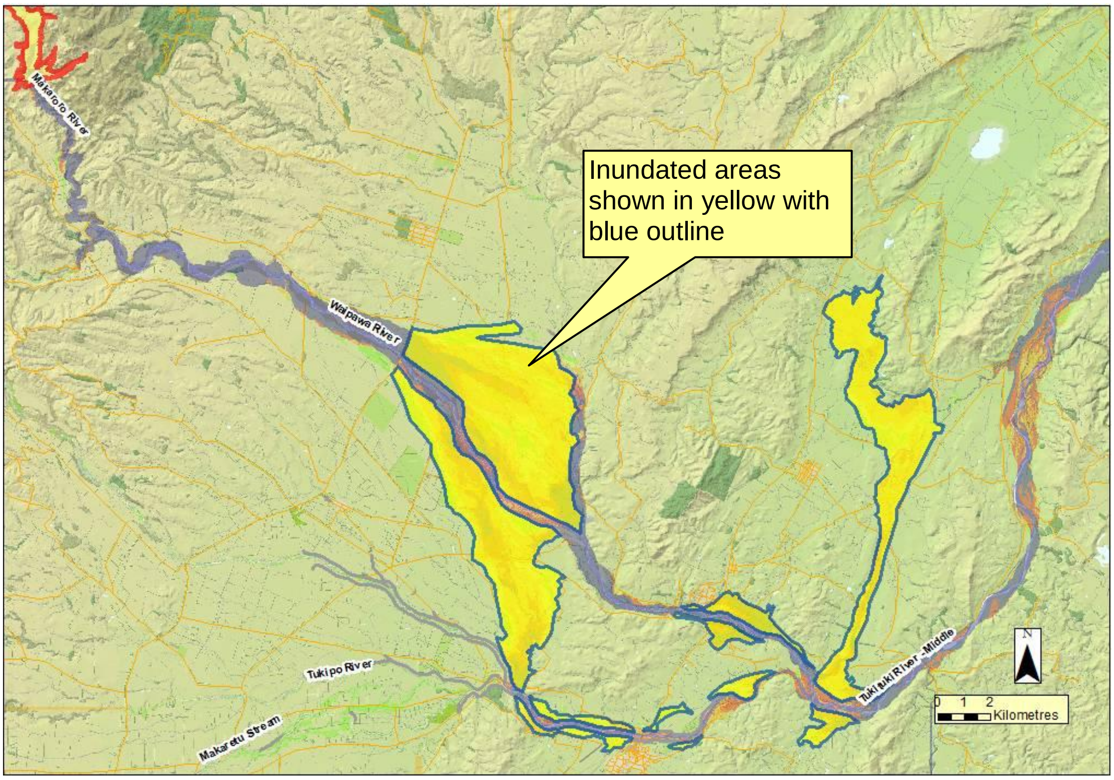

Figure 26: Inundated area outside of normal flood risk area on rivers, sunny day failure, 45,000 m^3/s scenario.

The inundated areas at risk shown in Figure 26 add up to approximately 7800 Ha in area.

6.2 Sunny Day 2-Hour Failure

The 2 hour, sunny day failure scenario produced a peak discharge of 25,000 m^3/s at the dam. The following two figures show the similarity of results for the 25,000 m^3/s and for the 45,000 m^3/s scenarios.

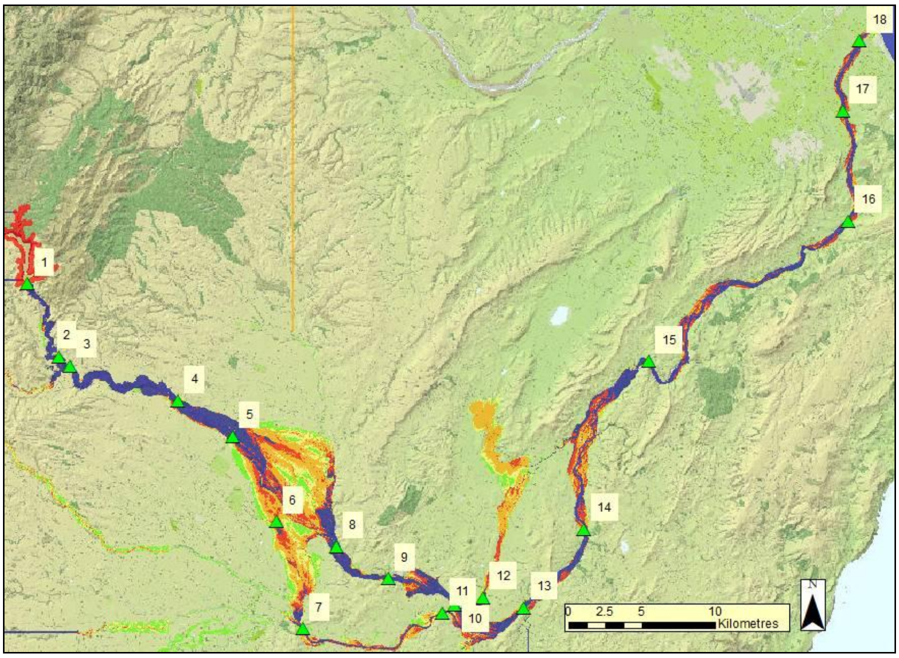

Figure 27: Flood extent results for sunny day failure, 25,000 m^3/s scenario

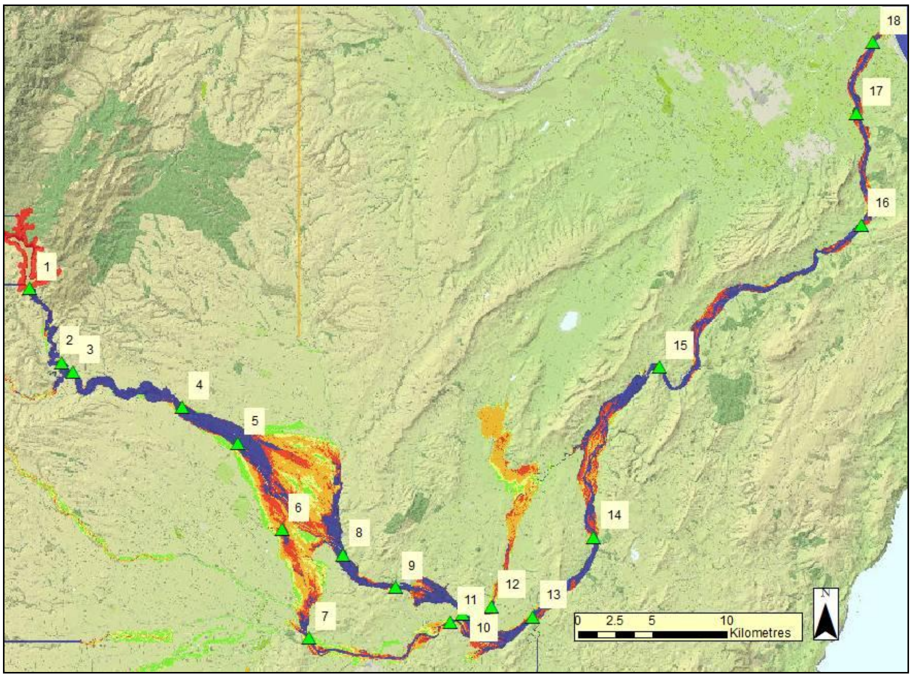

Figure 28: Flood extent results for sunny day failure, 45,000 m^3/s scenario

Figure 29 shows several discharge hydrographs at various locations in the river system, for the two peak flow scenarios of 25,000 m^3/s and 45,000 m^3/s.

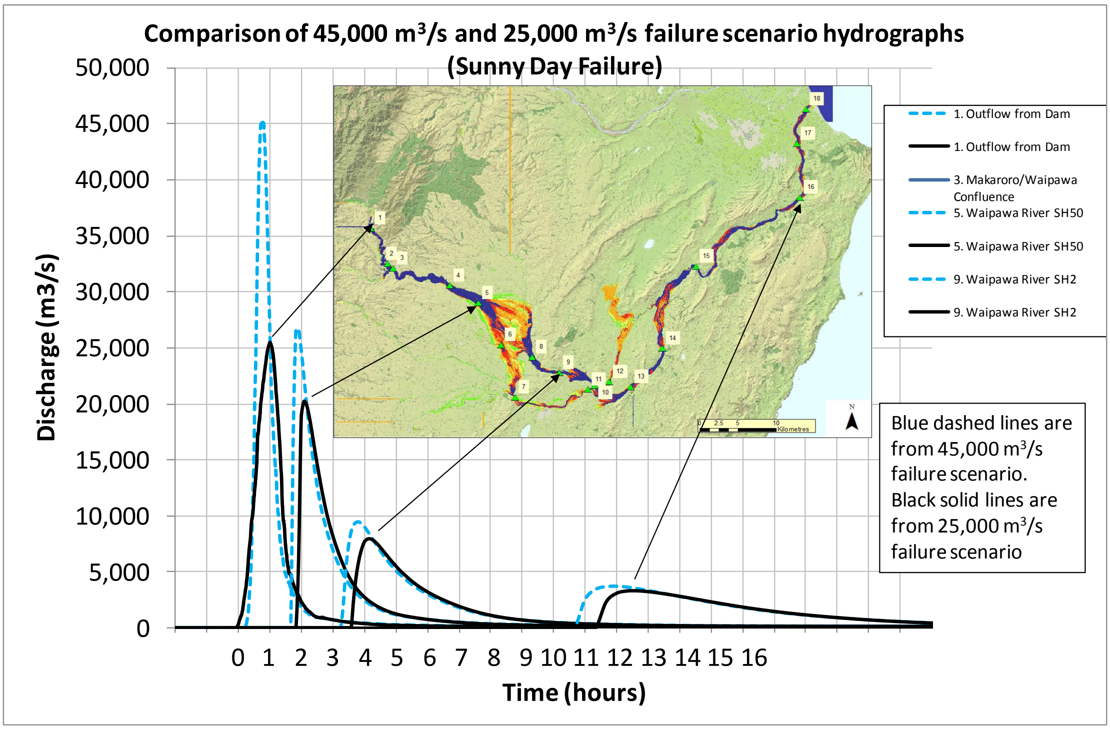

Figure 29: Discharge hydrograph comparison for 45,000 m^3/s and 25,000 m^3/s scenario, sunny day failure

As either of the flood scenarios shown in Figure 29 travel down the river system, the natural attenuation of the river system causes the resulting flood waves to converge to similar values. This helps explain why there are only minor differences in flood extents for the 25,000 m^3/s failure scenario, as compared to the 45,000 m^3/s scenario. Since the results are very similar, no further results from the 25,000 m^3/s scenario runs are included in this report. Output from these scenarios are available as GIS layers.

6.3 Rainy Day 1-Hour Failure

As part of the 'Rainy Day' scenario, a moderately severe background flood with a return period of approximately 20 years was included in the model run. In order to produce results which included the combination of the dam break and background flood, the background flows were input to the model as constant inflows equal to the peak flood flow over the whole duration of the model run. This ensured the coincidence of peak background flood and peak dam failure flows, albeit in a conservative manner. This is considered to be conservative because the likelihood of a dam failure in which the peak dam break flow occurs at any location in the catchment at the exact same time as the background flood is peaking is extremely small, since both flood events (the dam failure and the background flood) have relatively short durations. . Figure 30 shows the discharge values which were applied to the model as constant inflows. The actual values of the discharges applied are not overly significant, since the discharge value from the dam failure (49,000 m^3/s) is many times greater than the background flow values.

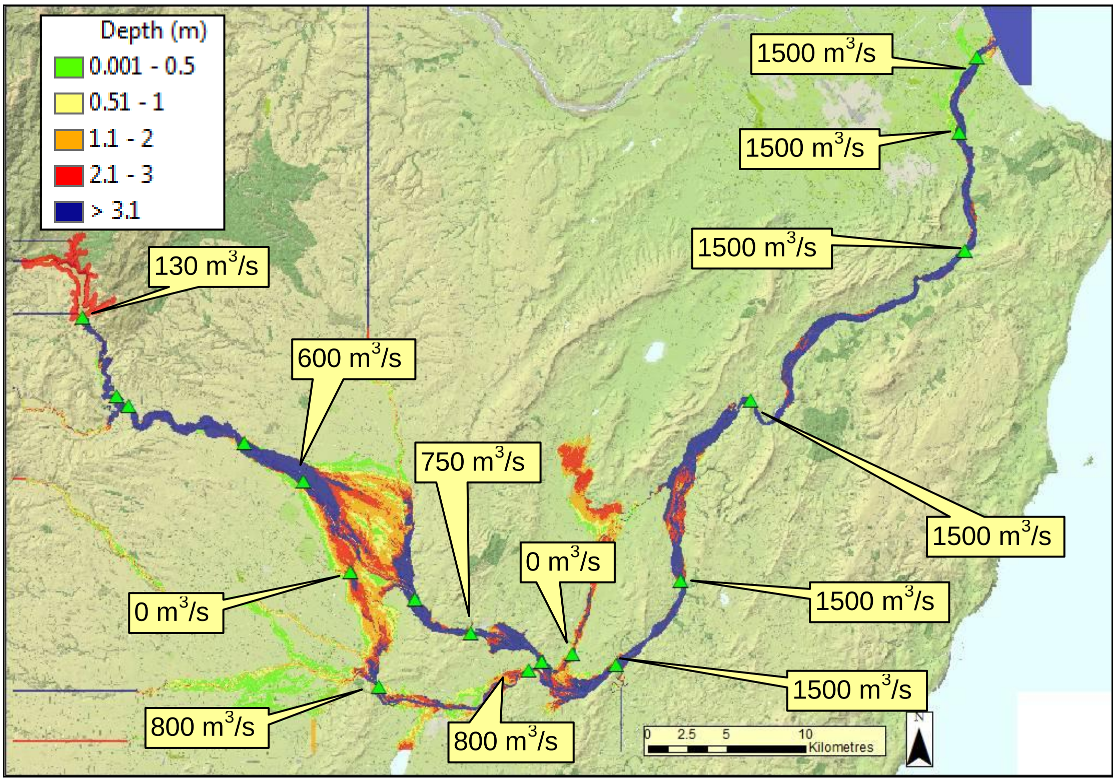

Figure 30: Constant discharge values in rivers for rainy day failure, 49,000 m^3/s scenario

The discharge values shown in Figure 30 were applied to the model such that when the dam failure was induced, these flows were already present in the river network. As a result of having a reasonable sized flood already occurring in the channel, the travel times for the dam break flood wave are shorter than for the sunny day scenario. As an example, the travel time for the flood wave from the dam site to Black Bridge is approximately 13 hours 40 minutes for the sunny day failure, and about 11 hours for the rainy day failure. For this distance of approximately 116 km, the average velocity for the sunny day failure is 2.9 m/s, while it is 2.4 m/s for the rainy day failure. This difference in overall speed is not a critical factor in the dam break analysis, and both scenarios produce almost identical flooding extents.

The discharge hydrographs for the rainy day failure are shown in Figure 31.

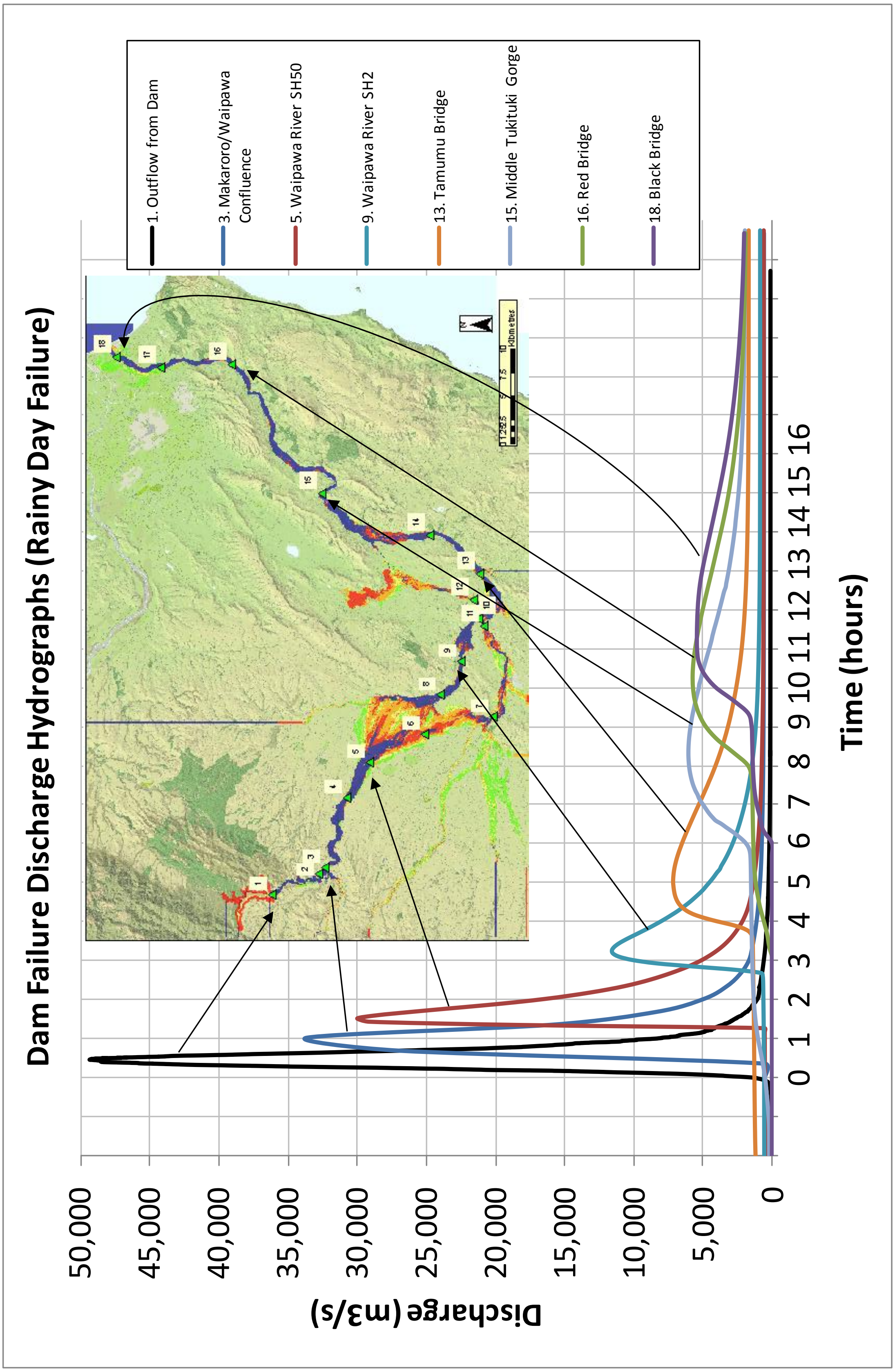

Figure 31: Discharge hydrographs rainy day failure, 49,000 m^3/s scenario.

The results showed relatively small differences in flood extents from the rainy day scenario as compared to the sunny day scenario. Figure 32 shows the outline extents of flooding for the two scenarios at the location with the greatest difference, near Waipukurau.

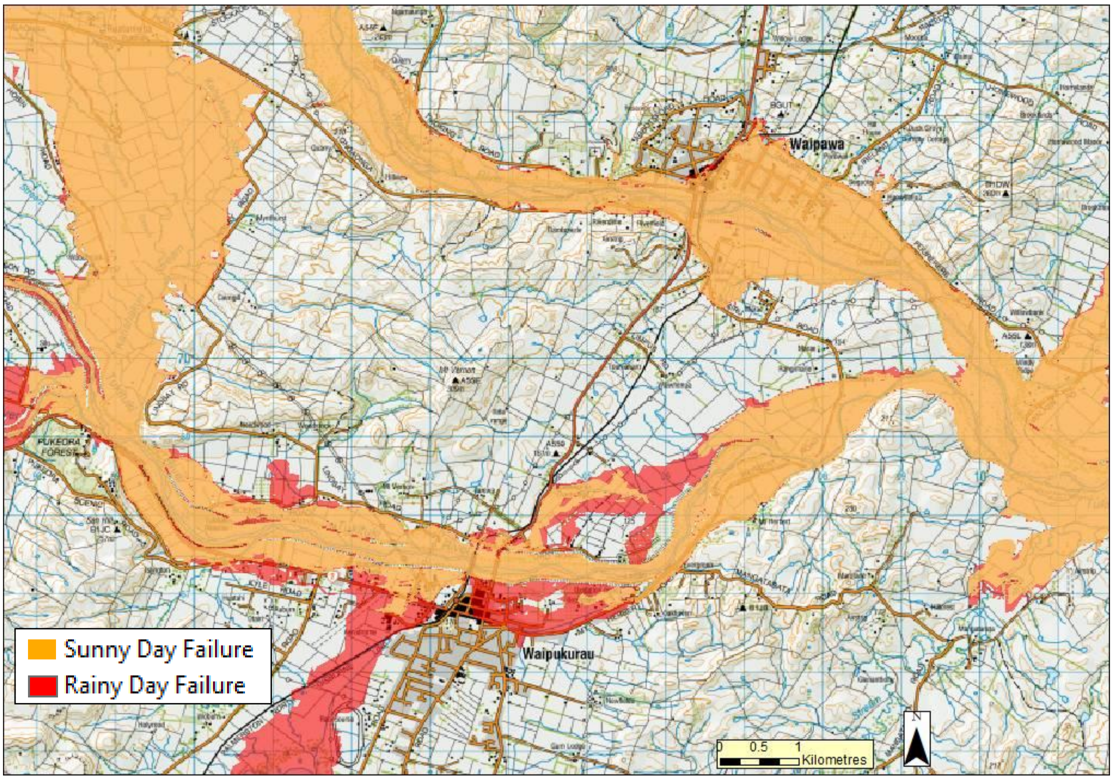

Figure 32: Differences between sunny and rainy day failures (both 1-hour failure scenarios)

The flood depths at most locations were approximately 0.5 m higher for the rainy day failure, as compared to the sunny day failure. However, in most cases this does not translate to significantly greater extents, with the exception of the area near Waipukurau as shown in Figure 32. The differences are not overly significant, considering the overall impact the potential dam failure may bring about on the catchment.

6.4 Rainy Day 2-Hour Failure

Results from the rainy day 2-hour failure scenario, with a peak discharge of 27,500 m^3/s were also very similar to the 1-hour failure (49,000 m^3/s peak). Since the results are very similar, no further results from these runs are included in this report. Output from these scenarios are available as GIS layers.

6.5 Morphological Changes During Dam Break

The dam break analysis carried out for this report does not include consideration of any morphological change to the river bed during the event, nor does it include sediment and other debris which may get deposited in the river bed and cause alternative paths for the flow. An example of this may be that a large blockage occurs on the Waipawa River, causing the main flow to deviate to the Tukituki River.

Such scenarios are plausible and would result in slightly different flood extents for the modelled dam break scenarios. However, the PIC rating for the proposed dam would not change.

Thought has been given to a variety of scenarios which could result in different flood extents, all with results which indicate catastrophic effects to rural areas as well as particular urban areas of Waipukurau and Waipawa. The existing ground elevation and slopes tend to concentrate the flow of water to the same areas regardless of morphological changes, with the resulting difference being some areas experiencing less flooding, at the expense of other areas incurring greater flooding. In developing an Emergency Action Plan for the dam, additional dam break analyses could be carried out to determine if other properties should be included within the plan. Such a plan should ideally have a buffer zone and include any and all of the potentially affected properties, taking into account the existing potential flow paths.

7 Infrastructure at Risk

There are approximately 16 bridges, two sewage treatment plants with oxidation ponds, as well as many kilometres of roads along the flood path which would likely suffer damage in the dam break scenarios modelled. Figure 33 shows the location of the main bridges and oxidation ponds.

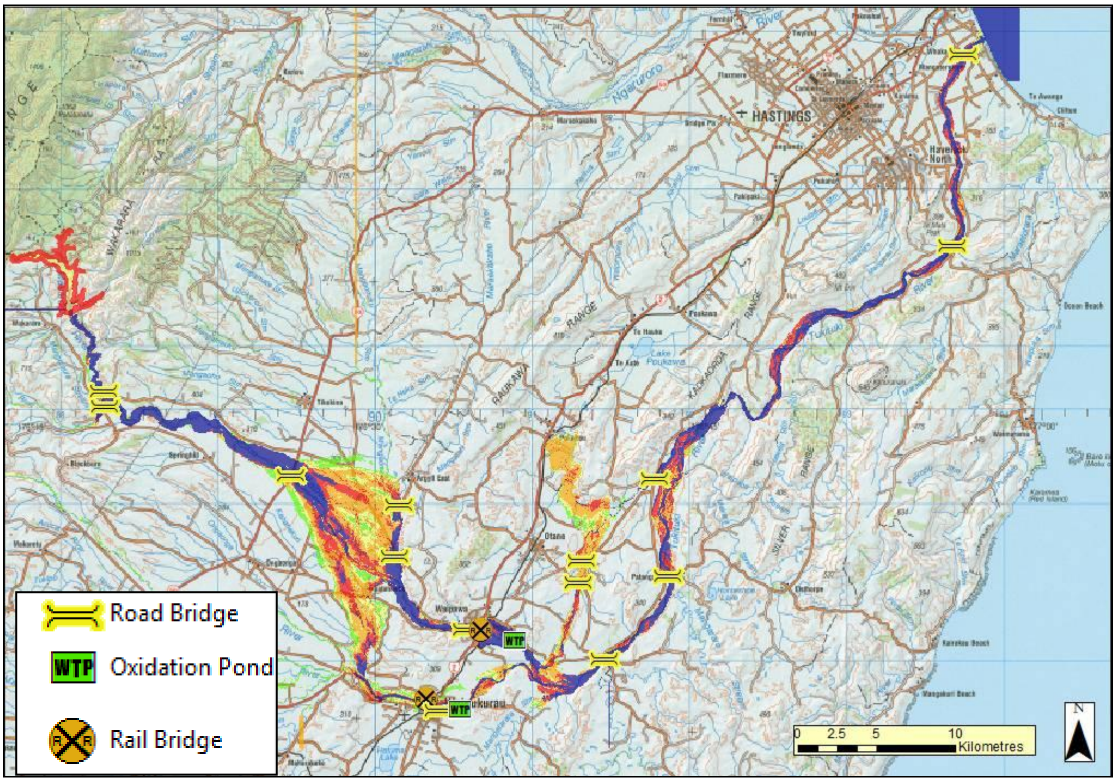

Figure 33: Locations of bridges or infrastructure at risk, sunny day failure, 45,000 m^3/s

While no detailed investigation has been done to examine the potential effects on the infrastructure, it is anticipated that significant to severe damage would likely result in most of these structures.

The actual consequences of this type of failure is likely to be costly repairs and rebuilding. However, the actual disruption to the population would likely be limited to the towns of Waipawa and Waipukurau.

8 Population at Risk (PAR)

It is necessary to estimate the population-at-risk (PAR) in order to assign the dam a potential impact category (PIC). A preliminary count of houses at risk in the floodable areas indicated approximately 373 houses flooded to a depth of greater than 0.5 m. The 2006 New Zealand Census estimates the number of persons per household to be approximately 2.7, giving a population-at-risk of approximately 1000. The flood extents in Waipawa and Waipukurau are shown in Figure 34 and Figure 35, along with property boundaries, and air photos showing the residential houses at risk.

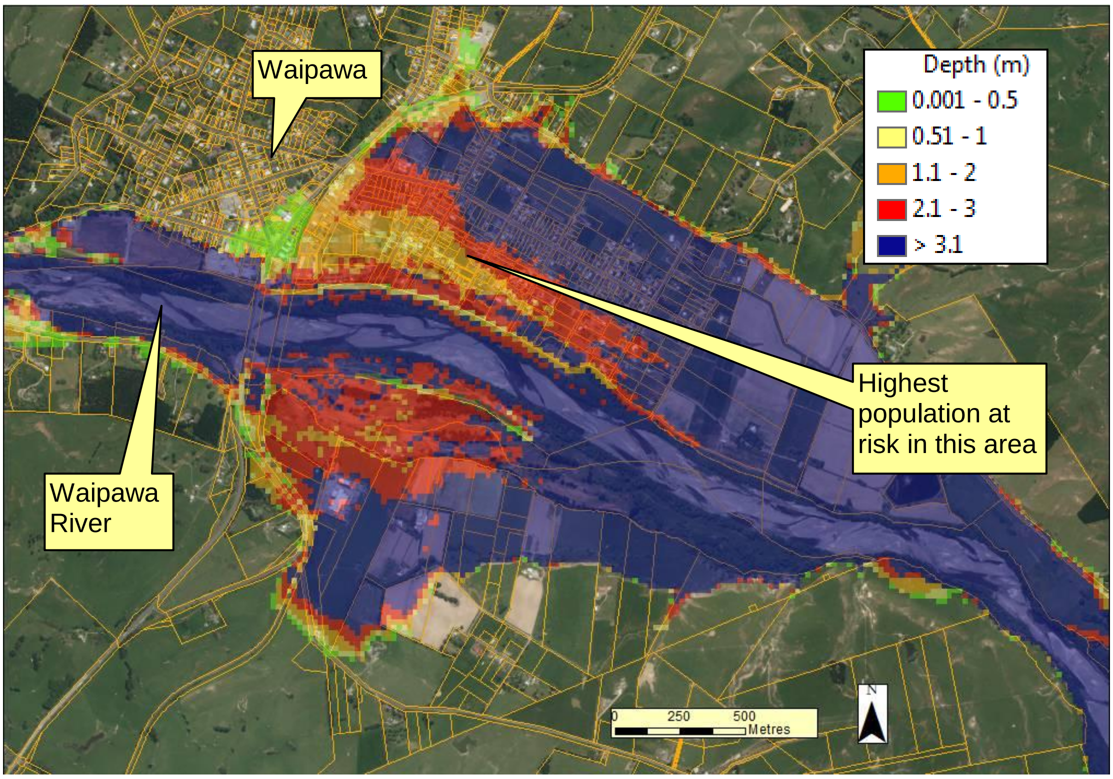

Figure 34: Waipawa, showing properties to indicate population-at-risk, sunny day failure, 45,000 m^3/s

Note that the dam hazard classification system (refer to Section 9) considers a property to be at risk when the flood depth is greater than 0.5 m. Thus, properties within the areas shaded green (flood depths from 0.001 to 0.5 m) in the preceding flood maps are not at risk, but have been included to assist in determining the likely greatest extent of flooding.

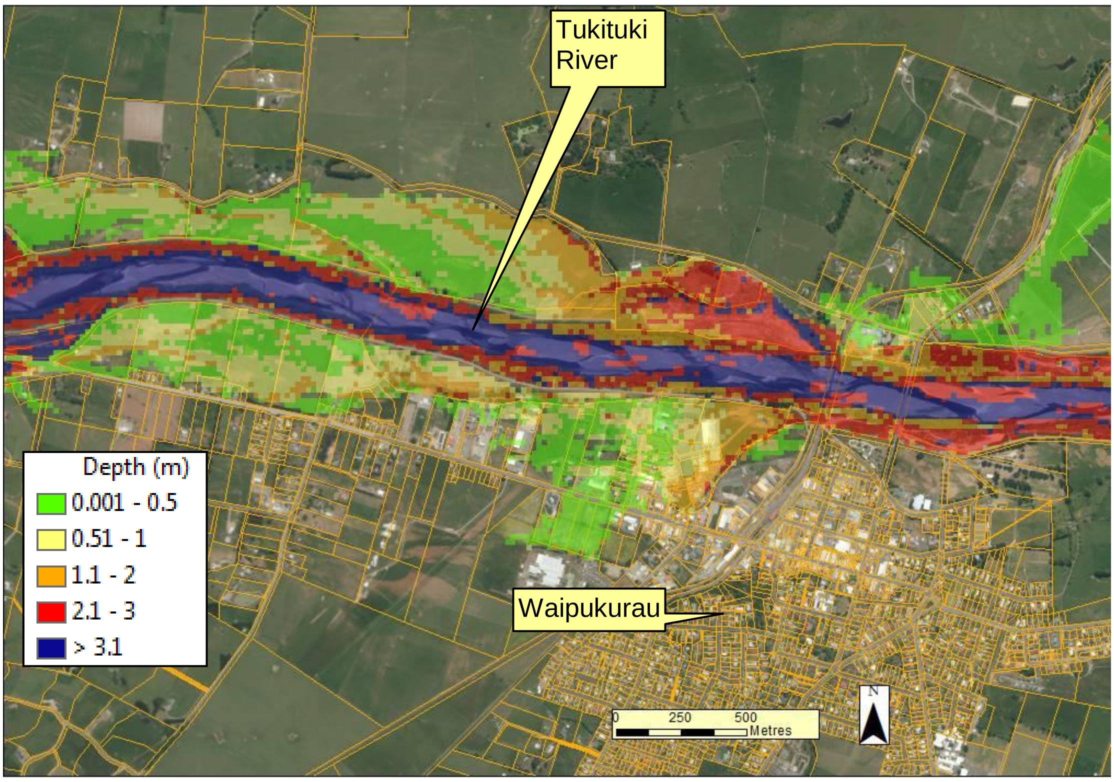

Figure 35: Waipukurau, showing properties to indicate population-at-risk, sunny day failure, 45,000 m^3/s

The following table shows the rough distribution of the population-at-risk, with highest concentration being located in Waipawa on the left bank of the Waipawa River, downstream of the SH2.

Location	Estimate of houses at risk*	Population estimate (based on 2.7 person/house)
Left Bank Waipawa River d/s SH50	18	49
Right Bank Waipawa River d/s SH50	16	43
Waipawa Town	316	853
Waipukurau Town	16	43
Other	7	19

*Note that the estimate of houses at risk does not take into account actual floor levels which may be substantially higher than ground levels. Thus, the actual number of houses at risk may be less than stated.

A reminder is provided here that the derivation of population-at-risk is a hypothetical exercise in order to determine the appropriate design standard for the proposed dam. In this case, there is a population-at-risk of greater than 100 persons, which indicates that dam will have a high potential impact. Since this risk exists, the dam has been designed to a standard that provides the highest level of protection to the population-at-risk.

9 New Zealand Dam Safety Guidelines

The Building (Dam Safety) Regulations (2008) (currently deferred until July 2014 by the Government) for New Zealand provide guidance for the safe planning, construction and operation of dams in New Zealand. Dams are placed in categories of potential impact based on several criteria. The first criterion examines the potential damage level, with the definitions shown in Table 1.

Damage level	Residential houses	Critical or major infrastructure		Natural environment	Community recovery time
		Damage	Time to restore to operation		
Catastrophic	More than 50 houses destroyed	Extensive and widespread destruction of and damage to several major infrastructure components	More than 1 year	Extensive and widespread damage	Many years
Major	4-49 houses destroyed and a number of houses damaged	Extensive destruction of and damage to more than 1 major infrastructure component	Up to 12 months	Heavy damage and costly restoration	Years
Moderate	1-3 houses destroyed and some damaged	Significant damage to at least 1 major infrastructure component	Up to 3 months	Significant but recoverable damage	Months
Minimal	Minor damage	Minor damage to major infrastructure components	Up to 1 week	Short-term damage	Days to weeks

In relation to residential houses, "destroyed" means rendered uninhabitable.

Critical or major infrastructure includes:

1. Lifelines e.g. power supply, water supply, gas supply, transportation systems, wastewater treatment.
2. Emergency facilities e.g. hospitals, police, fire services.
3. Large industrial, commercial, or community facilities, the loss of which would have a significant impact on the community.
4. The dam, if the service the dam provides is critical to the community and that service cannot be provided by alternative means.

Table 1: Categories of Assessed Damage Level (Building (Dam Safety) Regulations 2008) (Red outline indicates the assessment for the Makaroro Dam)

The results of the analysis shown in preceding parts of this report indicate that the potential damage level would be in the major or catastrophic category, since at least 50 houses could be flooded to depths of around 3 m. The damage to major infrastructure is likely to involve road and rail bridges, as well as flood protection stopbanks. However, these are not critical 'lifeline' features, and alternative routes and services are likely to be able to be implemented in relatively short notice. Despite this, the most appropriate category for the infrastructure component should be major or catastrophic. Damage to the natural environment may also be widespread, with the likely consequences being erosion of river channels and deposition of silt and debris after the flood wave has passed. Again, this indicates an assessed damage level of major/catastrophic since the recovery time is likely to be greater than several months.

Table 2 provides guidance on the Potential Impact Category (PIC) to select, based on the population-at-risk, and the assessed damage level.

Assessed damage level	Population at risk (PAR)			
	0	1-10	11-100	100+
Catastrophic	High PIC	High PIC	High PIC	High PIC
Major	Medium PIC (see note 4)	Med/High PIC (see note 4)	High PIC	High PIC
Moderate	Low PIC	Low/Med/High PIC (see notes 3 and 4)	Med/High PIC (see note 4)	Med/High PIC (see notes 2 and 4)
Minimal	Low PIC	Low/Med/High PIC (see notes 1, 3 and 4)	Low/Med/High PIC (see notes 1, 3 and 4)	Low/Med/High PIC (see notes 1, 3 and 4)

Notes:

1. With a PAR of 5 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 2 or more lives will be lost

Table 2: Potential Impact Category Table (Building (Dam Safety) Regulations 2008) (Red outline indicates the assessment for the dam)

For the above table, the PAR is defined in the Regulations as all those people who are likely to be affected by flood depths in excess of 0.5 m. For the proposed dam, the PAR is greater than 100 people.

Based on these descriptions, the dam is considered to be in the HIGH potential impact category. In addition to minimum standards for design, a high PIC dam will require an appropriate Dam Safety Assurance Programme (DSAP) under the Regulations. The DSAP will need to include an Emergency Action Plan (EAP) that will detail the actions that the owner, operations personnel and relevant Government and Local Authorities should take if an incident or emergency develops that threatens the safety of the dam. The format and detail required in the EAP is set out in the New Zealand Dam Safety Guidelines (NZSOLD, 2000). Both the DSAP and EAP will be required prior to commissioning of the dam.

10 Limitations

The results presented in this report are based on computer models which provide an output based on the input parameters chosen. While care and research has gone into selecting the input parameters, in the unlikely event of a dam failure, there are many unknown variables which could alter the parameters and produce different results in terms of flood extents and timing of the flood wave. The scenarios examined and the range of parameters chosen have given a range of results, all of which comprise a set of extremely severe scenarios which are hypothetical and unlikely to ever occur.

11 Conclusion

A concrete-faced rockfill dam for water storage approximately 83 m high is proposed to be constructed on the Makaroro River. This report examined the potential downstream effects in the event of a hypothetical failure of the dam by using a 2 dimensional hydrodynamic computer model.

As mentioned earlier in this report, the dam break analysis is undertaken in order to determine the potential downstream effects, which in turn guides the setting of standards to adopt for the dam design, construction and operation. In this particular case, the dam design incorporates features which are designed to prevent a catastrophic failure from occurring, including adoption of the highest design standards in the dam building industry. While this may seem to make the results from the dam break analysis redundant, it is important to consider the dam break results, particularly in terms of an Emergency Action Plan, in the highly unlikely event of imminent failure of the dam.

The dam break modelling results show significant out of channel flow occurs across the Ruataniwha Plains from the Waipawa River; near the towns of Waipawa and Waipukurau; down the old bed of the Waipawa River to the Papanui Stream. The effects from such a failure would likely be severe in terms of damage to infrastructure, and would include a population-at-risk (flooded to a depth greater than 0.5 m) of approximately 1000 people.

The analysis has shown that the dam should be classified as having a potential impact category of ‘High’, due to the population-at-risk being greater than 100, and the likely damage to infrastructure being major or catastrophic.

Output from the analysis is also available as layers to be used in a geographic information system, which in turn can be incorporated into an Emergency Action Plan for the dam.

12 References

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