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1 September 2025

**Sepik Development Project – Dam Break Risks Assessments**

PanAust Limited (**PanAust**) is committed to meeting its domestic and international legal obligations and strives to exceed the expectations of communities with which it works. As part of its commitment to ensuring that the stakeholders of the Frieda River Project are informed, PanAust releases publicly the following attached technical reports relating to the Frieda River Hydro-Electric Project (**FRHEP**).

- The first report is the 2024 Technical Report prepared by SRK Consulting (**2024 Report**).
- The second report is a 2018 Project Selection Phase Selection Assessment (**2018 Report**), also prepared by SRK Consulting.

The 2024 Report supersedes the 2018 Report and is the current assessment of dam embankment risks for the FHREP**.**

**2024 Report**

This Report follows a standard interrogation process for all large dam projects, with independent experts required to consider failure scenarios and their potential impacts to inform necessary mitigation measures and emergency planning. The 2024 Report is the current dam break analysis for the FRHEP, as it is based on the current design of the dam. It is more comprehensive than the 2018 Report.

- The 2024 Report analyses the risks of a failure of a potential dam embankment and identifies potential mitigants to those risks.

- It considers a large amount of additional information, including dam design and information about historical flood levels and demographics of villages downstream.

- It models the potential consequences of a minor, partial or large-scale breach of the potential dam embankment at various times of year.

- It identifies steps to be taken to enhance the security and safety of the potential dam embankment used in the model.

- It explains where its updated analysis differs from the 2018 Report.

- It incorporates safety and engineering features adopted since the 2018 Report to mitigate risks.

The purpose of the 2024 Report is to assess a potential dam embankment failure to identify areas where further mitigants are required and additional engineering works need to be undertaken. It is not a model of the risks of the final dam for the FRHEP, which will be completed as part of the final dam design process.

**2018 Report**

The 2018 Report was preliminary in nature and was conducted to inform the initial design selection of the Frieda River Hydroelectric Project and has been superseded by the 2024 Report. The 2018 Report covers the following.

- It is an inundation report and not an assessment of whether the dam will break. It shows potential effects of instantly releasing certain amounts of water from a generic dam.
- The model does not take into account the dam wall profile or partial releases of water.
- It does not model solids flows, which slow down the flow of water.
- It does not model the effect of sediment on water depths and the time water will take to spread.
- It does not include any safety systems or protections, which are included in modern dams, against the flow of water.
- Together, these inputs mean that the model is intentionally conservative and does not reflect the full physical environment surrounding the FHREP. This approach results in faster flow times and deeper water levels than is possible in the subsequent design adopted for the dam, as analysed in the 2024 Report.

The purpose of the 2018 Report was to provide a preliminary baseline scenario where a generic dam failed and no extra safety or engineering steps were taken to mitigate flow of water. It was not intended to model the risks of the final dam for the FRHEP, but rather a discovery step to inform the necessary requirements of the design.

**Further information**

Further information on the Frieda River Project and the FRHEP in English and Tok Pisin are available online at: https://www.friedariver.com/media-centre/fact-sheets/

PanAust values community engagement and consultation with all stakeholders about the Frieda River Project. For any enquires in relation to the technical reports, please contact info.friedariver@panaust.com.au

Yours faithfully,

Philip McCormack

**Philip McCormack**
Director, Frieda River Limited

***Annexures***

- 2024 Technical Report prepared by SRK Consulting.
- 2018 Project Selection Phase Selection Assessment, also prepared by SRK Consulting.

FINAL

# FRHEP Dam Risk Assessment – Technical Report

FRHEP, Frieda River, Papua New Guinea

PanAust Limited

SRK Consulting (Canada) Inc. ▪ CAPR003035 ▪ September 2024

srk consulting

**FINAL**

# FRHEP Dam Risk Assessment – Technical Report

FRHEP, Frieda River, Papua New Guinea

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## Appendices

Appendix A Flooding and Velocity Overview Maps
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# Executive Summary

SRK was commissioned by PanAust Limited to analyze the risks of a failure of the FRHEP embankment leading to an uncontrolled release from the facility and potential impacts on downstream communities. This document details the analysis methodology and results.

The general steps of the analysis included:

- Reviewing experience at other dams
- Estimating the probability of a FRHEP embankment failure using statistical methods
- Estimating the potential downstream consequences of a FRHEP embankment failure.

The key results were as follows:

- The experience at other dams shows that there are a number of dams in the world that have performed well in similar environments to the FRHEP. Many dams have experienced and survived large earthquakes with little to no damage, and there are also many dams that have been designed for earthquake conditions similar to those expected at the FRHEP.
- The statistical methods used to estimate the probability of failure of the proposed FRHEP embankment showed that with high quality design, construction, operations and governance, the FRHEP embankment can achieve probabilities of failure that are within the range of good hydro-electric dams elsewhere in the world. However, if these high standards are not met, then the dam could have a higher and unacceptable risk of failure. Therefore, the adoption of high standards is critical
- The analysis of possible downstream flooding and fatality risks in the event of a FRHEP embankment breach showed that flood depths depend primarily on the magnitude of the breach and can range from within natural flooding levels to several meters above the downstream riverbanks. Estimated flood arrival times at downstream communities range from hours to days, and therefore the risk of downstream fatalities can be controlled by community warning system and emergency response plans.

Based on these findings, ensuring that failure probabilities and downstream risks associated with the FRHEP embankment remain within tolerability limits will require both:

- high quality design, construction, operations and governance; and
- an effective warning and response system.

Other regions subject to a dam failure risk develop emergency response plans that include alarm systems and local responses such as evacuation or access to higher ground. We recommend developing alarm systems and community response plans with the communities downstream of the FRHEP embankment.

# 1 Introduction

## 1.1 Scope of Report

Frieda River Limited (FRL), a wholly owned Papua New Guinea (PNG) registered subsidiary of PanAust, is assessing the feasibility of the Frieda River Project in northwest PNG. The project has four interdependent elements – the Frieda River Copper-Gold project, the Frieda River Hydroelectric Project (FRHEP), the Sepik Infrastructure project, and the Sepik Power Grid project. The proposed FRHEP dam and reservoir will supply power for mine-related activities and third-party customers, and the reservoir will also store the mine tailings and waste rock.

SRK was commissioned by PanAust Limited to analyze the risks of a failure of the FRHEP embankment leading to an uncontrolled release from the facility and potential impacts on downstream communities. The results of the analysis will be used to support legal advice relating to disclosure and consultation requirements for the project under PNG law. This technical report details the risk analysis methodology, key results, and recommendations. A Plain English summary report will also be provided.

## 1.2 FRHEP Background

The FRHEP is located in the Sandaun Province of northwest PNG as indicated in Figure 1. The project site is in the remote Thurnwald Range, in mountainous and rugged terrain with dense rainforest. The site is subject to high rainfall and moderate-to-high seismicity, with average annual rainfall on the order of 8,000 mm and estimated 475-year seismic ground accelerations of 1.6-2.4 $m/s^2$.

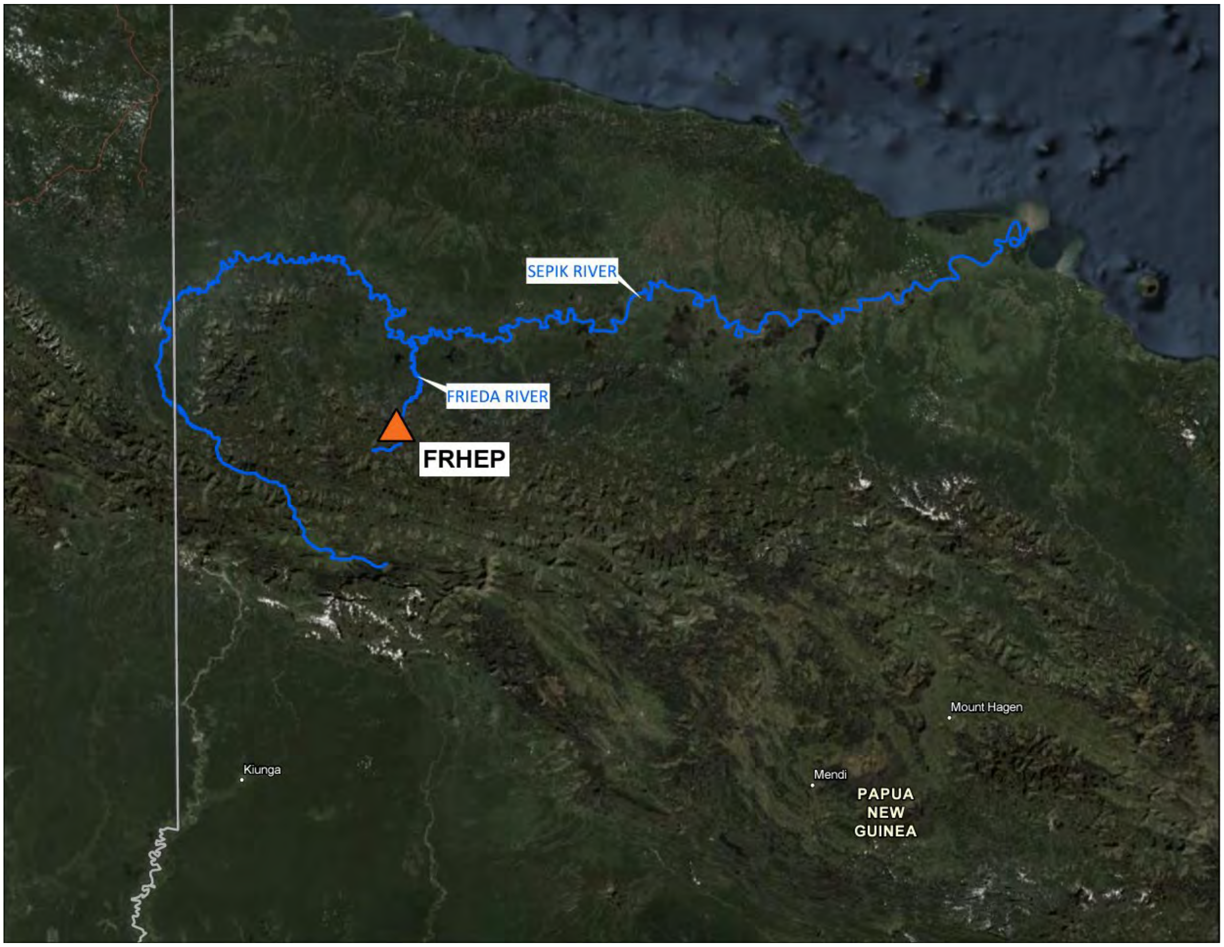



**Figure 1: FRHEP location**

The Sepik River is the primary drainage for the region, as indicated in Figure 2. The FRHEP is proposed to be located on the Frieda River, a tributary of the Sepik. The Wario River runs parallel to and just east of the Frieda, and could also be impacted by outflows from a failure of the FRHEP embankment. There are several communities in the potential zone of inundation along the Frieda, Wario, and Sepik Rivers, with a total population in the order of 3,500 people. Housing is typically constructed from wood, thatch and palm leaves and includes raised platforms to accommodate river flooding. Community livelihoods are closely tied to the river, so villages are relocated as the channel changes with time.

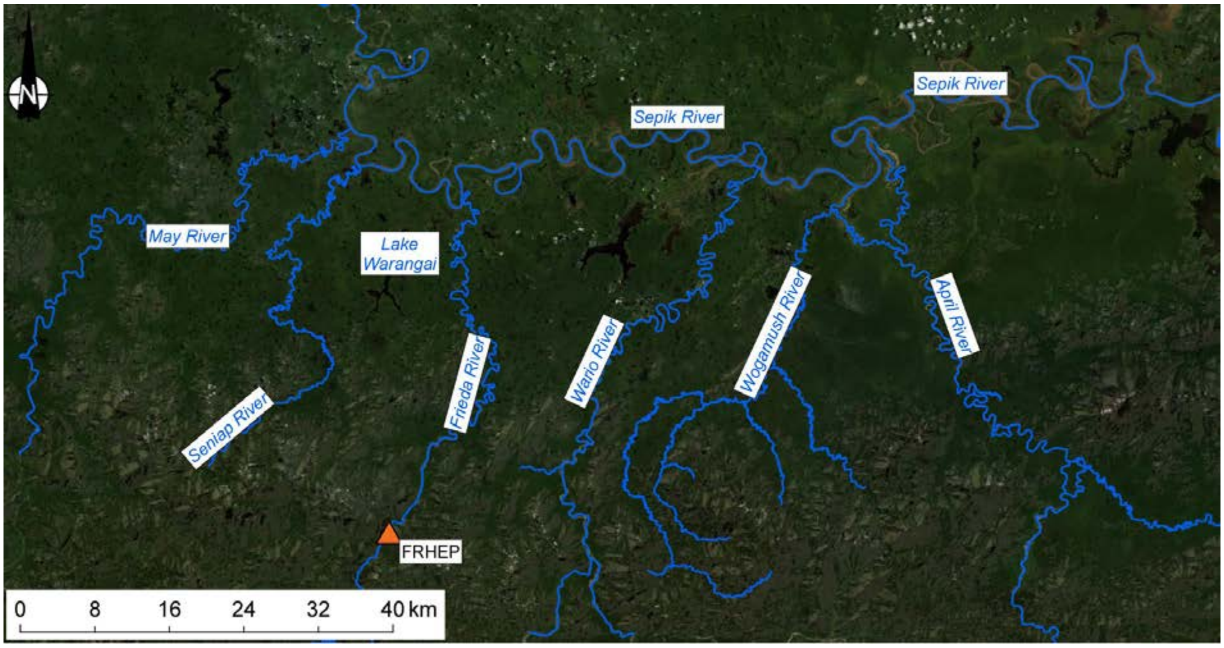



**Figure 2: Major rivers in the study area**

The FRHEP reservoir will be created by construction of an embankment within a narrow reach of the Frieda River. The proposed embankment is a 190m high rockfill structure with an asphalt core. A typical section is shown in Figure 3.

The FHREP reservoir will have a total capacity of roughly 9.6 B $m^3$. Over the life of the mine, a combined volume of 2.13B $m^3$ of mine tailings and waste rock is proposed to be deposited in the reservoir. As indicated in Figure 4, the deposition plan includes keeping the waste rock more than 1000 m from the dam, and placing tailings only in the upper reaches of the reservoir behind barrier of deposited waste rock. Expected water, waste and tailings levels are compared to the FRHEP embankment elevation in Figure 5.

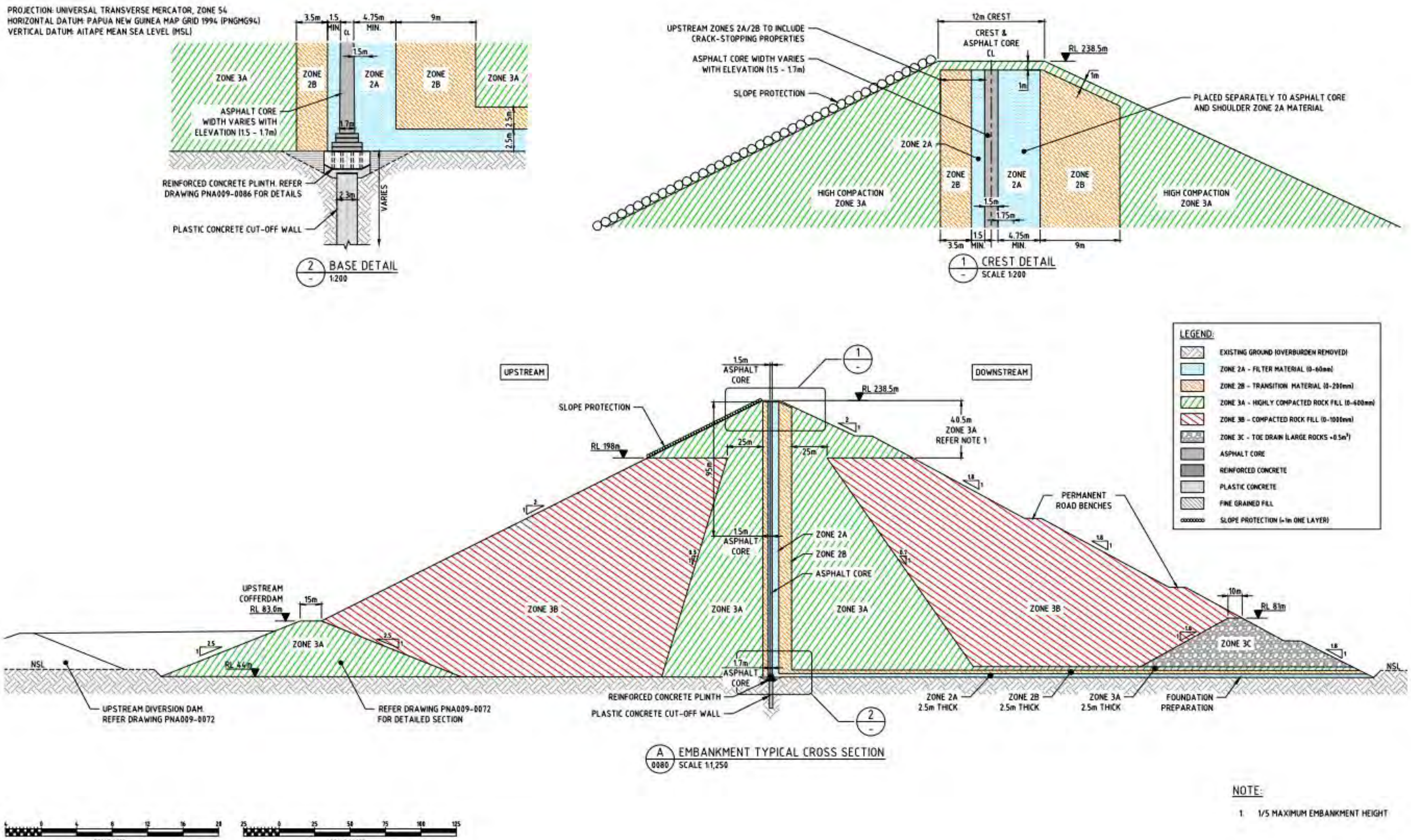



**Figure 3: Design typical section through FRHEP Embankment**

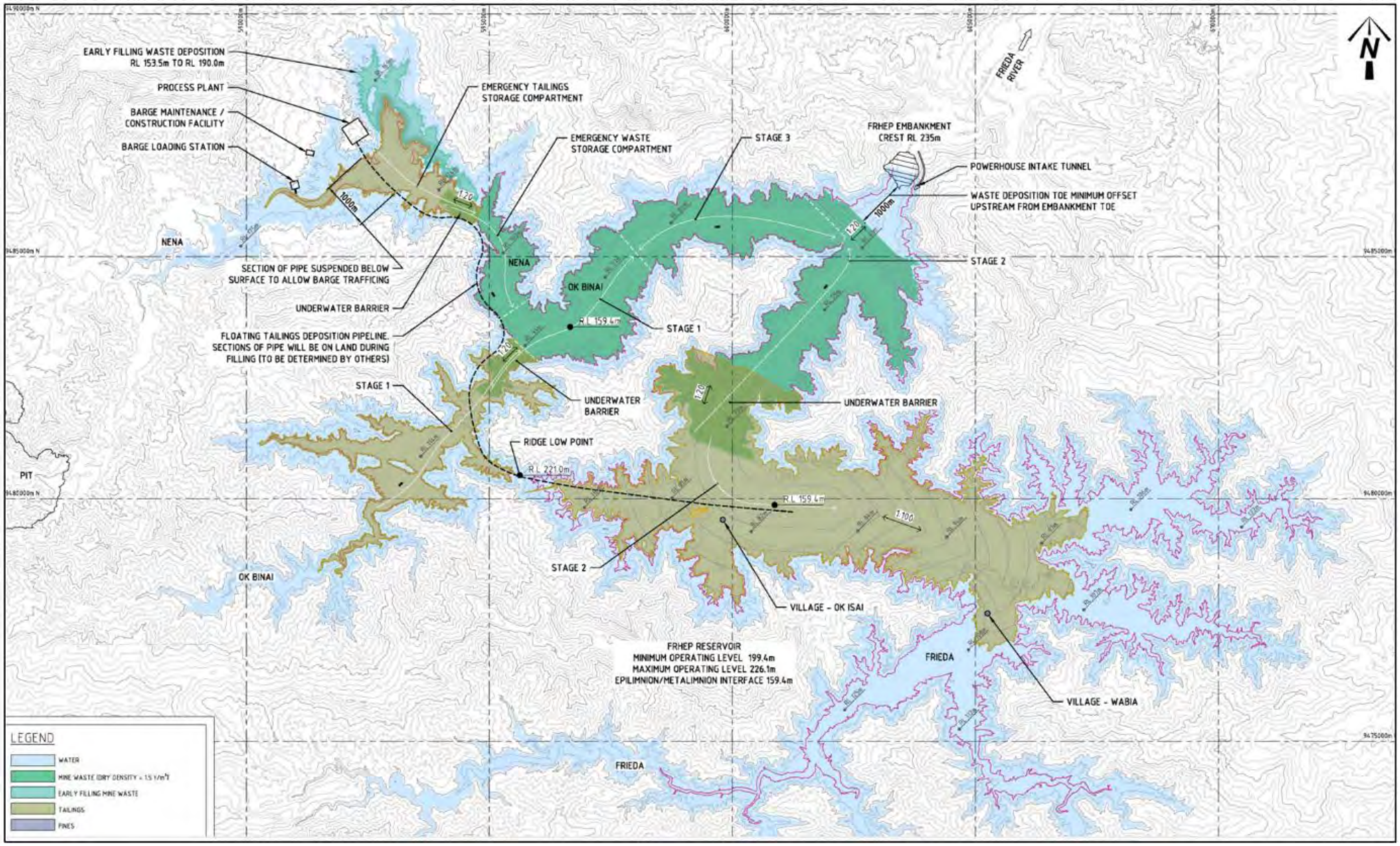



**Figure 4: FRHEP reservoir footprint illustrating proposed waste rock and tailings locations**


**Figure 5: Proposed FRHEP water, waste and embankment elevations**

# 2 Methodology

The general steps adopted for the FRHEP dam failure risk assessment included:

- Reviewing experience at other dams
- Estimating the probability of a FRHEP embankment failure
- Estimating the potential downstream consequences of a FRHEP embankment failure.

These steps are described in more detail in the following sub-sections.

## 2.1 Review of Experience at Other Dams

### 2.1.1 Global dam failure statistics

Global statistics on hydroelectric dams and dam failures were assembled from the sources listed in Table 1. The ICOLD Bulletin 99 database is the most comprehensive and includes organizational and technical causes of failure. Table 2 and Table 3 show the cause descriptions included in the ICOLD database and the grouping of causes used for the FRHEP analysis.

**Table 1: Global Dam Failure Databases**

| | Number | Information |
|---|---|---|
| USCOLD Dams and Incident Database (2022) | 27,455 dams and 1244 incidents (444 failures) | ▪ Dam information including height, age, type, location<br>▪ Incident descriptions |
| ICOLD World Register of Dams (2024) | 62,353 dams | ▪ Dam information including height, age, type, location, core type |
| ICOLD Bulletin 99 Update (2019) | 321 failures | ▪ Dam information including height, age, type, location<br>▪ Detailed categorized failure causes |
| Dam Failure Mechanisms and Risk Assessment (2016) | 1443 incidents | ▪ General dam information: height, age, type, location<br>▪ Most are from USCOLD incident database, contains some additional information on Chinese failures not present in other databases |

**Table 2: ICOLD Organizational Causes and Grouped Causes Used Herein**

| ICOLD Symbol and Descriptor | | FRHEP Study |
|---|---|---|
| BD | Design insufficiencies | Design |
| BC | Construction insufficiencies | Construction |
| BM | Maintenance or surveillance | Operation & Maintenance |
| BO | Inadequate operation (including spillway gates) | |
| NN | None or unclear | None identified |

**Table 3: ICOLD Technical Causes and Grouped Causes Used Herein**

| ICOLD Symbol and Descriptor | | FRHEP Study |
|---|---|---|
| GC | Geotechnical issues | Geotechnical |
| ST | Structural issues | Structural |
| MA | Material ageing | Material Aging |
| IF | Overtopping (OT) due to inadequate freeboard | Inadequate Capacity |
| IA | Overtopping (OT) due to inadequate available capacity | |
| II | Overtopping (OT) due to inadequate installed capacity | |
| HF | Hydromechanical equipment malfunction or failure (including loss of power supply) | Hydromechanical Malfunction |
| UN | Unknown | Unknown (left blank) |

### 2.1.2 Observations of Dam Performance

The FRHEP embankment is located in a moderate-to-high seismic region and a key consideration is how it will perform under seismic loading. The global dam failure statistics were therefore supplemented by an additional survey focused on how dams with similarities to the FRHEP embankment have performed (or have been designed to perform) under significant seismic loads.

The survey included the following:

- Major dams in Papua New Guinea
- Asphalt core and rockfill dams located in high seismic areas, prioritizing high dams (e.g. over 100 m)
- Other dams that had experienced significant earthquake events and had documented their performance before and after the event.

Information sources included USGS databases (2024), Furumura et al (2011) for specific Japanese events, and the Swaisgood (2003) database.

## 2.2 Estimating FRHEP Embankment Failure Probabilities

Ranges of failure probabilities were estimated using the available global failure statistics and applying the following two steps:

1. First, estimate baseline probabilities of failure for all dams higher than 15 m and for asphalt core dams.
2. Second, assess the significance of other factors such as seismicity and dam quality, and estimate adjustments to the baseline probabilities of failure where warranted.

These calculations are described in more detail in the next two subsections.

### 2.2.1 Estimating Baseline Probabilities of Failure

The methodology to estimate baseline probabilities of failure comprised the following steps:

1. Preprocess the ICOLD database to remove dams under 15 m high and to separate out dams with asphalt cores.
2. Estimate baseline failure rates[1] for all dams directly from the pre-processed ICOLD data.
3. Estimate benchmark failure rates for asphalt core dams using recommendations from NASA's probabilistic risk assessment guidelines (NASA 2011).

The latter step warrants further explanation. The NASA guidelines recommend Bayesian probability methods to estimate failure rates for systems that may have limited data or no observed failures to date, using the general form of Bayes' theorem:

$$Posterior\ distribution = \frac{Prior\ Distribution\ \times Likelihood}{Normalizating\ Constant} \propto Prior\ Distribution\ \times Likelihood$$

The prior distribution in this case is the distribution of failure rates of all dams from the pre-processed ICOLD database. The likelihood distribution is derived from the performance of asphalt core dams and takes into account the limited number of such dams, their limited years of operation, and the fact that there have been no failures (to date). The posterior distribution then provides the expected (future) failure rates for asphalt core dams. Using recommendations from NASA (2011), conjugate prior and likelihood distributions were selected to allow the posterior distribution to be solved analytically:

- Prior distribution: Gamma (α, β), where α is the number of all dam failures observed over β dam-years.
- Likelihood distribution: Poisson (k, T) where k is the number of observed asphalt core dam failures over T dam-years.
- Posterior distribution: Gamma with mean: $\bar{\bar{x}} = \frac{\alpha+k}{\beta+T}$ and variance: $var(x) = \frac{\alpha+k}{(\beta+T)^2}$

The key inputs and results are from both steps 2 and 3 above are presented in Section 3.2.1.

### 2.2.2 Estimating Adjustments to Baseline Probabilities of Failure

There are many factors that will affect a particular dam's probability of failure including: specific dam characteristics and design choices; site conditions such as climate, seismology, and geological setting; and the quality of the dam design, construction, operations and governance. As mentioned previously, not all this information is readily available in the current failure databases. In fact, most databases only include very basic information such as the dam age and height. The ICOLD Bulletin 99 database is the most comprehensive and includes reviewed and categorized failure causes.

[1] The terms "probability of failure" and "failure rate" have different meanings in some context but are used interchangeably herein.

Information available in the databases that could be relevant in estimating adjustments to baseline failure rates includes:

- Dam height.
- Dam age and year of construction / failure.
- Dam type (general, does not include raise type or core material, but can differentiate between rockfill, earth fill, and concrete dams).
- Incident context (pre-failure condition, i.e. flood, seismic, normal, etc.).
- Region (country and continent).
- Technical failure cause.
- Organizational failure cause.
- Failure type (overtopping, structural, internal erosion).

SRK also added information on the 1-in-475 year PGA (Peak Ground Acceleration) to the database to provide an indication of seismic setting. These PGA values were sourced from the Global Earthquake Model (GEM) Seismic Hazard Map (version 2023.1 – June 2023) based on dam latitude and longitude.

Additional preprocessing steps included:

- Binning dam height bins into the following bins: 0-20 m, 20-50 m, >50 m
- Binning dam age bins into the following bins: construction, <1 year, 1-5 years, >5 years
- Creating binary “yes”, “no” columns for each failure cause and type
- Binning seismic setting into the following PGA bins: 0-0.15 $m/s^2$, 0.15-0.3 $m/s^2$, >0.3 $m/s^2$

Adjustments to baseline rates were estimated using the simple form of Bayes’ theorem below. The results were also expressed as adjustment factors (AF) to indicate how strongly the presence of a modifying condition increases or decreases the probability of failure.

$$P(Failure\ |Indicator) = \frac{P(Indicator\ |Failure)\ P(Failure)}{P(Indicator)}$$

$$AF = \frac{P(Failure\ |Indicator)}{P(Failure)} = \frac{P(Indicator\ |Failure)}{P(Indicator)}$$

The results of these analyses are provided in Section 3.2.2 below.

## 2.3 Assessing Potential Consequences

The potential downstream consequences of a FRHEP embankment failure were estimated using a dam breach and outflow model set up in HEC-RAS 2D, a well-known hydrologic modeling software. The model was run many times to assess the effects of different assumptions about the timing and size of the breach. The results were then used to estimate the range of potential downstream impacts, with a focus on risks to the downstream communities.

### 2.3.1 Dam Breach Modelling

Preliminary dam breach modeling (SRK AU 2018a) was carried out for dam classification purposes and to inform selection of appropriate design criteria. This analysis modelled a 'worst-case' scenario assuming 100% of stored tailings, waste rock and water were released instantaneously. While this assumption is appropriate for the selection of conservative dam classification and design criteria, given that the proposed feasibility level design places tailings behind waste rock approximately 1 km from the main embankment, it is overly conservative for risk assessment and emergency planning purposes.

The current study considered a range of breach sizes and timing in order to better understand the range of possible downstream consequences. The modelling basis and key differences between the two sets of model runs are summarized in Table 4.

**Table 4: Differences Between Previous Dam Breach Modelling and Current Study**

| | Previous Dam Breach Model | This Study |
|---|---|---|
| General approach | Modelled 'worst-case' scenario for dam classification purposes | Modelled three breach sizes to bracket the plausible ranges of failures. The modelled scenarios are summarized in Table 5 |
| Software | HEC-RAS 2D | |
| Modelling domain | Both studies used the same modelling domain extending approximately as shown in the result figures included in Section 3.3 and Appendix A. | |
| Base topography | Provided LiDAR near dam and coarse SRTM data (90 m resolution) downstream | Same LiDAR near dam, and open-source ALOS PALSAR satellite topography with 12.5m resolution downstream (sourced in July 2007, representing the seasonal low-flow condition) |
| Release volumes | Assumed 100% of tailings, waste, and water released instantaneously. Release volumes of 9.6 $Bm^3$ and 10.8 $Bm^3$ (with and without PMF volume) | Assumed only water could be released, tested incremental volumes ranging from 0.3 $Bm^3$ to 6.4 $Bm^3$ (see details below) |
| Breach hydrographs | US NWS Breach model | Combination of empirical equations, weir equations, and judgement (additional details provided below) |
| Downstream river flows | Not considered | Added initial base flows in the Sepik River to represent seasonal high flood scenarios as interpreted from PanAust river station monitoring database. |
| Surface roughness (Manning's n-values) | 0.2 in stream channels, 0.4 in lowland areas, 0.6 in highland areas | |

#### Modelled Scenarios and Release Volumes

Three release volume scenarios were selected to bracket the range of potential impacts:

1. **Minor breach**: 2 m of water released, representing a loss of freeboard while maintaining the integrity of the dam.

2. **Intermediate or partial breach**: 10 m of water released, representing a significant failure and loss of dam contents.
3. **Maximum breach:** ~67 m of water released, representing losing all water stored above the waste elevation.

The "maximum breach" scenario assumes the embankment will be eroded down to an elevation approaching that of the waste rock platforms. Although further down-cutting is hypothetically possible, there would be little high-energy water left in the reservoir below that level. The "maximum breach" scenario is also consistent with the maximum breach size in the dam failure case history database (the Teton dam failure).

Differences in breach timing were assessed by including two variants of each scenario. The first assumed the breach occurred during seasonal-low initial water levels in the Sepik River, and the second assumed seasonal-high initial water levels.

**Table 5: Summary of Modelled Breach Scenarios**

| Scenario Name | Height of Stored Water Released (m) | Release Volume ($m^3$) | Breach Timing Variants | |
|---|---|---|---|---|
| Minor Breach | 2 | 0.26 $Bm^3$ | During seasonal low water levels in Sepik River | During seasonal high water levels in Sepik River |
| Intermediate Breach | 10 | 1.24 $Bm^3$ | | |
| Maximum Breach(1) | ~67 (max) | 6.37 $Bm^3$ | | |

Notes:

1 The maximum waste/tailings elevation is 159.4, or about 75 m below the crest of the FRHEP embankment, and about 70 m below the maximum water depth

## Breach Hydrographs

There are a number of empirical equations and simplified models available to estimate maximum breach outflow rates ("peak flow" in dam breach jargon) and breach development times ("time to failure"). Unfortunately, the empirical equations were all developed from case histories involving much smaller dams with very different storage configurations than the FRHEP. Therefore a range of possible peak flows was developed using a combination of empirical and ogee crest weir equations, and a range of possible times to failure was selected from literature. It was found that the modelled outflows were far less sensitive to the tested ranges of peak flow and time to failure compared to the assumptions about breach depth and release volume. The assumptions listed in Table 6 and resulting breach hydrographs were adopted, and resulted in the breach outflow hydrographs provided in Figure 6. The hydrographs tails were extended to 21 days from the time of breach to capture both the arrival and dissipation of the breach flood at downstream villages.

**Table 6: Summary of Modelled Breach Hydrographs**

| Scenario Name | Peak Flow ($m^3/s$) | Time to Failure (hours) |
|---|---|---|
| Minor Breach | 5000 | 2 |
| Intermediate Breach | 20,000 | 2 |
| Maximum Breach[(1)] | 65,120[3] | 4[3] |

Notes:

[1] These breach parameters are equivalent to the Teton Dam failure, which is the largest failure of an earthfill dam available in the historical records.

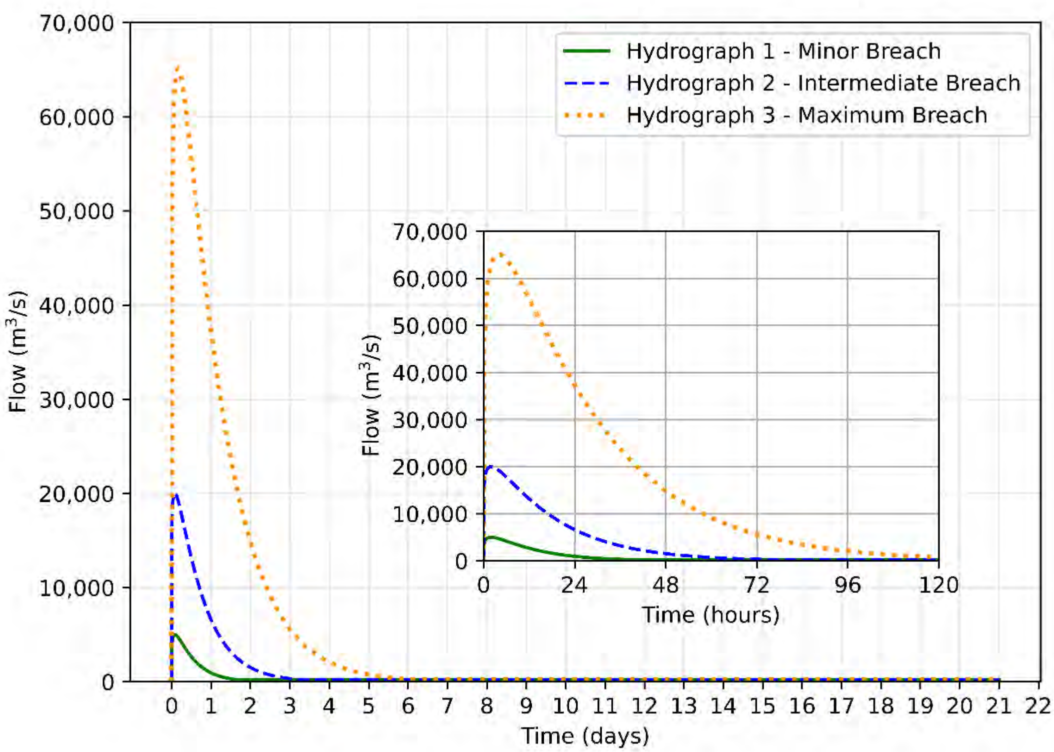



**Figure 6: Breach hydrographs for modelled scenarios**

### 2.3.2 Flood Hazard Assessment

#### General Observations

PanAust provided the following anecdotal observations about communities and natural floods on the Sepik River:

- Communities are typically located on slightly elevated banks adjacent to the river.
- Communities move with the river (which has been observed to have moved 40 m to 60 m at Iniok over the last 10 years).
- Banks are typically 3 to 4 m above normal river levels, and community houses are typically on 1 to 2 m high stilts above the banks.
- Communities are subject to regular seasonal flooding which typically does not impact the homes.
- The Sepik River width close to Iniok is approximately 200 m wide, but kilometers wide during natural flood events.

Figure 7 was provided by PanAust and shows a typical community located along the Sepik River.

**Figure 7: Community located along the Sepik River. Photo provided by PanAust.**

### Relevant Information from River Monitoring Data

SRK reviewed river monitoring data supplied by PanAust, including measurements from various stations along the Frieda, Sepik, and Wario Rivers. The key finding was that the Sepik River experiences seasonal water level fluctuations of approximately 6 m, while all other rivers in the area experience approximately 4 m of seasonal variation.

### Hazard Assessment

To assess the hazards to downstream communities arising from a potential failure of the FRHEP embankment, results of the dam breach modeling were compared to natural flooding levels. The hazard assessment steps were as follows, for each community within the dam breach inundation zone:

1. Compare the predicted dam breach outflow water levels to the seasonal high water level.
   a. If the maximum breach outflow water level is below the seasonal high water level, the breach outflow was considered non-hazardous.
2. Compare the maximum depth of the breach outflow to hazard criteria illustrated in Figure 8
   a. If the maximum depth is greater than 3.5 m (measured as distance above the seasonal high-water level), the breach outflow was considered hazardous to humans.
   b. If the maximum depth above the seasonal high water level is less than 3.5 m, calculate the depth x velocity (DV) value
      i. If DV is in the 'Low Danger Zone' of Figure 8, the breach outflow flood was considered to be low danger to the community.
      ii. If DV is in either the 'Judgment Zone' or the 'High Danger Zone' of Figure 8, the breach outflow flood was conservatively assumed to high danger to the community.

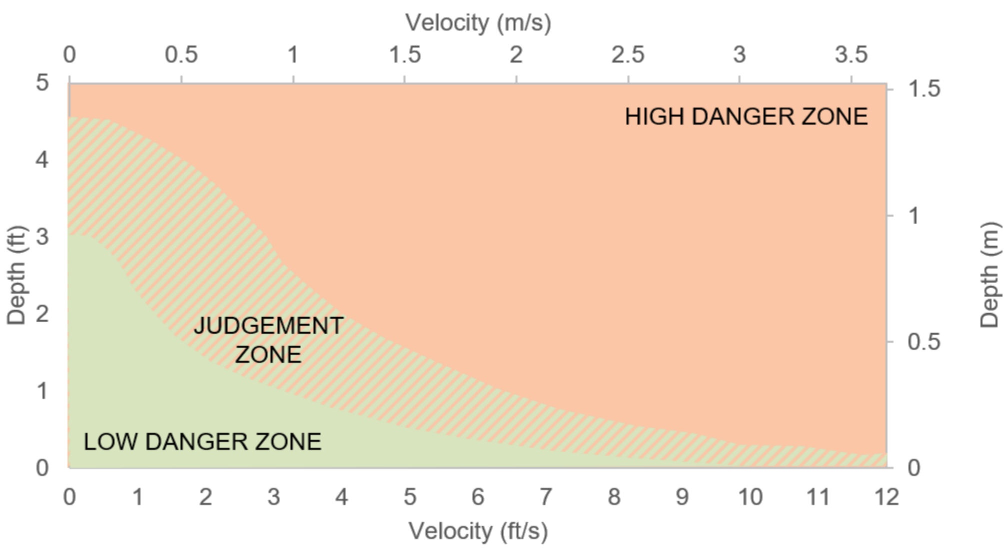



**Figure 8: Depth – velocity hazard relationship for adults (from USBR 1988).**

# 3 Results

## 3.1 Experience at Other Dams

### 3.1.1 Summary of Global Failure Statistics

Table 7 summarizes the number of dams and failures observed by dam type.

**Table 7: Number of Dams and Failures by Dam Type**

| | Total Number of Dams | Number of Dam Years | Number of Failures |
|---|---|---|---|
| All dams > 15 m | 51,724 | 1,951,433 | 265 |
| Asphalt core dams | 123 | 2,824 | 0 |
| Rockfill dams | 3,372 | 72,910 | 27 |
| Earthfill dams | 33,962 | 1,522,097 | 182 |
| Other and unknown | 14,267 | 353,602 | 56 |

#### Seismic Setting

Table 8 summarizes the number of dams and failures by seismic setting.

**Table 8: Number of Dams and Failures by Seismic Setting**

| 1-in-475 year PGA | Total Number of Dams | Number of Failures |
|---|---|---|
| Unknown | 21678 | 245 |
| 0 to 0.15 | 30270 | 50 |
| 0.15 to 0.3 | 6167 | 5 |
| > 0.3 | 3931 | 6 |

#### Causes of Dam Failures

Table 9 shows dam failure statistics by specific cause and failure mode. Due to the limited information about specific causes in most of the available dam failure databases, the groupings “All Organizational Issues” and “All Technical Issues” were adopted for further analysis.

**Table 9: Number of Failures by Cause and Failure Mode**

| | Structural Failures | Internal Erosion Failures | Overtopping Failures | All Failures |
|---|---|---|---|---|
| **All Organizational Issues** | **65** | **77** | **87** | **218** |
| ▪ Design | 49 | 72 | 56 | 166 |
| ▪ Construction | 13 | 17 | 11 | 38 |
| ▪ Operation & Maintenance | 11 | 7 | 28 | 45 |
| ▪ None identified | 24 | 30 | 46 | 96 |
| **All Technical Issues** | **80** | **97** | **100** | **262** |
| ▪ Geotechnical | 55 | 89 | 21 | 151 |
| ▪ Structural | 18 | 7 | 3 | 27 |
| ▪ Material Aging | 3 | - | - | 3 |
| ▪ Inadequate Capacity | 5 | 1 | 75 | 81 |
| ▪ Hydromechanical Malfunction | 4 | 2 | 12 | 17 |
| ▪ Unknown | 8 | 11 | 34 | 51 |
| **All Failures** | **89** | **107** | **133** | **320** |

Note: numbers in the above table do not sum to the totals because some failures included more than one cause or failure type.

### 3.1.2 Earthquake Performance of Other Dams

Table 10 summarizes current hydropower dams located in Papua New Guinea and the PGA associated with the highest seismic event they have experienced, where available.

**Table 10: Summary of Hydropower Dams in Papau New Guinea**

| Dam | Commissioned | Type | Height (m) | Highest PGA ($m^2/s$) Experienced |
|---|---|---|---|---|
| Yonki | 1992 | Earth-fill embankment | 60 | 1.2 |
| Ok Menga | 1988 | Run of river | Unknown, low | N/A |
| Rouna 2 | 1967 | Gravity | 21 | N/A |
| Pauanda | 1983 | Run of river | Low height | 0.65 |
| Ramu | 1975 | Gravity | 30 | N/A |
| Sirinumu | 1971 | Rockfill | 32 | N/A |
| Lake Hargy | 1989 | Concrete gravity weir | Unknown, low | N/A |
| Warangoi | N/A | Run of river | Unknown, low | N/A |

Table 11 summarizes global facilities that have experienced significant seismic events and survived with little to minor damage. The review only identified one example of an asphalt core dam that has experienced a significant earthquake.

**Table 11: Summary of Other Global Dams that have Survived Significant Earthquakes**

| Dam | Location | Type | Height (m) | Highest PGA ($m^2/s$) Experienced |
|---|---|---|---|---|
| Coyote | USA | Earthfill | 43 | 6.2 |
| Aya | Philippines | Earthcore rockfill | 102 | 5.7 |
| Makio | Japan | Earthcore rockfill | 77 | 5.6 |
| Colbún Dam | Chile | Zoned earth with sandy clay core | 116 | 3.5 - 6.6 |
| Aratozawa Dam | Japan | Rockfill | 74.4 | 3.5 - 6.6 |
| Lexington | USA | Earthfill | 63 | 4.4 |
| Almaden | USA | Earthfill | 32 | 4.3 |
| Guadalupe | USA | Earthfill | 43 | 4.1 |
| Rinconda | USA | Earthfill | 12 | 4.0 |
| Leroy Anderson | USA | Earthcore rockfill | 72 | 4.0 |
| Midorikawa Hojo Dam | Japan | Rockfill | 35 | 4.0 |
| Uvas | USA | Earthfill | 32 | 3.9 |
| Calero | USA | Earthfill | 30 | 3.7 |
| Diayo | Philippines | Earthcore rockfill | 60 | 3.7 |
| Canili | Philippines | Earthcore rockfill | 70 | 3.7 |
| Vasona | USA | Earthfill | 10 | 3.6 |
| Shintsuruko Dam | Japan | Rockfill | 96 | 2.0 – 5.0 |
| Shichikasyuku Dam | Japan | Rockfill | 90 | 2.0 – 5.0 |
| Surikamigawa Dam | Japan | Rockfill | 105 | 2.0 – 5.0 |
| Stevens Creek | USA | Earthfill | 37 | 2.9 |
| La Paloma Dam | Chile | Zoned dam with clay core | 82 | 1.8 - 3.5 |
| Los Cristales Dam | Chile | Rockfill with asphalt core | 31 | 0.9 - 2.9 |
| Sagurigawa Dam | Japan | Rockfill | 119.5 | 0.5 - 1 |
| Sagae Dam | Japan | Rockfill | 112 | 0.5 – 1.0 |

Table 12 shows some examples of other dams that have been constructed in a seismic setting similar to the FRHEP, including three asphalt core dams. It also includes the FRHEP in the first row for comparative purposes.

**Table 12: Examples of Asphalt Core Dams Constructed in Similar Seismic Settings**

| Dam | Location | | Completion Date | Height (m) | GSHAP Seismic Hazard Map PGA ($m^2/s$) (1 in 475 years) |
|---|---|---|---|---|---|
| FRHEP | Papua New Guinea | Asphalt core | TBD | 191 | 1.6 – 2.4 |
| Moglice | Albania | Asphalt core | 2020 | 167 | 1.6 – 2.4 |
| Yele | China | Asphalt core | 2005 | 124 | 3.2 – 4.0 |
| Quxue | China | Asphalt core | 2016 | 165 | 2.4 – 3.2 |
| Zipingpu | China | Concrete face rockfill | 2006 | 156 | 5.37 |
| Sogamoso | Colombia | Concrete face rockfill | 2015 | 190 | 4.10 |
| Mazar | Ecuador | Concrete face rockfill | 2011 | 166 | 4.19 |
| La Parota | Mexico | Concrete face rockfill | 2020 | 189 | 7.28 |
| Mohmand | Pakistan | Concrete face rockfill | TBD | 213 | 3.25 |
| Antamina | Peru | Concrete face rockfill | 2001 | 240 | 3.19 |
| Chaglla | Peru | Concrete face rockfill | 2016 | 202 | 4.23 |
| Yacambu | Venezuela | Concrete face rockfill | 2011 | 158 | 3.38 |
| Candelaria | Chile | Rockfill | 1994 | 170 | 8.70 |
| Pakal Dul (Drangdhuran) | India | Rockfill | 2023 | 167 | 5.11 |
| Takase | Japan | Rockfill | 1978 | 176 | 3.57 |
| Tokuyama | Japan | Rockfill | 2008 | 161 | 3.28 |
| Seven Oaks | USA | Rockfill | 1999 | 167.6 | 6.68 |
| Tehri | India | Rockfill / earthfill | 2006 | 260 | 3.31 |
| Moglica | Albania | Rockfill with asphalt core | 2019 | 166.5 | 3.71 |
| Fierza | Albania | Rockfill with clay core | 1978 | 166.5 | 3.21 |
| Gotvand Oliya | Iran | Rockfill with clay core | 2012 | 182 | 3.37 |
| Maroon | Iran | Rockfill with clay core | 1999 | 165 | 4.23 |
| Roodbar e Lorestan | Iran | Rockfill with clay core | 2017 | 155 | 2.97 |
| Haraz | Iran | Rockfill with clay core | 2022 | 150 | 3.27 |
| Kigi | Türkiye | Rockfill with clay core | 2016 | 175 | 4.09 |
| Dalaman Akköprü | Türkiye | Rockfill with clay core | 2011 | 162 | 3.28 |

## 3.2 Estimated Probabilities of Dam Failure

### 3.2.1 Baseline Dam Failure Rates

Key inputs from the ICOLD database and the resulting estimates of baseline failure rates are summarized in Table 13, both for all dams greater than 15 m high and for asphalt core dams. The baseline failure rates are expressed in terms of an average annualized failure rate, which is equivalent to the probability that an "average" dam with the specified characteristics will fail in any given year.

**Table 13: Estimated Baseline Probabilities of Failure**

| | All Dams (>15 m) | Asphalt Core Dams |
|---|---|---|
| **Total Number of Dams** | 51,724 | 123 |
| **Number of Dam Years** | 1,951,433 | 2824 |
| **Number of Failures** | 265 | 0 |
| **Average Annualized Failure Rate** | 1.4E-4 | 9.8E-5 |
| **95% Prediction Interval** | 1.2E-4 to 1.5E-4 | 2.5E-6 to 3.6E-4 |

It is interesting to note the interplay between the very large database of "all dams" and the much smaller database of asphalt core dams. First, even though the asphalt core dam database does not include any failures, failure rates can be estimated using Bayesian statistics. Second, as shown in Figure 9, the fact that no failures have been observed shifts the average failure rate of asphalt core facilities below the average failure rate for all dams. This shift is relatively minor because of the much shorter experience to date with asphalt core dams (~2800 dam-years compared to ~2,000,000 dam years). Third, the 95% prediction interval for asphalt core dams is much wider than that for "all dams", but this doesn't mean asphalt core dams are more variable. On the contrary, the wider prediction interval simply reflects the limited global experience with asphalt core dams. If the current trend of no observed failures continues, the predicted range for asphalt core dams will be significantly lower and narrower than shown in Figure 9.

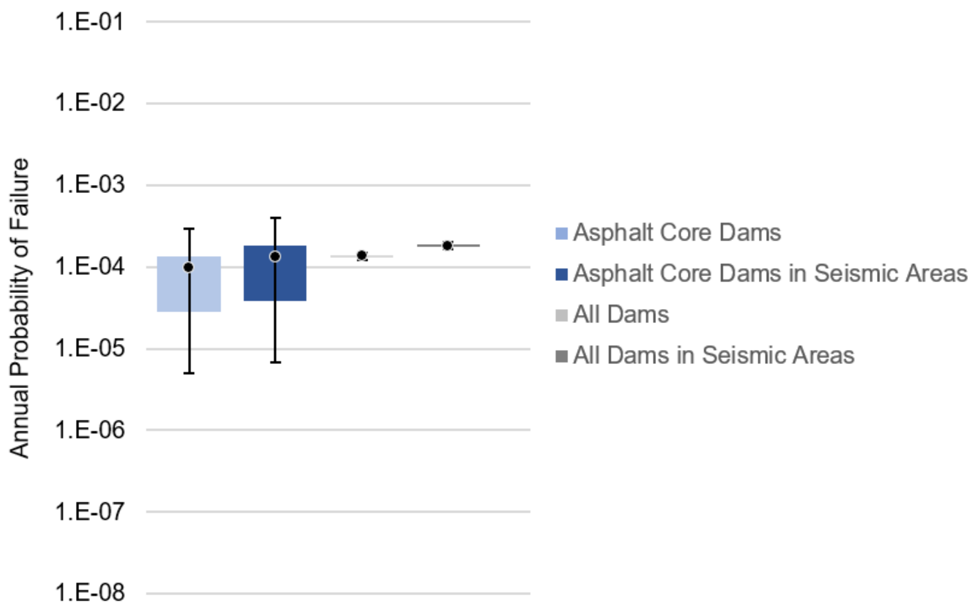



**Figure 9:** **Comparison of probabilities of failure for asphalt core dams and "all dams" with black error bars showing the 95% prediction interval and boxes indicating +/- one quartile from the mean**

### 3.2.2 Adjustments to Baseline

#### Influence of Seismic Setting

The revised failure rates and adjustment factors for seismic setting are shown in Table 14 and indicate that a low seismic setting results in a slight reduction in baseline failure rates, while a moderate-to-high seismic setting increases baseline failure rates by about 35%.

**Table 14:** **Estimated Adjustments for Seismic Setting**

| Seismic Setting | 1-in-475 year PGA | Revised Failure Rate (all dams >15 m) | Adjustment Factor |
|---|---|---|---|
| Low | < 0.3 | 1.30E-4 | 0.96 |
| Moderate to high | > 0.3 | 1.83E-4 | 1.35 |

#### Influence of Technical and Organizational Quality

The contribution of technical and organizational issues to historical dam failures is clear from the statistics summarized in Table 9, but calculating adjustment factors requires an additional assumption on how common technical or operational issues are in dams that have not failed. If these issues are very common in all dams, then the presence of an issue at a particular dam is not a strong indicator of that dam having a higher failure probability.

Most global databases do not present information on technical and organizational issues at dams that have not failed. The best information available comes from tailings disclosure data (Franks, 2022), which includes a question on whether each facility has a history of stability concerns. Of the 1743 tailings dams in the most recent data, the answer was “Yes” for 182 dams and “No” for 1527 dams (others responses were “unknown” or “did not answer”). If one assumes that a history of stability concerns is an indication that the facility also has technical or organizational issues, the probability of a facility having a technical or organizational issue must be at least 10% (i.e. 182/1743). Further considering that many dam failure investigations reveal issues that were unknown or undisclosed prior to the failure, this analysis adopted a higher estimate, specifically that 10% of dams have technical issues and 10% have organizational issues. Applying this assumption leads to the estimated adjustment factor shown in Table 15.

**Table 15: Estimated Adjustments for Presence of Technical or Organizational Issues**

| | Revised Failure Rate (all dams >15m) | Adjustment Factor |
|---|---|---|
| **No Technical or Organizational Issues** | | 0.009 |
| No Technical Issues | | 0.02 |
| No Organizational Issues | | 0.35 |
| **Base Case** | | 1.0 |
| Organizational Issues | | 6.7 |
| Technical Issues | | 9.4 |
| **Both Organizational and Technical Issues** | | 50.3 |

Note: due to overlap in cases that have both technical and organizational issues, the adjustment factors are not multiplicative

These adjustment factors suggest that the absence of any technical or organizational issues (*i.e.* a high standard of design, construction, operations and governance) would imply that probabilities by approximately two orders of magnitude, whereas the presence of technical or organizational issues (*i.e.* a low standard of design, construction, operations or governance) could increase baseline probabilities by approximately one-and-a-half orders of magnitude.

## Other Tested Conditions

Table 16 summarizes other dam conditions that were tested and whether they had a statistically significant relationship to the probability of dam failure.

**Table 16: Other Factors Tested for Significance**

| Factor | Relationship to dam failure probability? | Comments |
|---|---|---|
| Height | No statistical significance | There is a higher number of failures of smaller dams, but this is proportional to the higher number of smaller dams in the global population. |
| Dam age | **Significant** | See below |

| Factor | Relationship to dam failure probability? | Comments |
|---|---|---|
| Dam type | Some statistical significance | There was a statistical significance for specific types of failure modes, with rockfill dams appearing to be less susceptible to structural failures compared to earth fill dams. |
| Incident context | Some statistical significance | There was a statistical significance for specific failure modes, for example, a preceding flood condition was associated with overtopping failure modes. This was not considered to provide any new information |

Estimated adjustments for dam age, shown in Table 17 below, indicate that failures are much more frequent during construction and within the first five years of dam operations, and less likely after five years of operations. The age-adjustment factors were not applied in this assessment, but they do show the importance of strong oversight and governance during construction and initial operations.

**Table 17: Estimated Adjustments for Dam Age**

| Dam Age | Revised Failure Rate (all dams >15 m | Adjustment factor |
|---|---|---|
| During construction | | 6 |
| < 1 year | | 14 |
| 1 to 5 years | | 13 |
| > 5 years | | 0.7 |

### 3.2.3 FRHEP Failure Probability Ranges

Figure 10 and Table 18 collates the estimated probabilities of failure presented in the preceding sub-sections. The first two rows show the "baseline" results, *i.e.* estimated failure rates for all dams higher than 15 m, and failure rates for all asphalt core dams. The next two rows show the increases in estimated failure rates for all dams and asphalt core dams located in seismic areas. The remaining rows show how the estimated failure rates change depending on whether a dam has technical and organizational issues. The 'high quality' estimates assume there are no technical or organizational issues, *i.e*. that the quality of design, construction, operations, and governance is consistently high. The 'low quality' estimates assume there are both technical and organizational issues, *i.e.* that the quality of design, construction, operations, and/or governance is poor.

**Table 18: Summary of Estimated Probabilities of Failure**

| Scenario | | Mean | 95% Prediction Interval | |
|---|---|---|---|---|
| | | | Lower | Upper |
| Baseline | All dams | 1.4E-04 | 1.2E-04 | 1.5E-04 |
| | Asphalt core dams | 9.8E-05 | 2.5E-06 | 3.6E-04 |
| Seismic area | All dams in seismic area | 1.8E-04 | 1.6E-04 | 2.1E-04 |
| | Asphalt core in seismic area | 1.3E-04 | 3.4E-06 | 4.9E-04 |

| Scenario | | Mean | 95% Prediction Interval | |
|---|---|---|---|---|
| | | | Lower | Upper |
| High Quality *(no technical or organizational issues)* | All dams | 1.2E-06 | 1.1E-06 | 1.4E-06 |
| | All dams – seismic area | 1.7E-06 | 1.5E-06 | 1.9E-06 |
| | Asphalt core | 8.8E-07 | 2.2E-08 | 3.3E-06 |
| | Asphalt core – seismic area | 1.2E-06 | 3.0E-08 | 4.4E-06 |
| Low Quality *(both technical and organizational issues)* | All dams | 6.8E-03 | 6.0E-03 | 7.7E-03 |
| | All dams – seismic area | 9.2E-03 | 8.1E-03 | 1.0E-02 |
| | Asphalt core | 4.9E-03 | 1.3E-04 | 1.8E-02 |
| | Asphalt core – seismic area | 6.7E-03 | 1.7E-04 | 2.5E-02 |

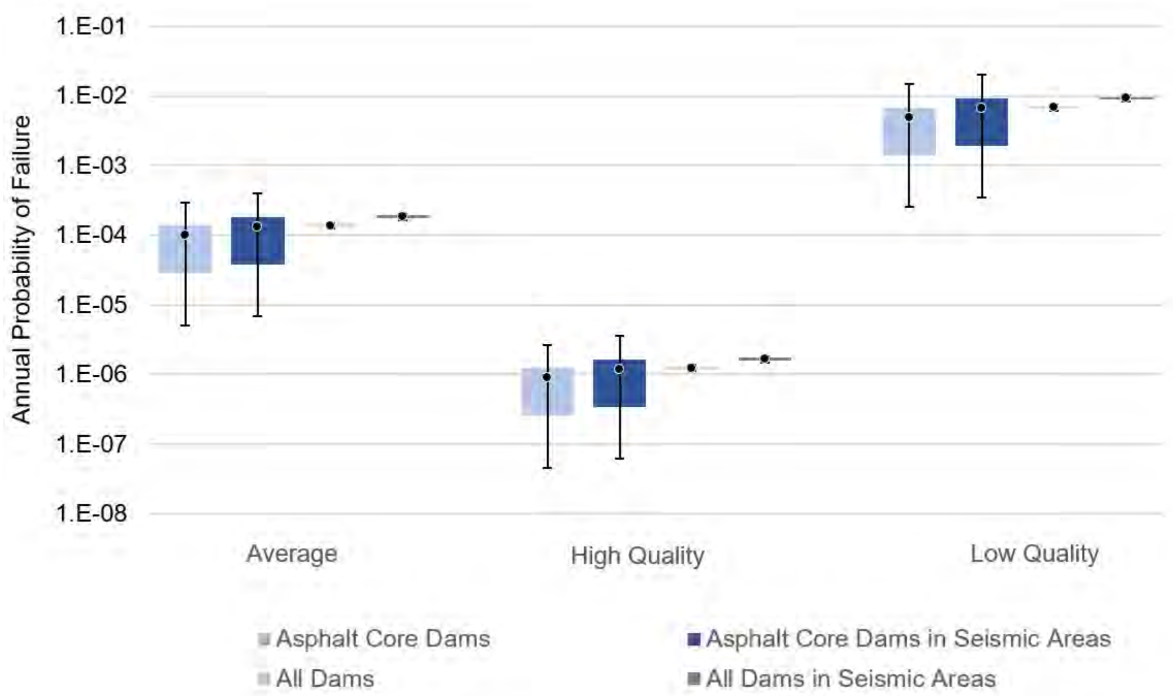



**Figure 10: Graphical presentation of estimated probabilities of failure**

## 3.3 Estimated Consequences

### 3.3.1 Dam Breach Modeling

Results of dam breach modelling and outflow flood hazard assessments are summarized by scenario and community in Table 19. Detailed results are provided in Appendix B which includes figures showing the flood extents and time series plots of depth and velocity for each scenario and community. PanAust informed SRK that the closest community to the FRHEP embankment, Paupe, will be relocated prior to constructing the facility and therefore it was not considered in the consequence assessment.

Inundation maps for minor breach and maximum breach are shown in Figure 11 and Figure 12. Inundation and breach flood velocity maps for all scenarios are provided in Appendix A.

**Table 19: Village-by-Village Hazard Assessment – Max depth at adjacent waterbody (m), [Max depth above seasonal high] (m), velocity (m/s)**

| Village Name | Population[1] | Adjacent Waterbody | (1) Minor Breach – 2 m 0.26 Bm3 Seasonal Low Water Level | (2) Intermediate Breach – 10m 1.24Bm3 Seasonal Low Water Level | (2)[2] Intermediate Breach – 10m 1.24Bm3 Seasonal High Water Level | (4) Maximum Dam Breach 6.37Bm3 Seasonal Low Water Level |
|---|---|---|---|---|---|---|
| Abagusai | Unknown | May River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Arapi | 48 | - | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Aumi | 196 | May River | Not Inundated | Not Inundated | 2.93, [-], 0.01 | 3.21, [-], 0.26 |
| Biaga | Unknown | Sepik River | Not Inundated | 9.43, [3.433], 0.11 | 14.15, [8.15], 0.02 | 16.87, [10.87], 0.23 |
| Hauna | 5,500 | Wario River | Not Inundated | 3.01, [-], 0.05 | 4.84, [0.84], 0.06 | 9.43, [5.43], 0.47 |
| Ibu 1 | 104 | May River | Not Inundated | Not Inundated | 4.71, [0.71], 0.02 | 6.97, [2.97], 0.17 |
| Imombi | Unknown | Sepik River | Not Inundated | 3.57, [-], 0.47 | 9.31, [3.31], 0.8 | 11.1, [5.1], 0.85 |
| Iniok | 3,500 | Sepik River | 2.49, [-], 0.45 | 7.34, [1.34], 0.98 | 11.17, [5.17], 0.78 | 15.88, [9.88], 1.81 |
| Iyie | Unknown | Seniap River | Not Inundated | Not Inundated | Not Inundated | 1.85, [-], 0.02 |
| Kubkain | 5,000 | Sepik River | 2.6, [-], 0.45 | 5.46, [-], 0.93 | 10.33, [4.33], 0.87 | 13.41, [7.41], 1.99 |
| Maposi | 201 | Wogamush River | Not Inundated | Not Inundated | Not Inundated | 5.56, [1.56], 0.12 |
| Mapupoli | Unknown | Wogamush River | Not Inundated | Not Inundated | Not Inundated | 11.29, [7.29], 0.66 |
| Moropote | 298 | - | Not Inundated | Not Inundated | Not Inundated | 5.32, [-], 0.03 |
| Mowi | 5,500 | Sepik River | 0.45, [-], 0.33 | 4.7, [-], 0.93 | 9.68, [3.68], 0.66 | 12.35, [6.35], 1.49 |
| Nein | 114 | Wario River | 7.58, [3.58], 0.1 | 12.55, [8.55], 0.23 | 12.55, [8.55], 0.22 | 18.16, [14.16], 0.47 |
| Nekiei | Unknown | Wario River | 1.26, [-], 0.07 | 8.14, [4.14], 0.17 | 8.14, [4.14], 0.17 | 15.93, [11.93], 0.36 |
| Oum 1 | 644 | Sepik River | 0.45, [-], 0.16 | 3.96, [-], 0.33 | 8.39, [2.39], 0.06 | 12.7, [6.7], 0.31 |
| Oum 2 | 517 | Sepik River | 2.45, [-], 0.26 | 5.95, [-], 0.53 | 10.38, [4.38], 0.08 | 14.68, [8.68], 0.85 |
| Oum 3 | 350 | Lake Warangai | Not Inundated | 1.24, [-], 0.03 | 6.12, [2.12], 0.02 | 14.51, [10.51], 0.07 |
| Painum | Unknown | May River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Paru | Unknown | Unnamed Lake | Not Inundated | Not Inundated | Not Inundated | 9.4, [5.4], 0.34 |
| Pei | 700 | Wario River | 6.71, [2.71], 0.21 | 11.99, [7.99], 0.3 | 11.99, [7.99], 0.3 | 17.31, [13.31], 0.51 |
| Pekwei | Unknown | May River | Not Inundated | Not Inundated | 7, [3], 0.01 | 7.28, [3.28], 0.14 |
| Sanapien | 2,500 | Sepik River | Not Inundated | 2.4, [-], 0.06 | 7.4, [1.4], 0.04 | 11.47, [5.47], 0.14 |

| Village Name | Population[1] | Adjacent Waterbody | (1) Minor Breach – 2 m 0.26 Bm3 Seasonal Low Water Level | (2) Intermediate Breach – 10m 1.24Bm3 Seasonal Low Water Level | (2)[2] Intermediate Breach – 10m 1.24Bm3 Seasonal High Water Level | (4) Maximum Dam Breach 6.37Bm3 Seasonal Low Water Level |
|---|---|---|---|---|---|---|
| Sinen | Unknown | Wario River | 3.51, [-], 0.33 | 9.01, [5.01], 0.48 | 9.01, [5.01], 0.48 | 15.06, [11.06], 0.7 |
| Sio | 10,000 | Wogamush River | Not Inundated | Not Inundated | Not Inundated | 4.47, [0.47], 0.13 |
| Sowano | 252 | Wogamush River | Not Inundated | Not Inundated | Not Inundated | 8.39, [4.39], 0.17 |
| Sumwari | 1700 | Wario River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Swagup | 350 | Sepik River | 4.27, [-], 0.35 | 7.29, [1.29], 0.66 | 11.55, [5.55], 0.22 | 13.29, [7.29], 1.06 |
| Tauri | 3,000 | Sepik River | 2.75, [-], 0.24 | 6.79, [0.79], 0.45 | 10.77, [4.77], 0.12 | 14.95, [8.95], 0.79 |
| Usok | Unknown | Wario River | 2.5, [-], 0.21 | 8.52, [4.52], 0.95 | 8.52, [4.52], 0.49 | 15.76, [11.76], 0.37 |
| Wakiawei | Unknown | Wogamush River | Not Inundated | Not Inundated | Not Inundated | 12.93, [8.93], 0.11 |
| Walio | 450 | Wario River | 7.15, [3.15], 0.12 | 10.54, [6.54], 0.23 | 10.54, [6.54], 0.23 | 14.1, [10.1], 0.29 |
| Wanamoi | Unknown | May River | Not Inundated | Not Inundated | 1.64, [-], 0.07 | 1.9, [-], 0.22 |
| Wanium | Unknown | May River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Wasuware | 25 | Wario River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Yabatauwe | 114 | Wogamush River | Not Inundated | Not Inundated | Not Inundated | 9.39, [5.39], 0.25 |
| Yapram | Unknown | May River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |
| Yei | 2,100 | May River | Not Inundated | Not Inundated | Not Inundated | Not Inundated |

Sources: VillageByVillage_HazardAssessment_REVA.xlsx

Notes: Green highlighting indicates potentially non-hazardous flooding to adults. Red highlighting indicates high danger flooding to adults. Values reported correspond to the maximum depth of water output from the model at the waterbodies adjacent to the community (m), the relative depth above the seasonal high water levels is indicated in square parenthesis (m), and the maximum velocity is (m/s).

[1] The populations reported were sourced from a combination of open-source census information and PanAust.

[2] Despite all scenarios being analyzed at seasonal low and high initial water levels in the Sepik, these variants are only shown for the intermediate breach scenario herein. Discussion is provided in Section 4.2.

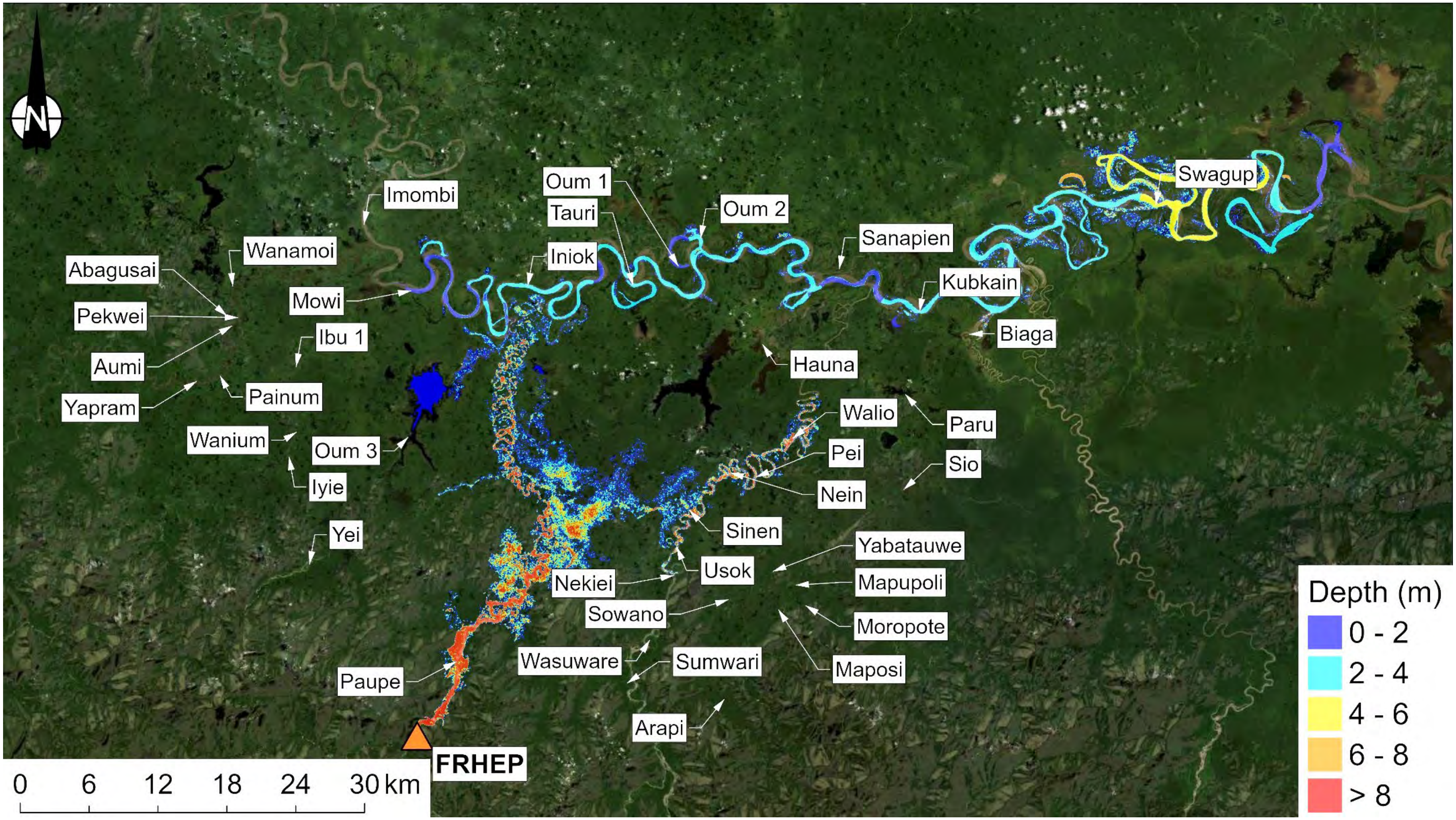



**Figure 11: Minor breach – 2 m breach height, 0.26 Bm$^3$ release volume.**

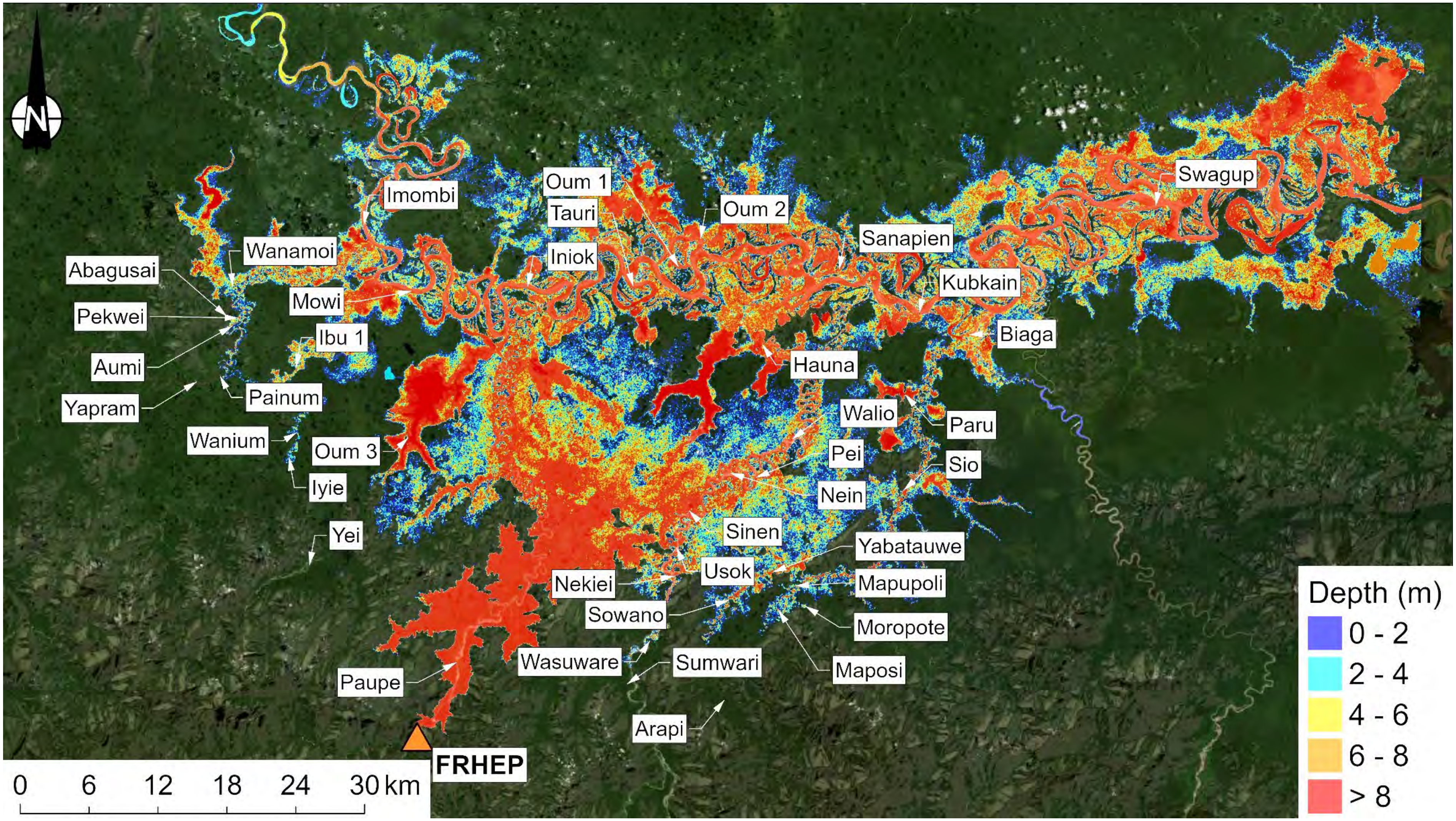



**Figure 12: Maximum breach - 66.7 m breach height, 6.37 Bm$^3$ release volume.**

### 3.3.2 Flood Arrival Times at Communities

Estimating times for the outflow flood to reach downstream communities require judgement because the currently available modelling tools are not able to capture processes that could increase flow velocities. In the model, the relatively flat topography downstream of the facility dissipates flow velocities and slows the progress of the flood wave. However, if the breach were to erode material and carve a new flowpath toward the Sepik River (as has been observed at other real world dam failures), it is likely that the flood would arrive sooner than predicted by the model. To account for this, SRK reviewed the flow path lengths to each community and assumed an upper bound velocity of 2 m/s to estimate the earliest possible arrival times.

The model results illustrating the outflow flood progression are provided in Figure 13 and Table 20 provides ranges of estimated flood wave arrival times for each community. The estimates suggest that outflow floods could reach the May River, Sepik River, and Wario River in as soon as 11, 6, and 5 hours, respectively.

### 3.3.3 Flood Durations at Communities

Estimates of flood duration, corresponding to the minimum amount of time until the modelled flood depths are less than 0.3 m, are also provided in Table 22. Total flood durations range from 19 to 20 days for the communities along the Sepik River, 10 to 18 days along the May River, and 19 to 20 days along the Wario River.

In addition to the uncertainties inherent in any dam breach the modeling, the flood duration estimates do not consider the time required to repair damage to infrastructure in the communities. Therefore, the total time that a population could be displaced could be longer.

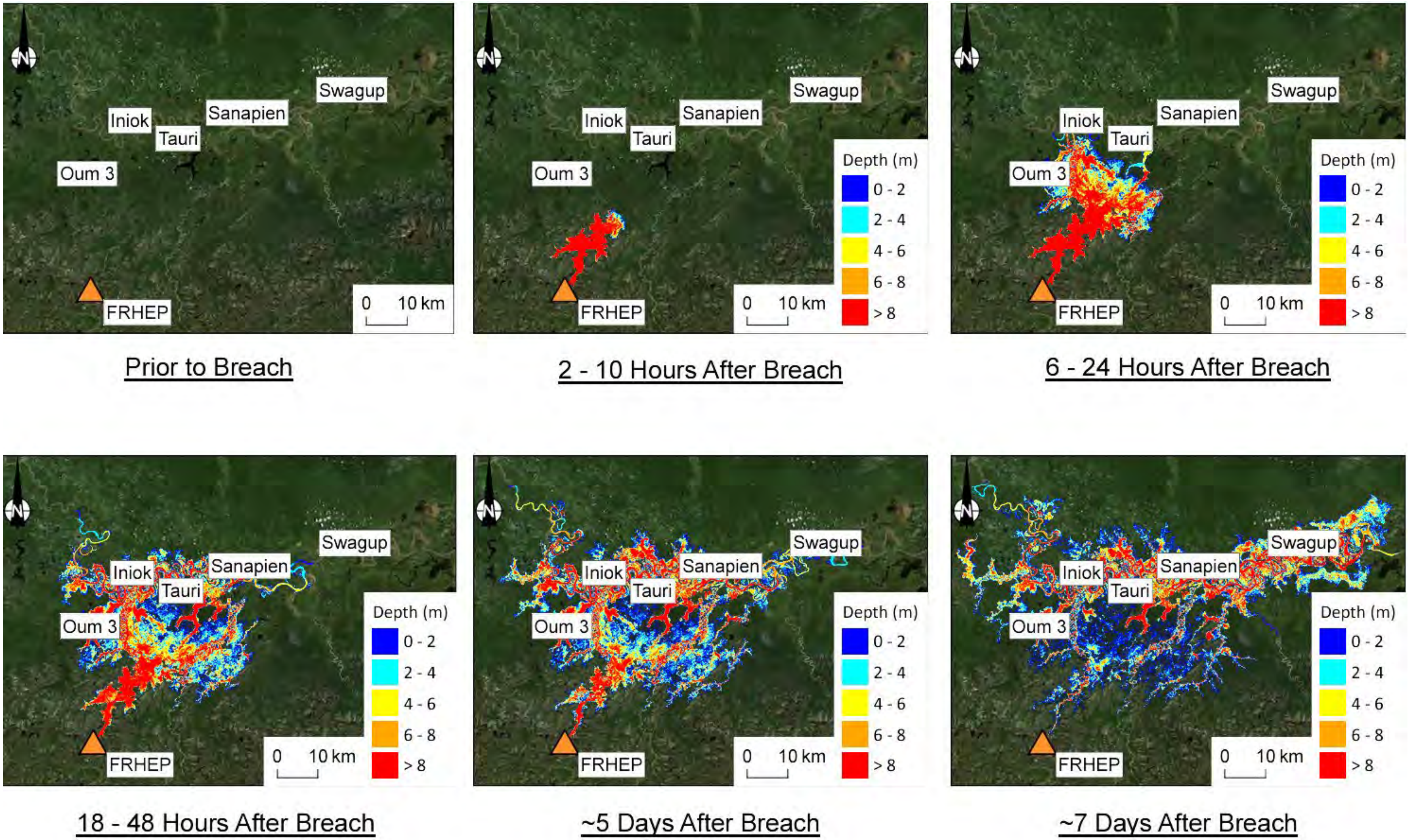



**Figure 13: Breach flood progression**

**Table 20: Range of Arrival and Flood Dissipation Times for Maximum Breach**

| Community | Population[1] | Arrival Time (hours) | Time to less than 0.3m Depth (days) |
|---|---|---|---|
| Aumi | | 12 - 92 | 17 |
| Biaga | | 15 - 57 | 19 |
| Hauna | > 5500 | 7 - 31 | 20 |
| Ibu 1 | | 11 - 60 | 18 |
| Imombi | | 10 - 31 | 20 |
| Iniok | > 3500 | 6 - 23 | 20 |
| Iyie | | 12 - 174 | 14 |
| Kubkain | > 5000 | 13 - 40 | 19 |
| Maposi | | 6 - 55 | 19 |
| Mapupoli | | 6 - 43 | 19 |
| Moropote | | 6 - 75 | 18 |
| Mowi | > 5500 | 9 - 27 | 20 |
| Nein | | 5 - 21 | 20 |
| Nekiei | | 4 - 17 | 20 |
| Oum 1 | | 10 - 32 | 20 |
| Oum 2 | | 10 - 30 | 20 |
| Oum 3 | > 350 | 5 - 26 | 20 |
| Paru | | 8 - 52 | 19 |
| Pei | | 5 - 24 | 21 |
| Pekwei | | 11 - 86 | 20 |
| Sanapien | > 2500 | 12 - 46 | 17 |
| Sinen | | 4 - 16 | 19 |
| Sio | | 7 - 60 | 20 |
| Sowano | | 6 - 30 | 19 |
| Swagup | | 18 - 56 | 20 |
| Tauri | > 3000 | 8 - 27 | 19 |
| Usok | | 4 - 16 | 20 |
| Wakiawei | | 6 - 37 | 20 |
| Walio | | 6 - 28 | 19 |
| Wanamoi | | 11 - 76 | 20 |
| Yabatauwe | | 5 - 32 | 10 |

Sources: ArrivalTimeEstimates_REVA.xlsx

Notes:

[1] Provided by PanAust via email communication

# 4 Discussion

## 4.1 Likelihood of a FRHEP Embankment Failure

### 4.1.1 Statistical Estimates

Sections 2.1. 2.2, 3.1 and 3.2 above presented several statistical analyses of data from other dams. The available databases include dams with characteristics or site conditions similar to the FRHEP embankment, and the statistical methods applied allow inferences to be drawn about potential failure of the FRHEP.

The key results are summarized in Table 21. All values in the table are annualized probabilities and can be compared to the global average annualized probability of failure of 1.4E-04, estimated for all dams in the global database with a height greater than 15 m. In brief:

- If the FRHEP embankment is consistent with average asphalt core dams in seismic areas, it is estimated to have the baseline probability of failure shown in the first row. This value is coincidentally very close to the global average; although probability of failure increases in seismic areas, that effect is balanced out by the reduction in probability of failure afforded by the FRHEP's asphalt core. But the 95% prediction intervals are wide because there is relatively limited experience with asphalt core dams.
- If the FRHEP embankment achieves a high quality of design, construction, operation and governance, the estimated probability of failure will be 100 times lower than the global average, as shown in the second row.
- On the other hand, if the FRHEP embankment experiences a low quality of design, construction, operation or governance, the estimated probability of failure will be roughly 50 times higher than the global average, as shown in the third row.

**Table 21: Estimated Probability of FRHEP Embankment Failure**

| | Mean | 95% Prediction Interval | |
|---|---|---|---|
| | | Lower | Upper |
| **Average asphalt core dam in seismic areas** | 1.3E-04 | 3.4E-06 | 4.9E-04 |
| **High quality asphalt core dam in seismic areas** | 1.2E-06 | 3.0E-08 | 4.4E-06 |
| **Low quality asphalt core dam in seismic areas** | 6.7E-03 | 1.7E-04 | 2.5E-02 |

### 4.1.2 Experience Elsewhere

The statistical approach was supplemented by surveys of dams with specific characteristics or site conditions similar to FRHEP. The survey results reported in Section 3.1.2 show there are five known hydropower dams currently operating in Papua New Guinea, with some in operation since the 1960s. Notable examples are the Yonki Dam, a 60 m high earth-fill embankment built in 1991 which survived a magnitude 7.1 earthquake in 2019, and the Pauanda Dam, of unknown type and height, which survived a magnitude 7.5 earthquake in 2018. The survey also found information about the seismic performance of dams in other locations. For example, Los Cristales Dam in Chile is an asphalt core dam that survived the magnitude 8.8 Maule earthquake in 2010, with a peak ground acceleration (PGA) estimated between 1.9 to 2.9 m/s$^2$.

The information in Section 3.1.2 about dams that have experienced significant seismic events with "none to minor damage" is summarized in Figure 14, which compares each dam's height to the measured or estimated PGA that it experienced. (Dam names and locations can be found in Table 11.) In addition, the open symbols in Figure 14 represent dams that have been constructed in seismic regions but not yet subjected to a significant earthquake. (Since those dams are not listed in Table 11, they are identified in the Figure 14 notes.) The position of the proposed FRHEP embankment is indicated by the orange symbol in Figure 14, and two patterns are noteworthy. Firstly, comparing the FRHEP symbol to the coloured shapes representing case histories of seismic performance shows that many dams have survived earthquake PGAs greater than the FRHEP OBE, but those dams are all much lower in height than the proposed FRHEP embankment. Secondly however, there are several dams elsewhere in the world that are in the same height range as the FRHEP embankment, and have been designed and constructed for seismic conditions similar to those at the FRHEP.

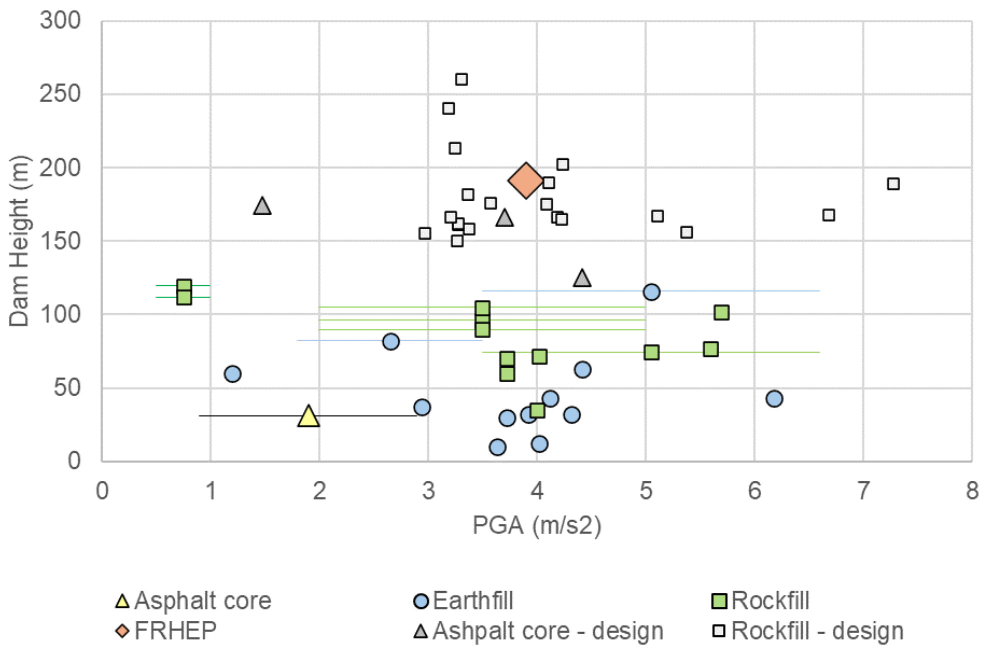



**Figure 14: Dams that performed well in significant earthquakes**

Available data from the Swaisgood (2003) database about the degree of damage experienced by rockfill and earthfill dams in earthquakes was also cited in Section 3.1.2. A subset of that database filtered to include only rockfill or earthfill dams that have experienced significant seismic events (PGA > 3 m/s$^2$) is summarized in Table 22. It is clear that the majority of rockfill and earthfill dams survived with "none or minor damage". The proposed FRHEP embankment will be constructed from rockfill, which appears from the Swaisgood (2003) data to perform better than earthfill and it will have an asphalt core that is expected to be more robust in earthquakes than the concrete facing used on many rockfill dams.

**Table 22: Damage to Dams Experiencing PGA > 3 m/s$^2$ (Data from Swaisgood 2003)**

| | Rockfill | Earthfill |
|---|---|---|
| **Total** | 8 | 13 |
| **None to minor damage** | 6 (75%) | 11 (61%) |
| **Moderate damage** | 1 (13%) | 3 (17%) |
| **Serious damage** | 1 (13%) | 4 (22%) |

### 4.1.3 Summary

The combination of statistical analysis and case histories indicates that the probability of a failure for the FRHEP embankment could be similar to, much less than, or much higher than global average probabilities of failure, and the determining factor will be the quality of design, construction, operations and governance.

Statistical analyses[2] always warrant qualification, in particular that conditions at a new site could be outside the range of past experience, and that the underlying data represent historical rather than current performance.

- Regarding the former limitation, the seismic conditions, foundation conditions, material quality, slope stability and other hazards at the FRHEP site have been noted in the project engineering reports and the requirements for follow up work have been clearly stated. It is reasonable to infer that these objective hazards could increase FRHEP probabilities of failure to the upper limit of the "low quality" prediction interval if design, construction, operations or governance are of low quality.
- Regarding the latter limitation, the ability to engineer a dam against hazards increases with time, and advances in seismic analysis and engineering have been particularly strong in recent decades. It is reasonable to expect that these improvements in the state of practice, if coupled with high quality design, construction, operations and governance, could result in FRHEP probabilities of failure being limited to the low end of the "average" range or high end of "high quality" range.

[2] Alternative approaches to quantify probabilities of failure were considered throughout this project, including the use of fault-event trees in the method commonly known as "quantitative risk assessment". Although this method has merits when the project is sufficiently advanced, the findings of the statistical analysis clearly showed that any quantitative risk assessment of the FRHEP that could be completed now would be dominated by assumptions about the quality of future design, construction, operations and governance. The SRK project team therefore recommended that quantitative risk assessment be left for a later stage of the project when the quality metrics can be better measured or estimated.

## 4.2 Potential Consequences of a FRHEP Embankment Failure

### 4.2.1 Key Patterns

The key patterns apparent in the dam breach modeling, are summarized in Figure 15:

- Outflows from FRHEP embankment breaches of 2 m or less do not cause downstream flooding (not shown in Figure 15)
- Outflows from breaches of 10 m or less generally remain within the river banks during seasonal low flows (Figure 15a) but cause downstream flooding during seasonal high flows (Figure 15b)
- A full breach that releases all water stored above the waste causes significant downstream flooding regardless of initial river levels (Figure 15c).

The tables in Section 3.2 show there are exceptions to the above at a few communities. But these patterns provide a concise summary of the dam breach and outflow model results.

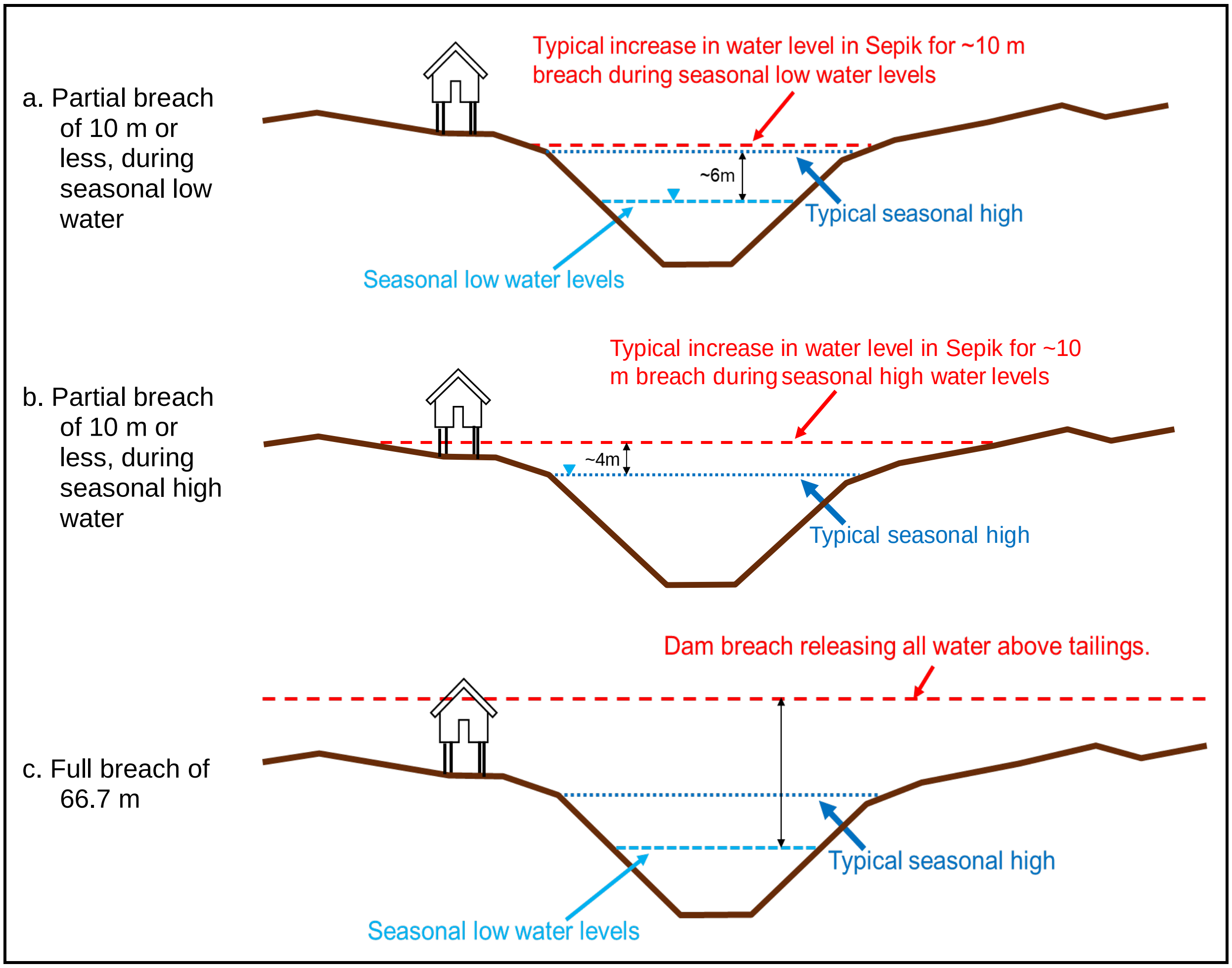



**Figure 15: Schematic illustration of key patterns in dam breach model results**

### 4.2.2 Possible Loss of Life

Potential fatality rates resulting from a breach of the FRHEP embankment will depend on the severity of the flooding at each community, the number of people in that community, and the ability for those people to move out of harm's way before the flood wave arrives.

Population estimates for major communities were provided by PanAust and are listed in Table 19 above. The ability for those people to avoid the flooding will depend on the available warning times and the adequacy of warning systems. Estimates of the time for the flood wave to reach each community were provided in Table 20 above, and are 4 hours or more for all communities (with the exception of Paupe, which is planned to be relocated). Based on experience at other dams, these arrival times are long enough to allow people to move out of harm's way if good warning system and community response plans are in place.

Many dam or river regulatory agencies have developed methods for estimating loss of life in flooding events, over wide ranges of flood severity, warning time and community preparedness. Results of one of these methods (RCEM, 2014) are summarized in Table 23 below. There are no expected fatalities for minor breach scenarios. For partial and full breaches, adequate and effective warning systems reduce the estimated number of fatalities by an order of magnitude.

Unfortunately, most of the methods used for this sort of calculation are based on American or European experience. The resulting numbers need to be viewed with caution when the method is applied to a very different part of the world. But the pattern shown in Table 23 will hold, specifically that the primary determinants of the risk to communities will be the adequacy of the warning system and community response plans. Both of these can reach a high standard if there is cooperation between the FRHEP team and downstream communities.

**Table 23: Estimated Fatality Ranges by Warning Effectiveness**

| **Scenario** | **Range of Potential Fatalities** | |
|---|---|---|
| | **Adequate and Effective Warning** | **No to Partial Warning** |
| Minor breaches | None expected | |
| Partial breaches<br>seasonal <u>low</u> initial water level | None expected | 1 – 8 |
| Partial breaches<br>seasonal <u>high</u> initial water level | 0 – 22 | 22 – 135 |
| Full breaches | 5 – 142 | 142 – 1284 |

## 4.3 Risk Evaluation

The term "risk evaluation" implies comparing an estimated risk to some measure of acceptability. The method most commonly applied in dam breach risk assessments is to compare the estimated probability of a breach and the estimated number of fatalities in the event of a breach to thresholds developed by dam safety regulators.

The example provided in Figure 16 is referred to as an "F-N plot". The horizontal axis provides a measure of impact, in this case the estimated number (N) of fatalities, typically on a logarithmic scale. The vertical axis indicates the probability or frequency (F) of those impacts, again on a logarithmic scale. Tolerability thresholds are typically shown as lines on the plot and vary by region and jurisdiction. Risks are generally considered acceptable when they plot below these thresholds, and unacceptable when they plot above. The F-N plot in Figure 16 includes threshold tolerability lines proposed by the US Army Corps of Engineers, the Australian National Committee on Large Dams, and the New South Wales Dam Safety Committee. Other operators and regulators of large dams propose very similar tolerability lines.

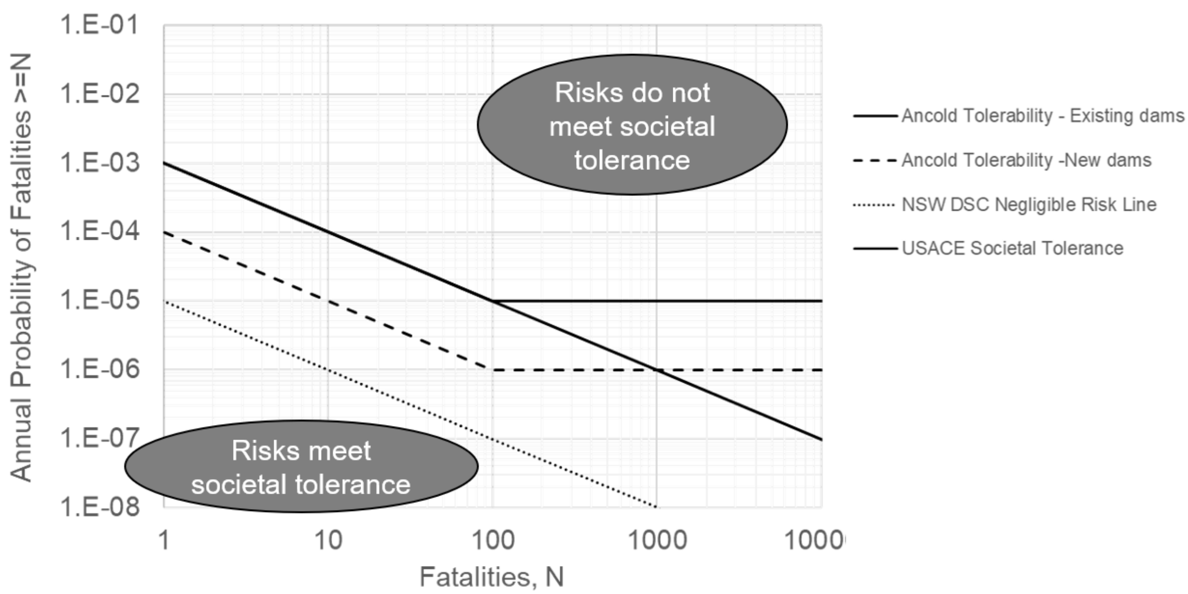



**Figure 16: Example F-N plot**

As discussed earlier, the likelihoods of failure are driven by the quality of investigation, design, construction, and governance. Figure 17 shows the ranges of estimated FRHEP failure probabilities plotted in the F-N format. It shows that FRHEP risks could range from much worse to a little better than global averages depending on quality of design, construction, operations, governance and getting to a range of acceptable failure likelihoods requires these all be the "best case".

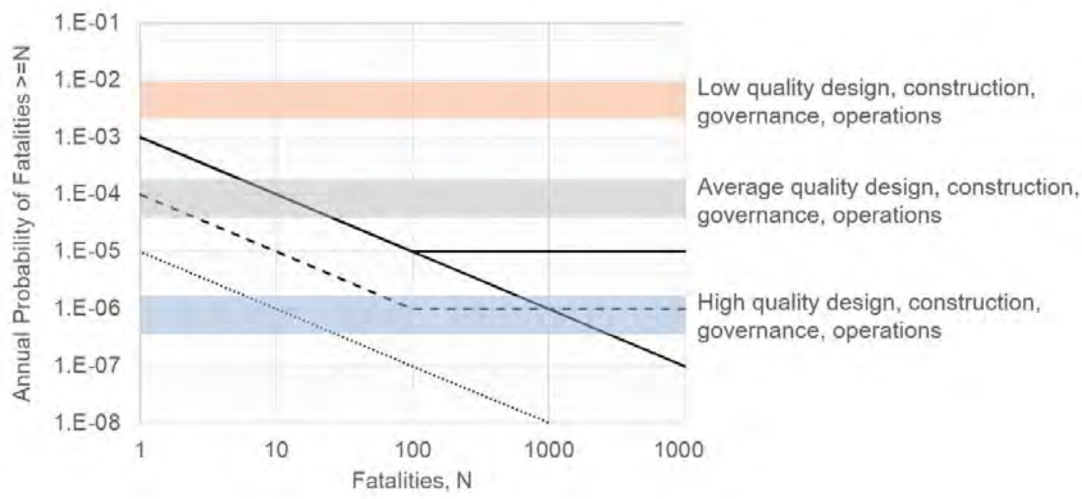



**Figure 17: Probability of failure ranges for FRHEP embankment shown on F-N plot**

Figure 18 plots the fatality estimates in the F-N format. The orange boxes indicate the range of potential fatalities associated with partial or less effective warning and evacuation and the green boxes show the ranges associated with good warning and evacuation. There are two sets of plots covering partial and full breaches. It shows that reducing the potential for fatalities and shifting left on the plot requires effective notification and response.

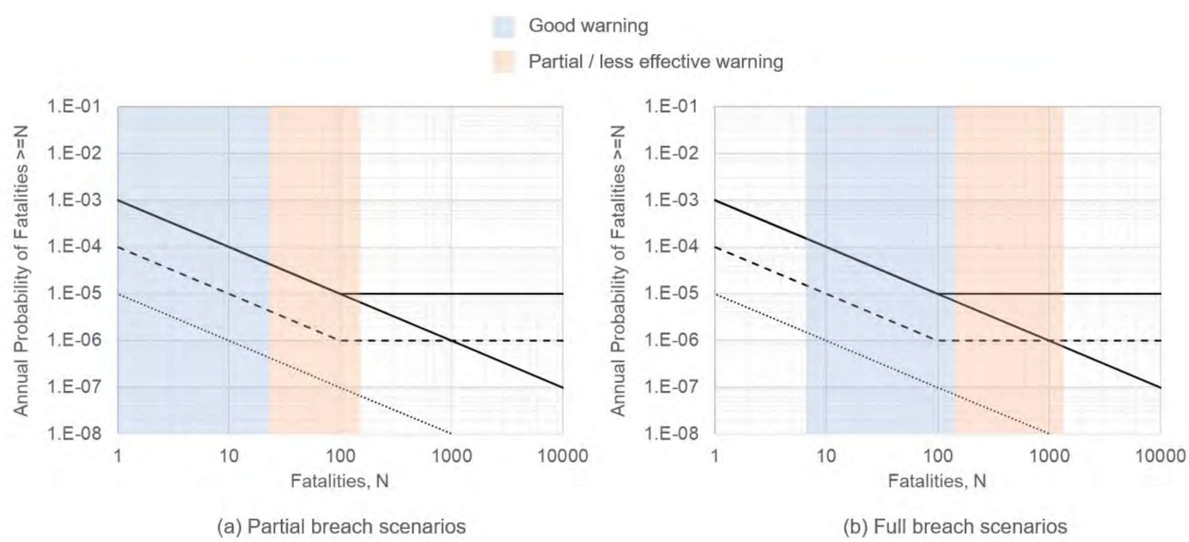



**Figure 18: Fatality ranges for FRHEP embankment breach outflow flood shown on F-N plot**

Figure 19 combines the FRHEP embankment failure probability and fatality estimates. The first observation is that the FRHEP embankment can present risks that range from intolerable to tolerable or even negligible. The arrows in Figure 19 indicate the major controls on failure probability and consequence severity. It is clear that ensuring the FRHEP embankment remains within tolerability limits will require BOTH high quality design, construction, operations, governance AND an effective warning and community response system.

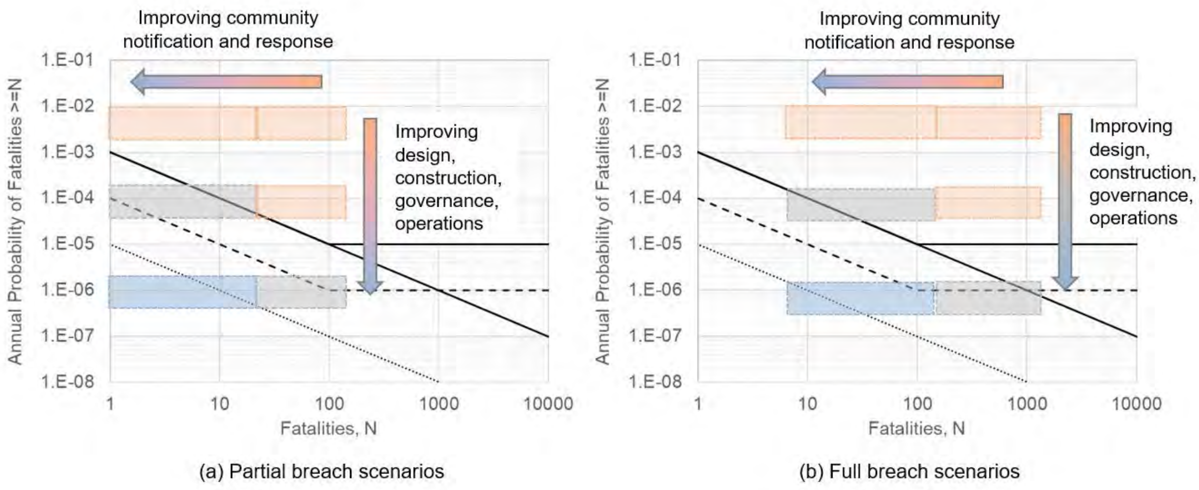



**Figure 19: Combined F-N results**

# 5 Conclusions and Recommendations

## 5.1 Conclusions

This risk assessment adopted statistical methods to estimate the probability of a failure of the proposed FRHEP embankment.  The results showed that with high quality design, construction, operation and governance, the FRHEP embankment can achieve probabilities of failure that are within the range of good hydroelectirc dams elsewhere in the world. However, if such high standards are not met, the dam could have a higher and unacceptable probability of failure. This highlights how critical the adoption of high standards is for the safety of the dam.

This risk assessment also analyzed possible downstream flooding and fatality risks in the event of a FRHEP embankment breach. Estimated flood depths depend primarily on the magnitude of the breach and can range from within natural flooding levels to several meters above the downstream riverbanks. Estimated flood arrival times at downstream communities range from hours to days, and therefore the risk of downstream fatalities can be controlled by community warning system and emergency response plans.

Ensuring that failure probabilities and downstream risks associated with the FRHEP embankment remain within tolerability limits will require both:

- high quality design, construction, operations and governance; and
- an effective warning and response system.

## 5.2 Recommendations

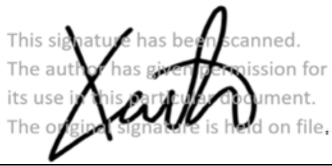
Specific recommendations regarding FRHEP design, construction, operations, and governance are presented in the “Selection Phase Study” documents, including a “Forward Works Report” that focuses on the next steps of engineering and project development.

Regarding warning and community response systems, the first priority is to seek informed input from the potentially affected communities. Specific topics arising from this risk assessment include:

- More complete population estimates for all communities.
- Current community responses to natural floods.
- Routes to higher ground or other emergency response plans.
- Possible warning/notification systems for future floods.
- Other methods to increase community awareness and emergency preparedness.

# Closure

This report, FRHEP Dam Risk Assessment – Technical Report, was prepared by

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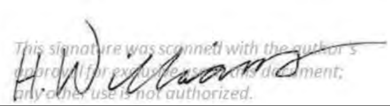

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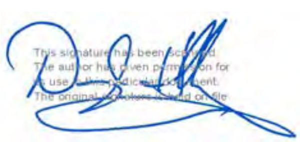

Daryl Hockley, PEng
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All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

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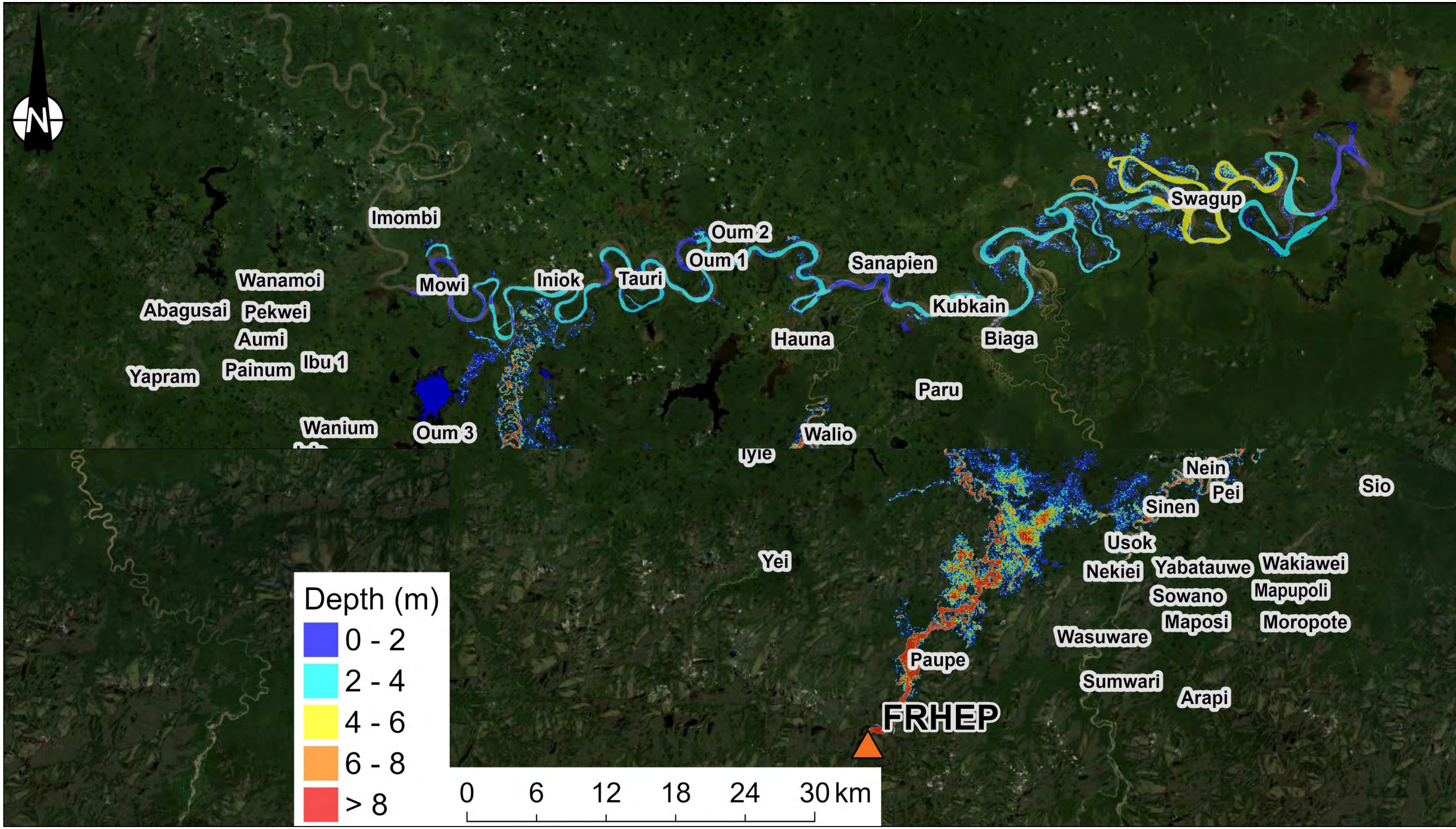
# Appendix A Flooding and Velocity Overview Maps




Notes:

srk consulting

PANAUST

Job No: CAPR003035

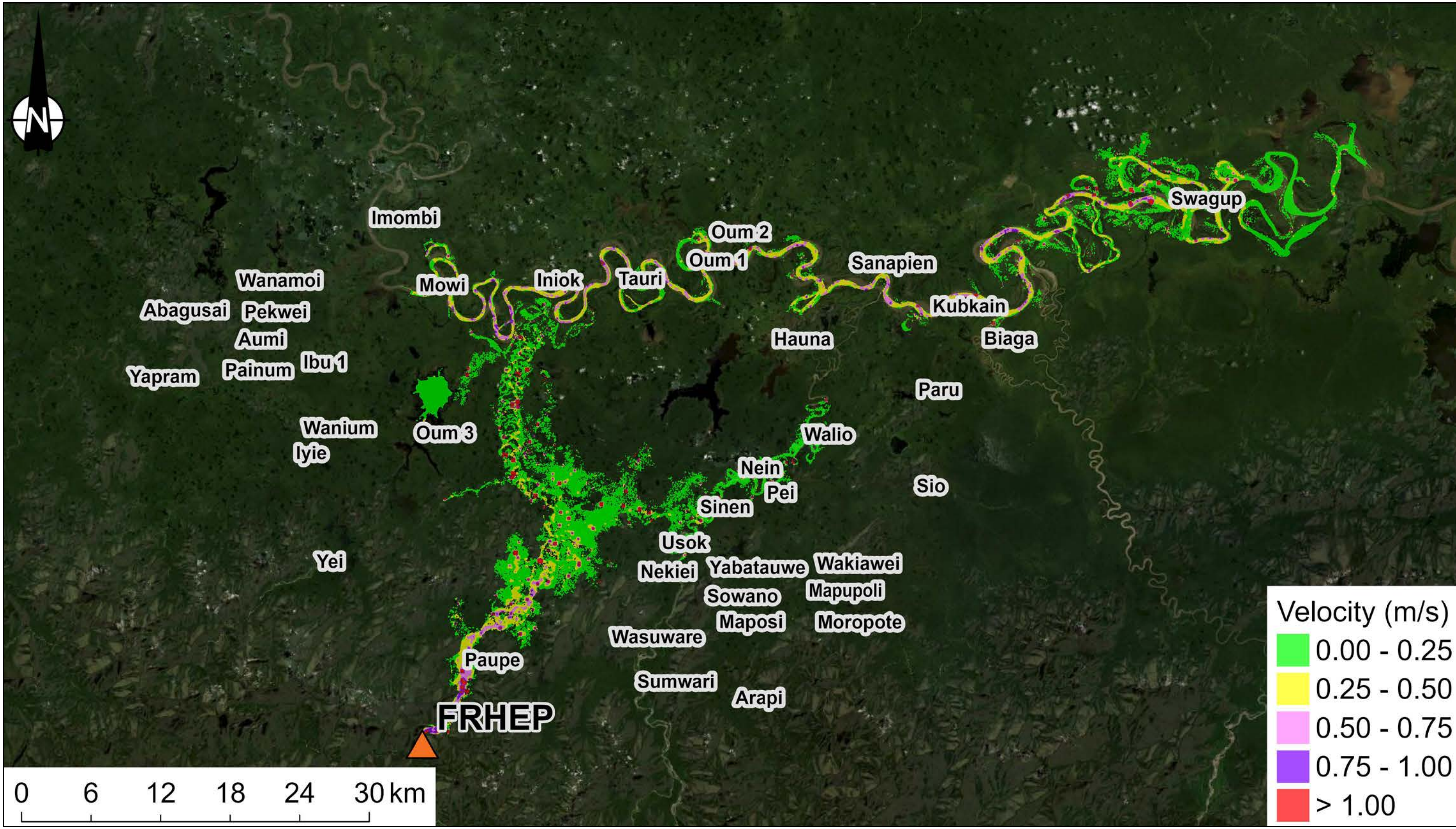
Filename: Overviews xxxx.pptx

Frieda River Dam

FRHEP Dam Risk Assessment Technical Report

**Scenario 1 – Minor Breach (Depth)**

| Date: | Approved: | Figure: |
|---|---|---|
| May 2024 | ABL | X.X |




Notes:

| srk consulting | PanAust | FRHEP Dam Risk Assessment Technical Report |
|---|---|---|
| | | **Scenario 1 – Minor Breach (Velocity)** |
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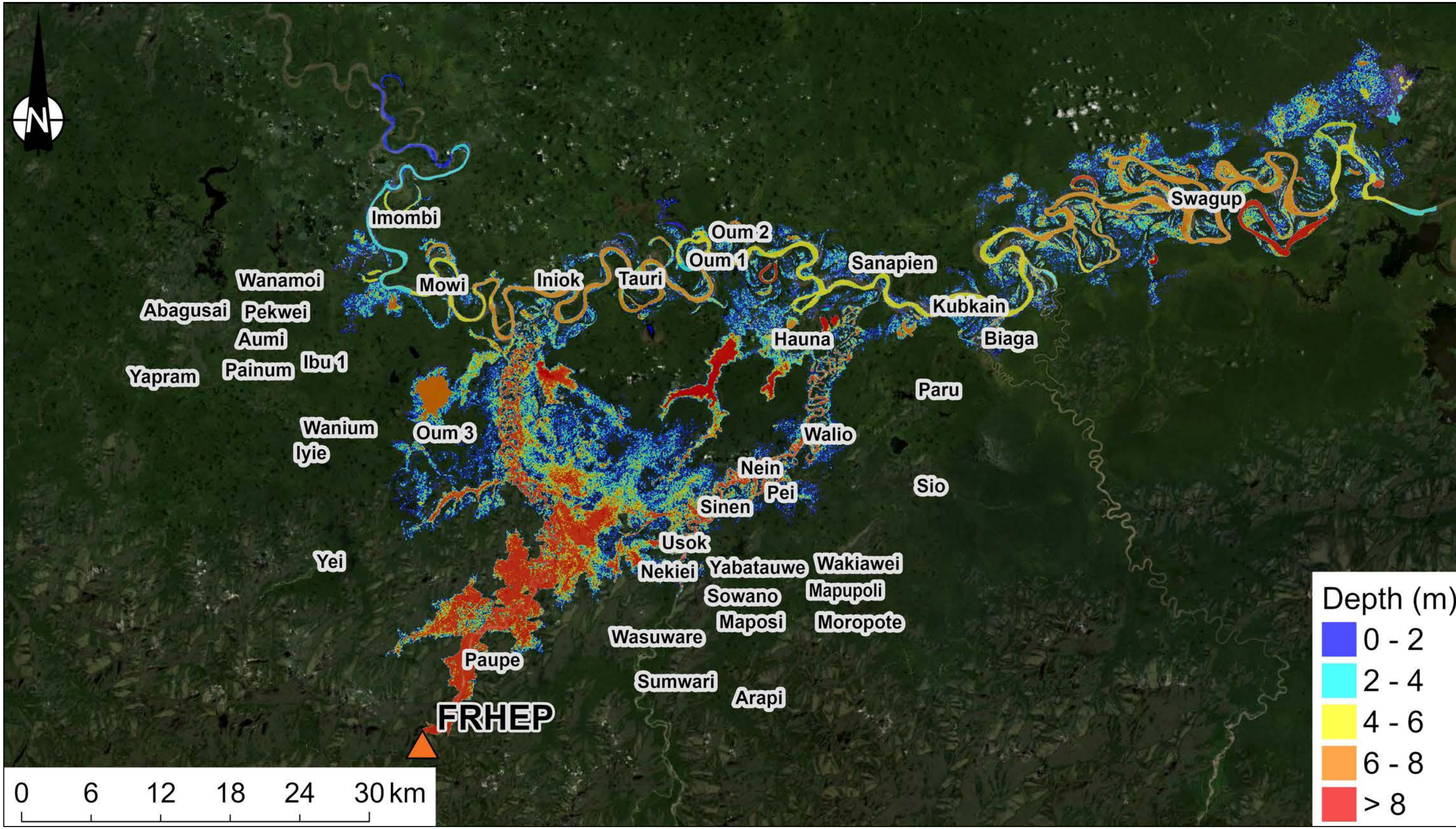


Notes:

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**Scenario 2 – Intermediate Breach – Seasonal Low Water Levels (Depth)**

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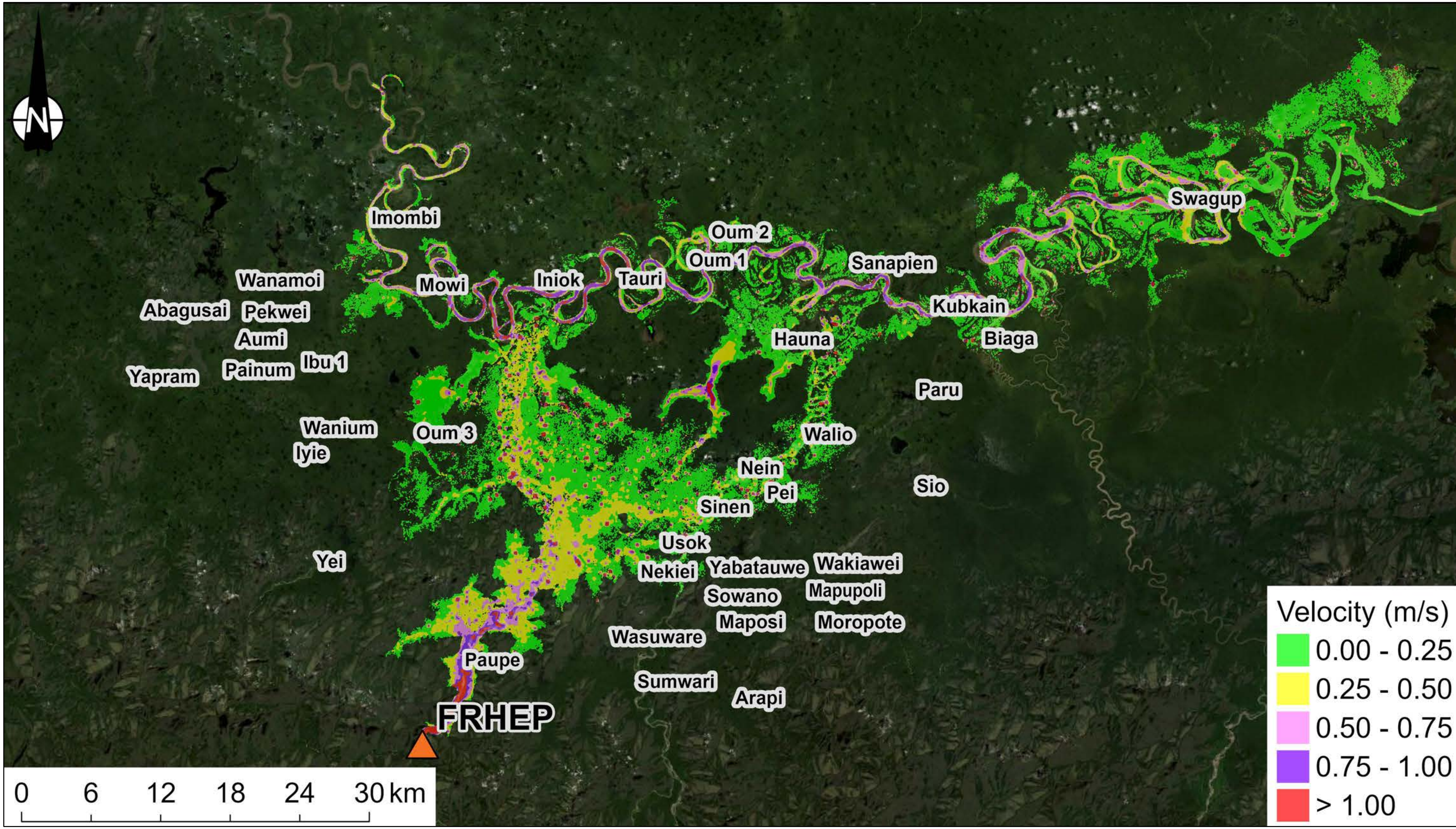
Filename: Overviews xxxx.pptx

Frieda River Dam

| Date: | Approved: | Figure: |
|---|---|---|
| May 2024 | ABL | X.X |




Notes:

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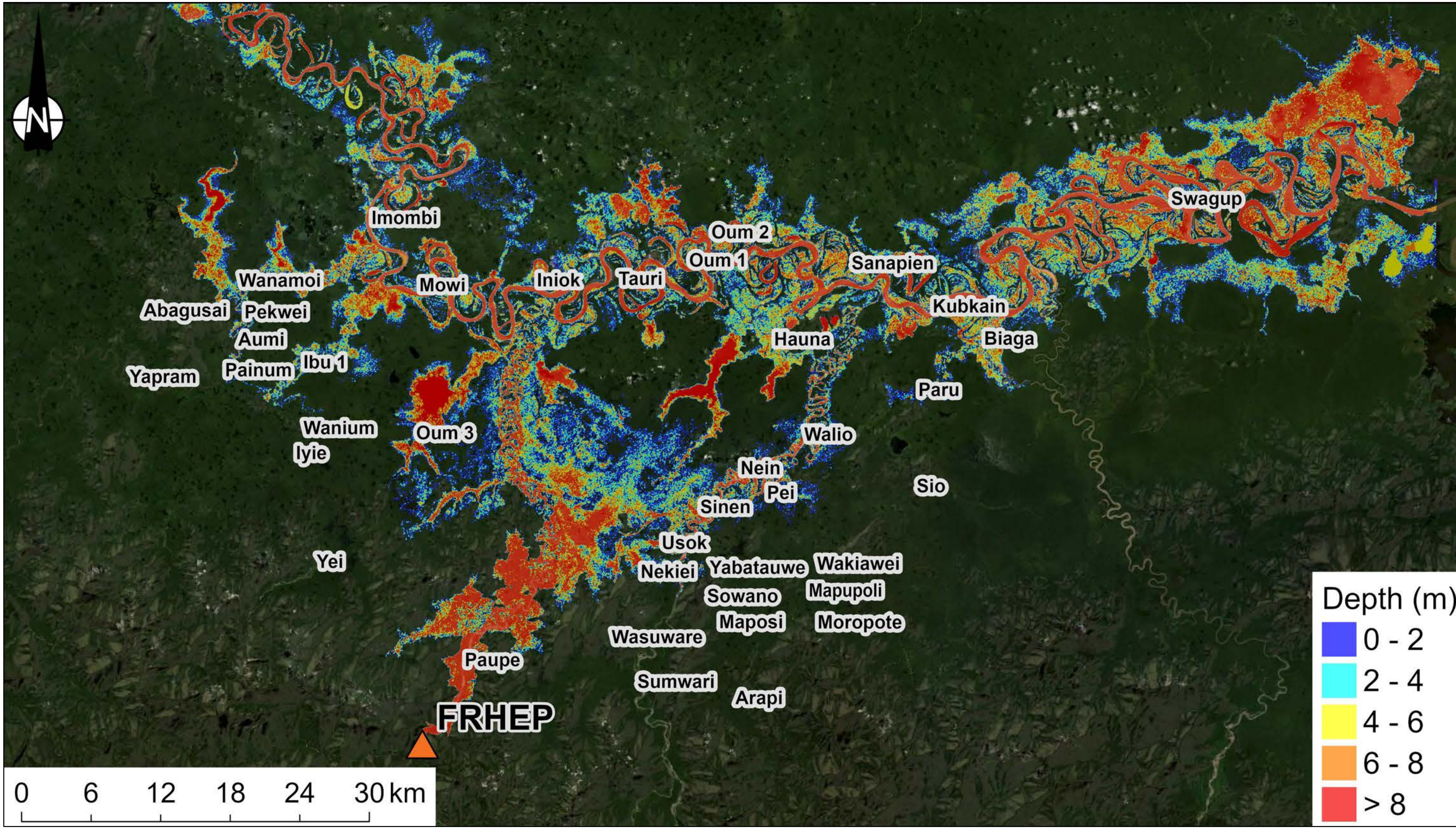
**Scenario 2 – Intermediate Breach – Seasonal Low Water Levels (Velocity)**

Job No: CAPR003035
Filename: Overviews xxxx.pptx

Frieda River Dam

Date: May 2024 | Approved: ABL | Figure: X.X

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| srk consulting | PanAust | FRHEP Dam Risk Assessment Technical Report |
|---|---|---|
| | | **Scenario 2 – Intermediate Breach – Seasonal High Water Levels (Depth)** |
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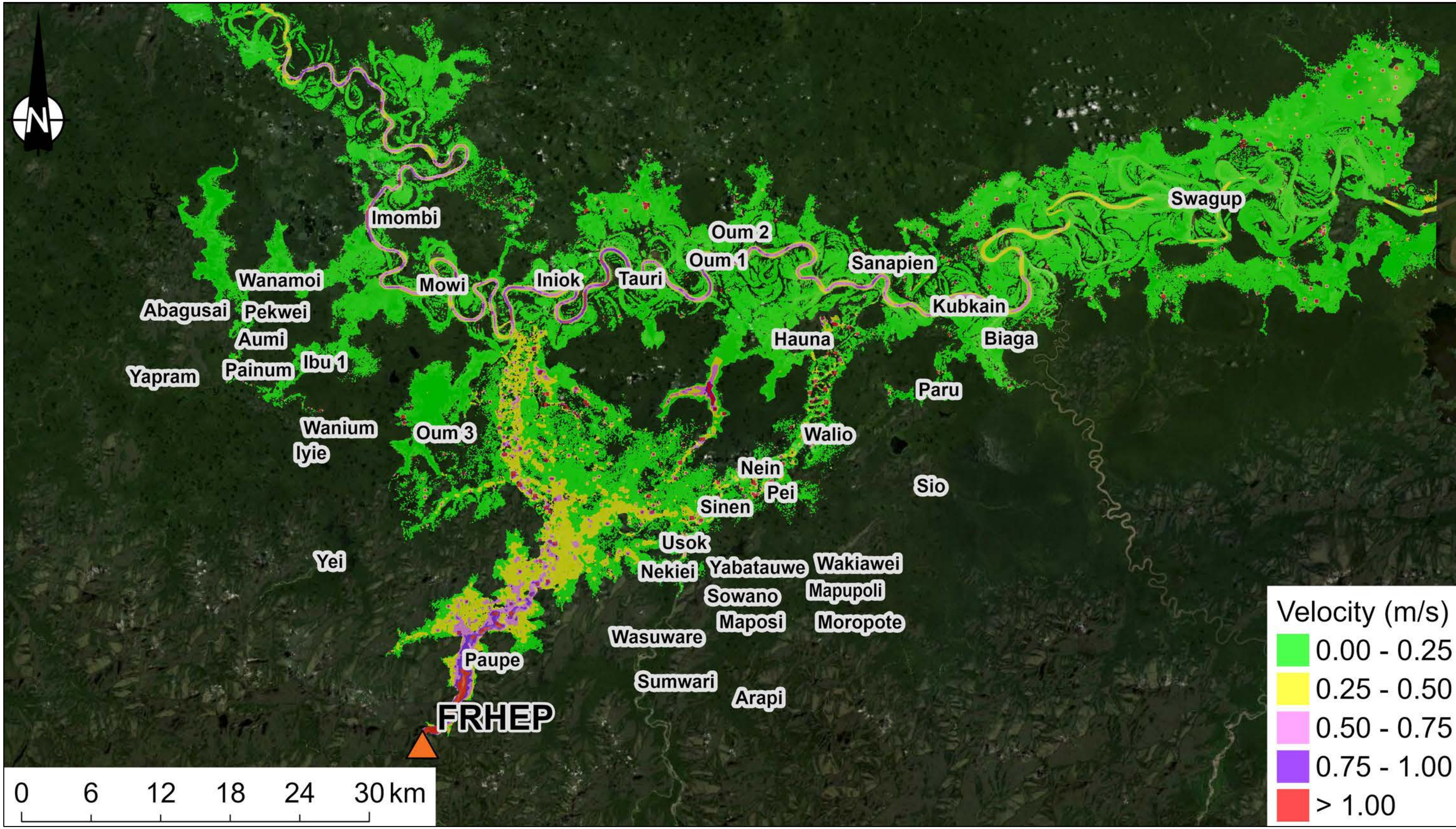


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srk consulting

PanAust

Job No: CAPR003035

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Frieda River Dam

FRHEP Dam Risk Assessment Technical Report

**Scenario 2 – Intermediate Breach – Seasonal High Water Levels (Velocity)**

| Date: | Approved: | Figure: |
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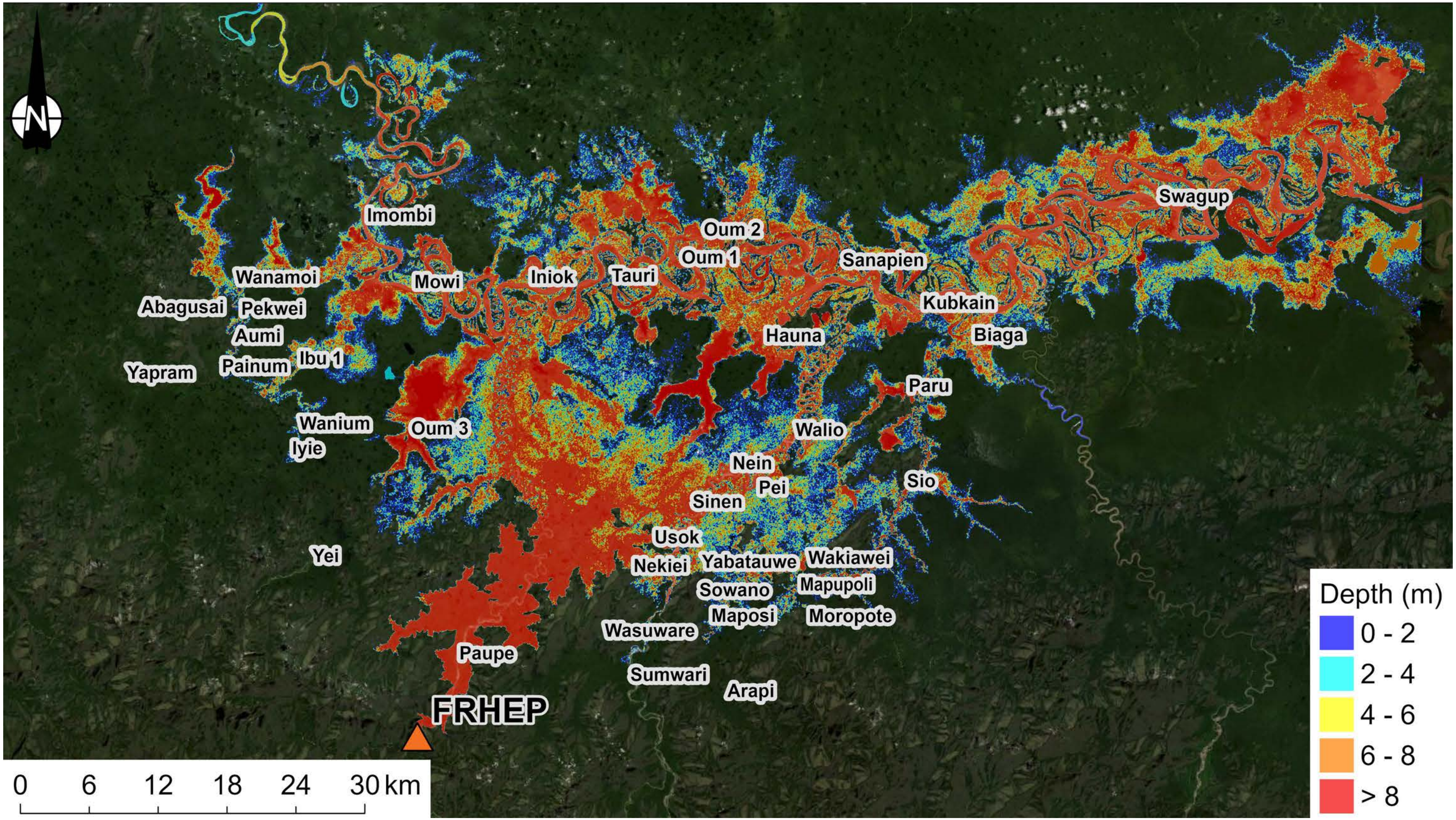
| May 2024 | ABL | X.X |

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|---|---|---|---|---|
| | | Scenario 3 – Maximum Breach | | |
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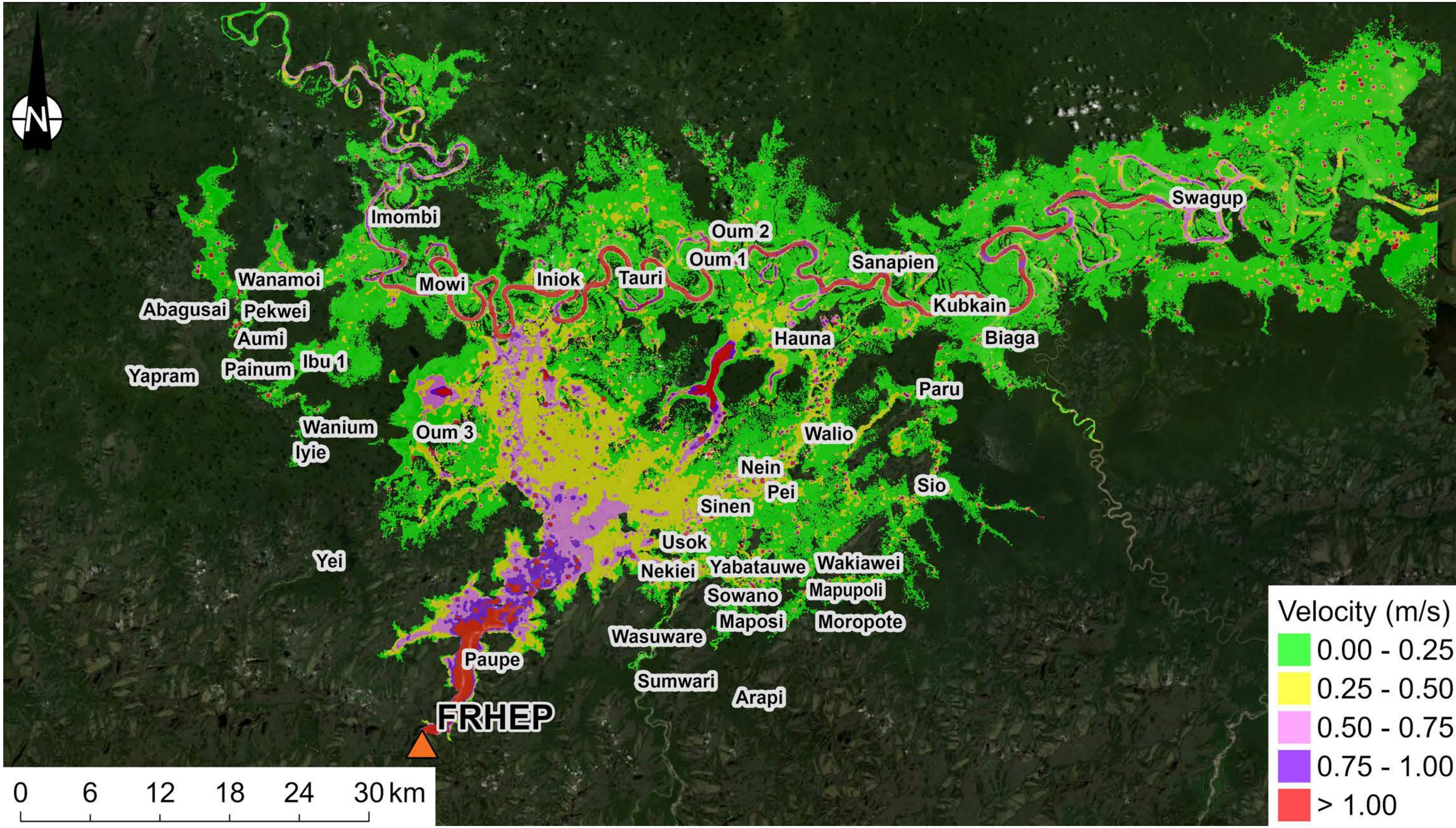
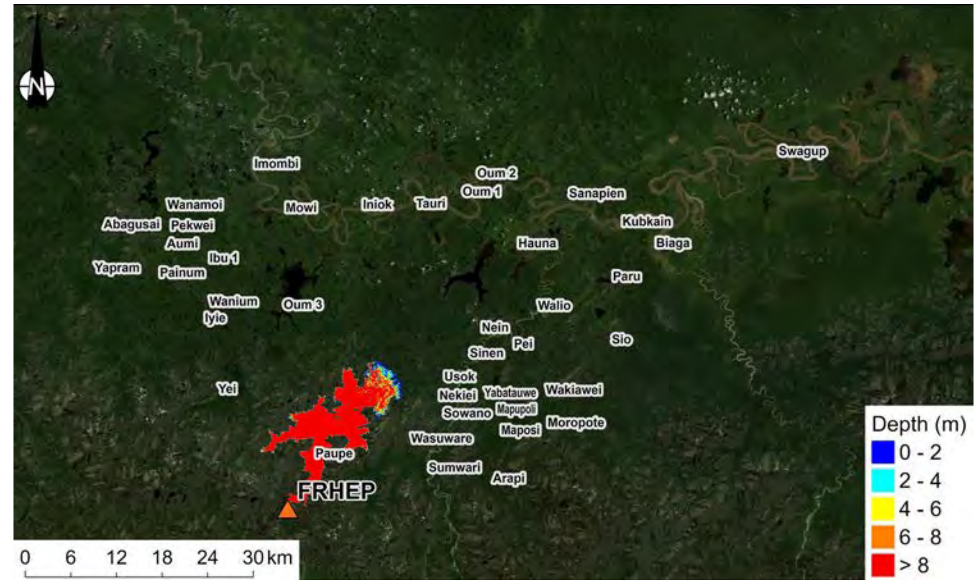


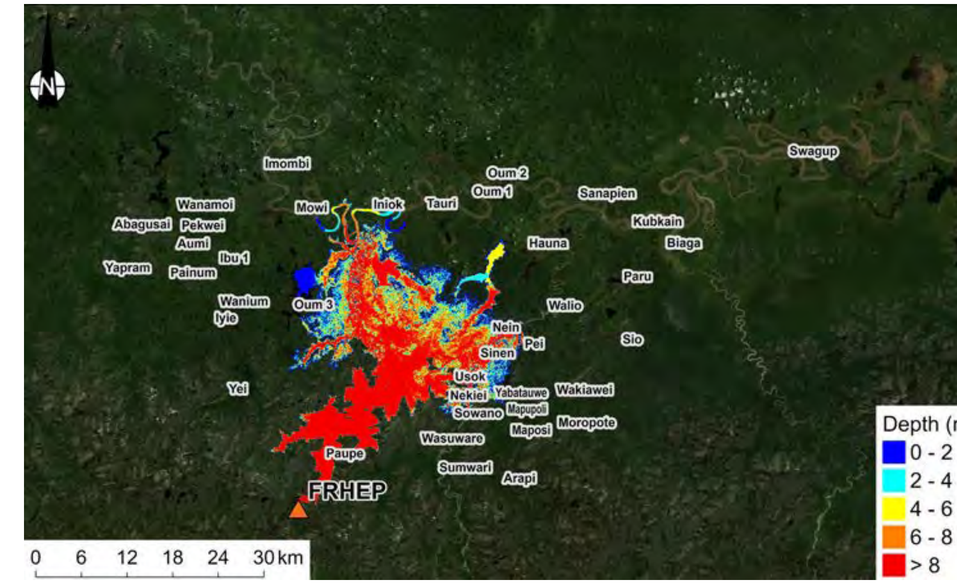

Notes:

| srk consulting | PanAust | FRHEP Dam Risk Assessment Technical Report |
|---|---|---|
| | | **Scenario 3 – Maximum Breach** |
| Job No: CAPR003035<br>Filename: Overviews xxxx.pptx | Frieda River Dam | Date: May 2024 / Approved: ABL / Figure: X.X |

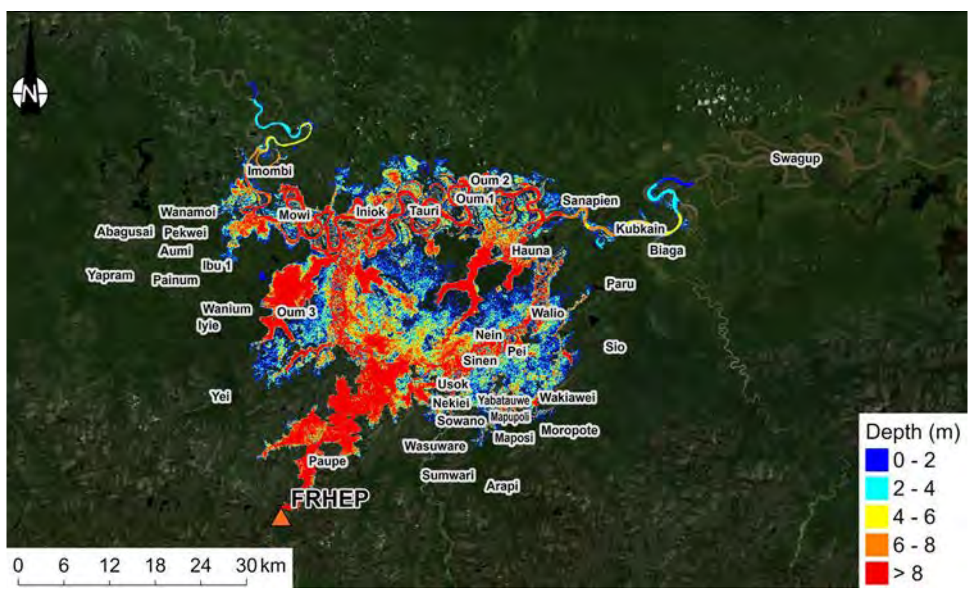

Prior to Breach



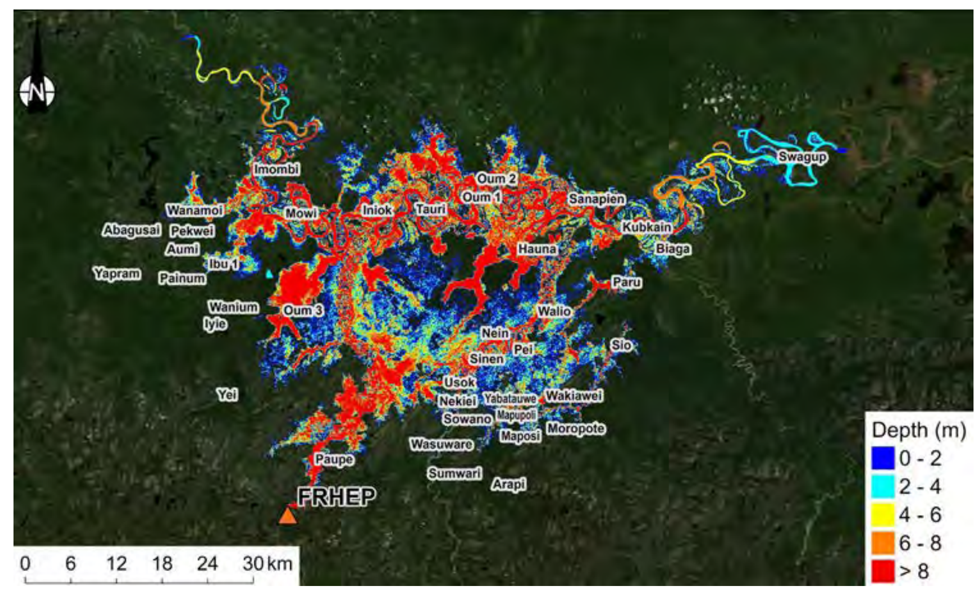

2 – 10 Hours After Breach



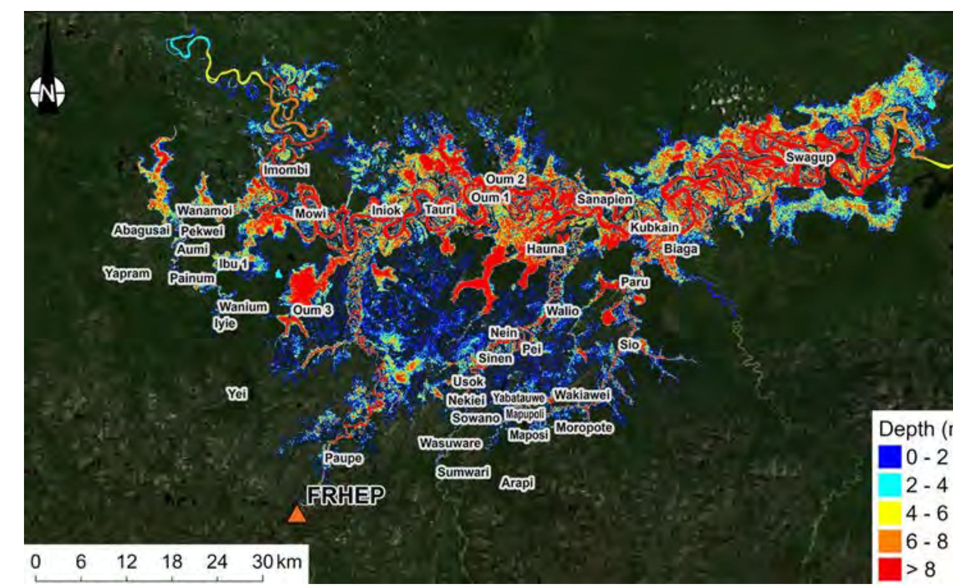

6 – 24 Hours After Breach




18 – 48 Hours After Breach




~ 5 Day After Breach




~ 7 Days After Breach

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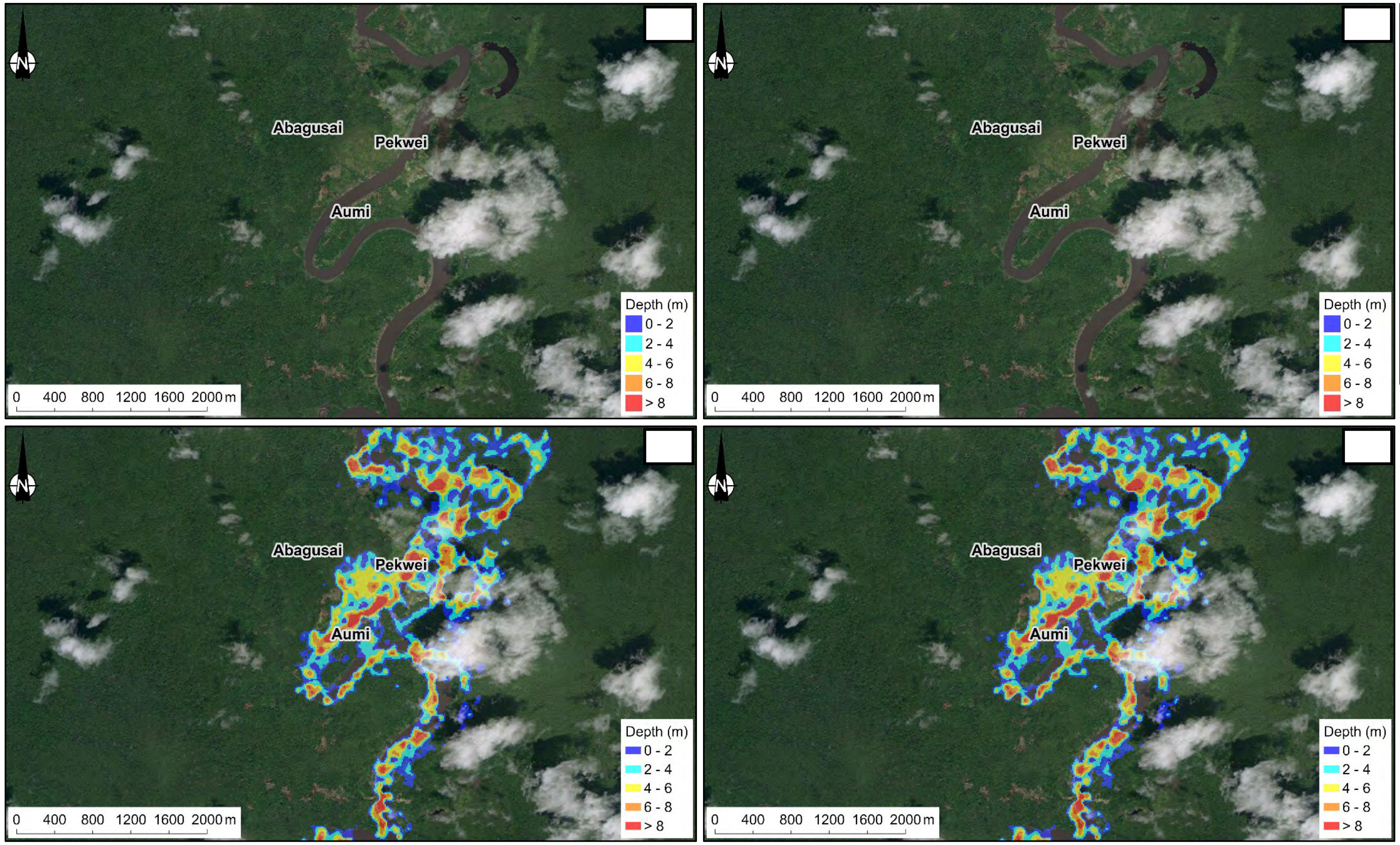
**Stages of Maximum Breach**

Job No: CAPR003035
Filename: Overviews xxxx.pptx

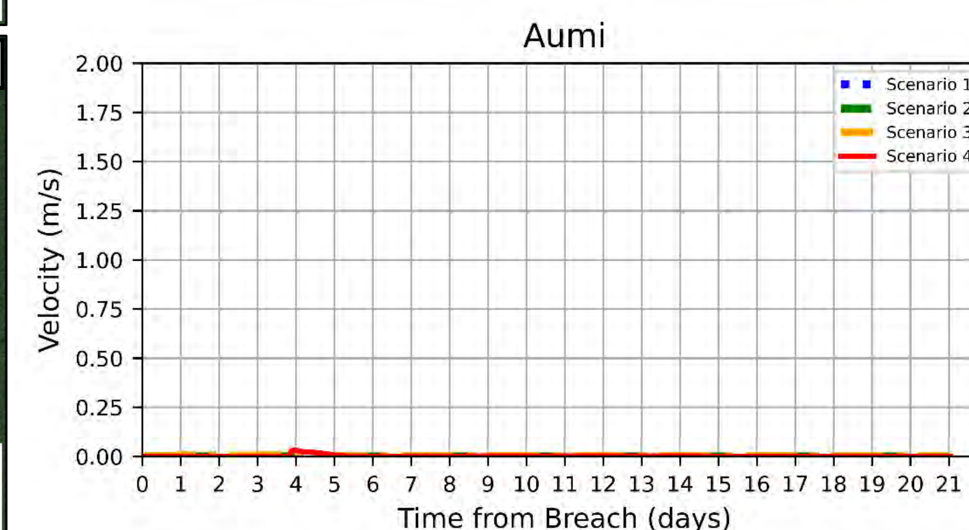
Frieda River Dam

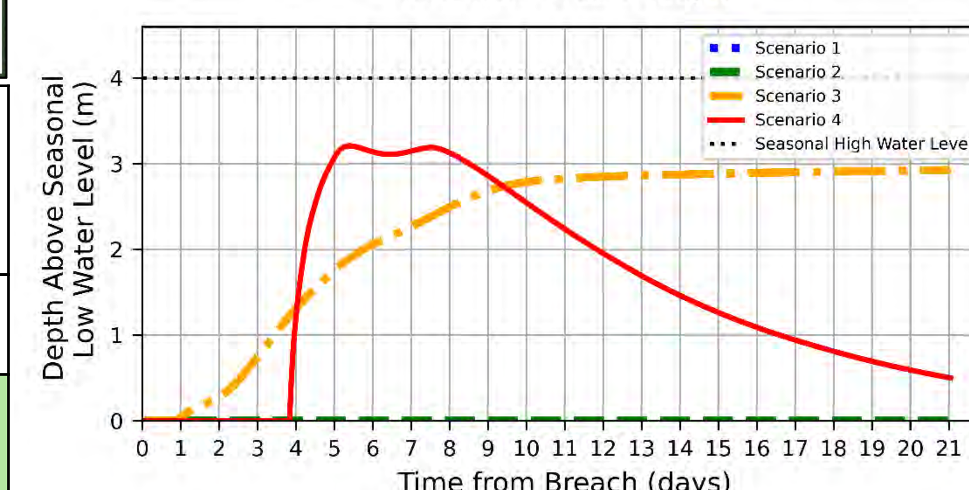
Date: May 2024 | Approved: ABL | Figure:

# Appendix B Village-by-Village Flooding Maps








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | 2.93, [-], 0.01 | 3.21, [-], 0.26 |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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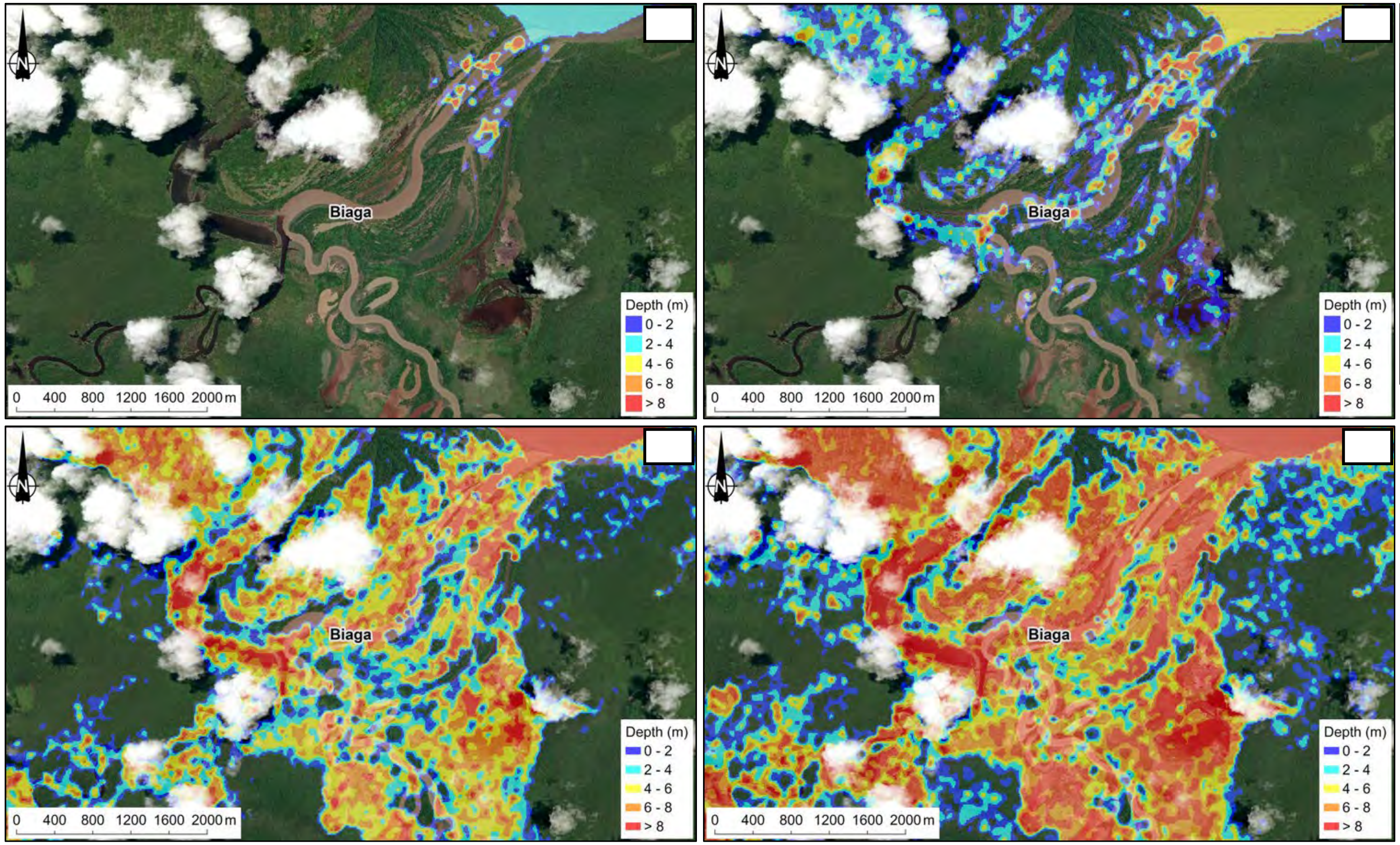
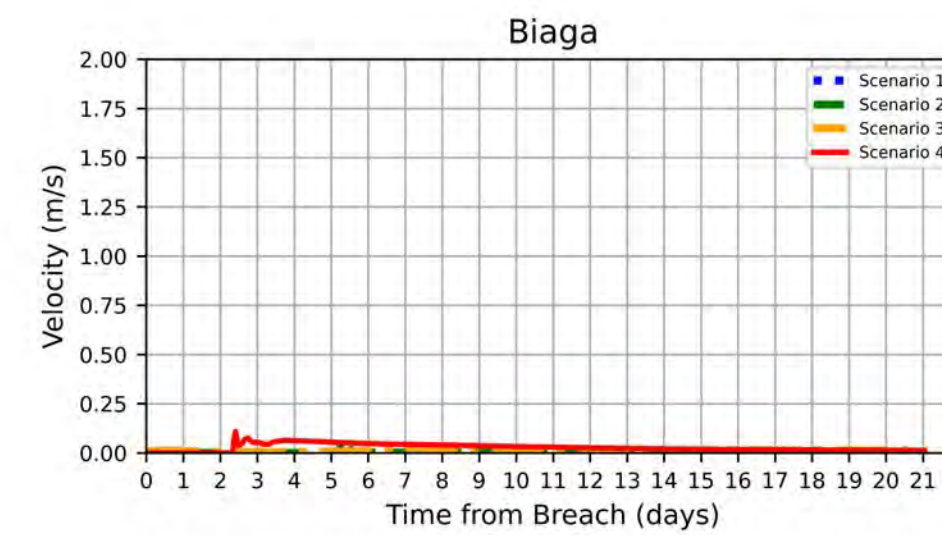
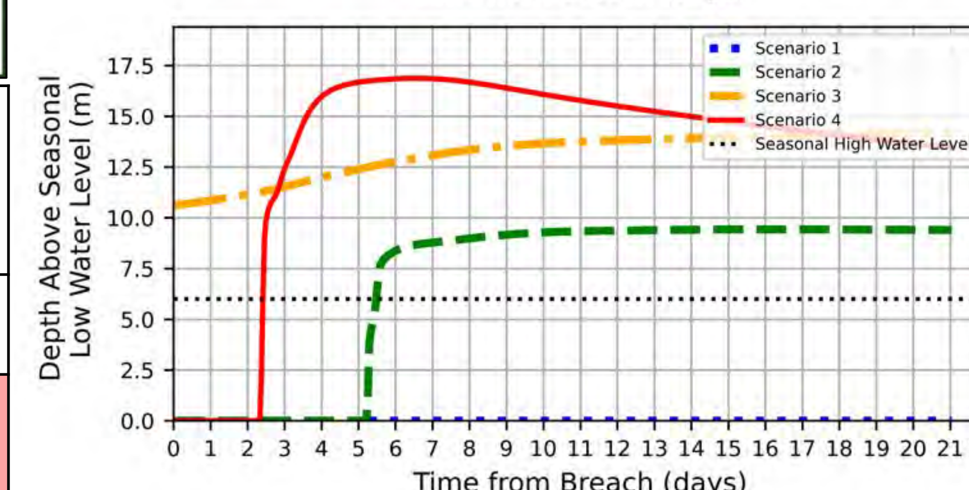
| Job No: CAPR003035<br>Filename: FriedaRiver_VillagebyVillageAssessment.pptx | Frieda River Dam | Date: May 2024 | Approved: ABL | Figure: X.X |








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | **0.26 $Bm^3$** | **1.24 $Bm^3$** | **1.24 $Bm^3$** | **6.37 $Bm^3$** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **9.433, [3.433], 0.11** | **14.15, [8.15], 0.02** | **16.87, [10.87], 0.23** |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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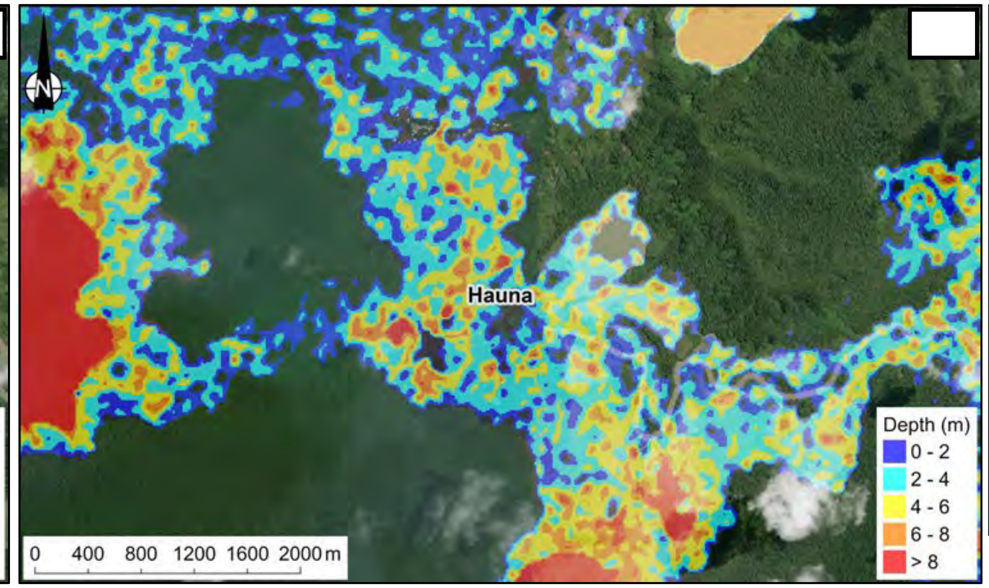



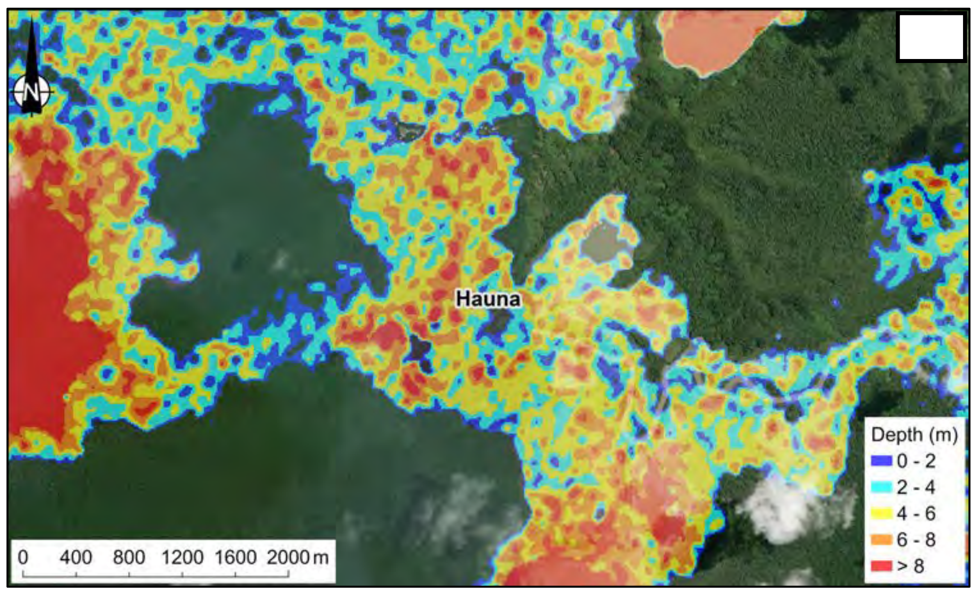


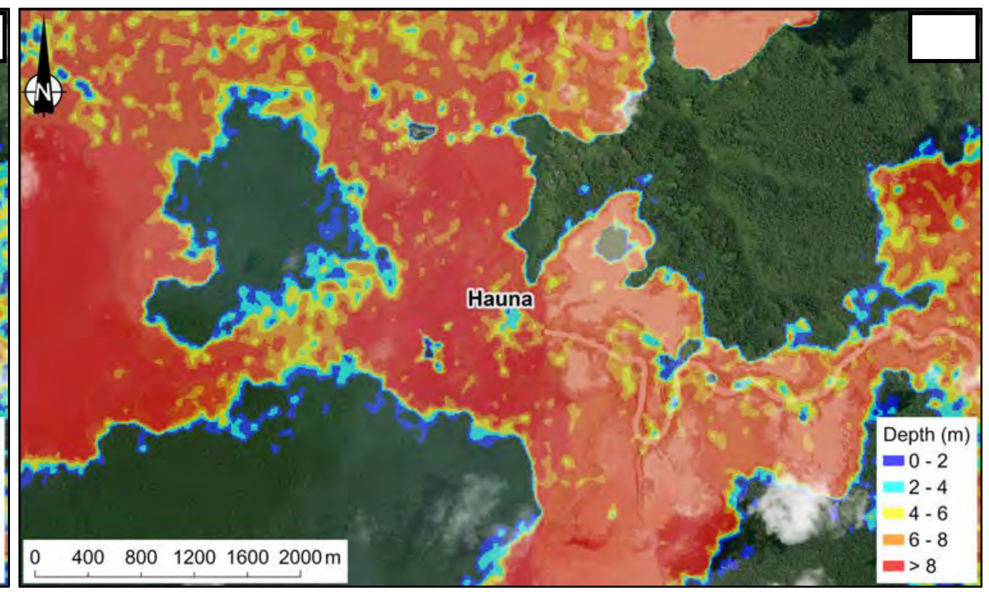


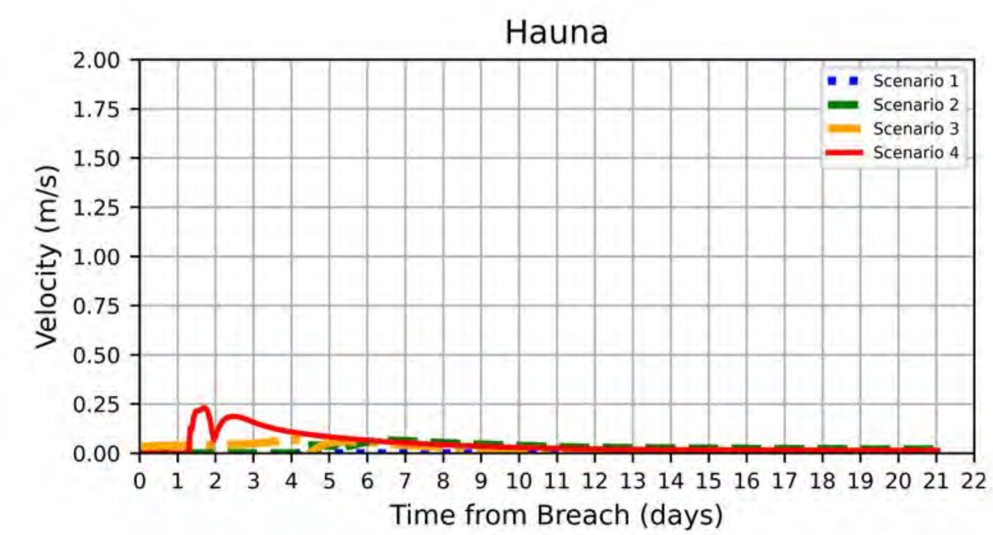


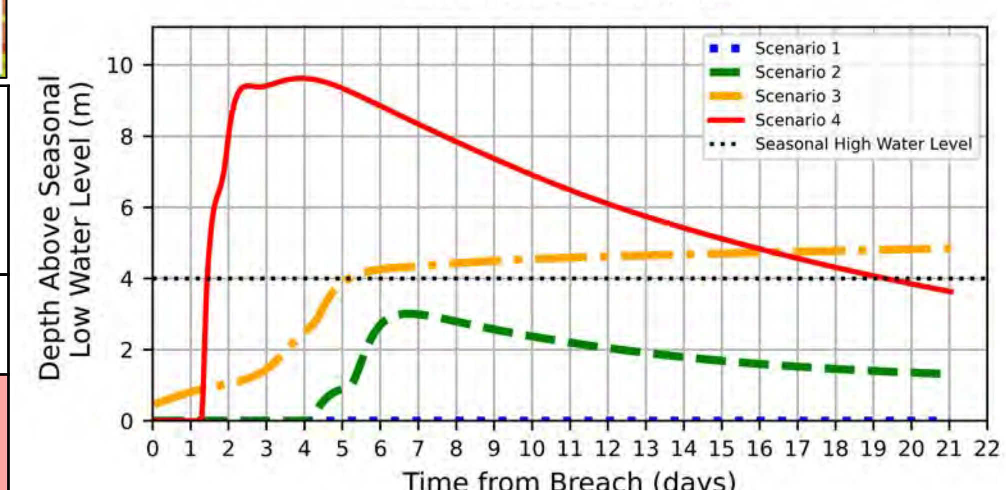


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | 3.01, [-], 0.05 | 4.84, [0.84], 0.06 | 9.43, [5.43], 0.47 |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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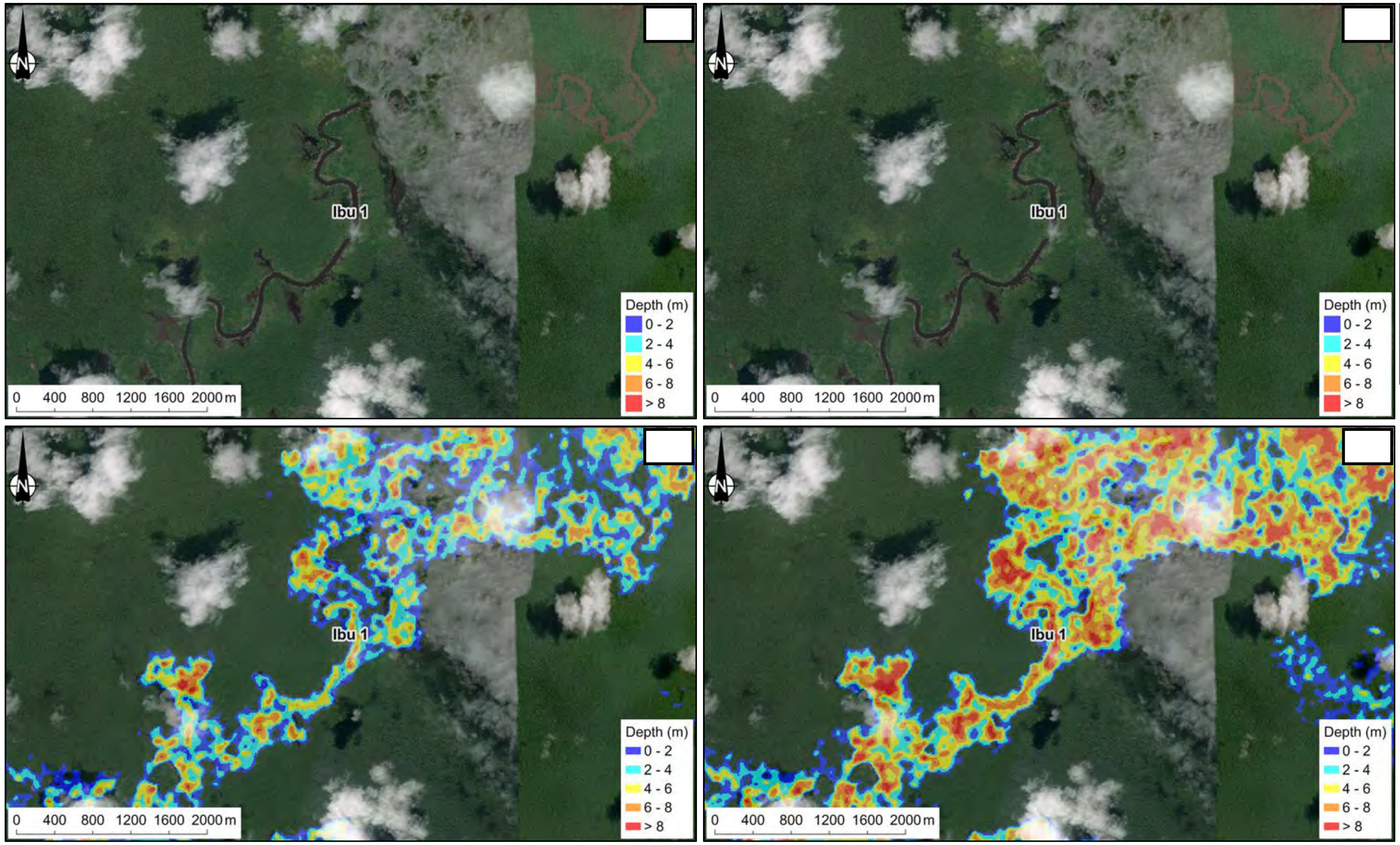



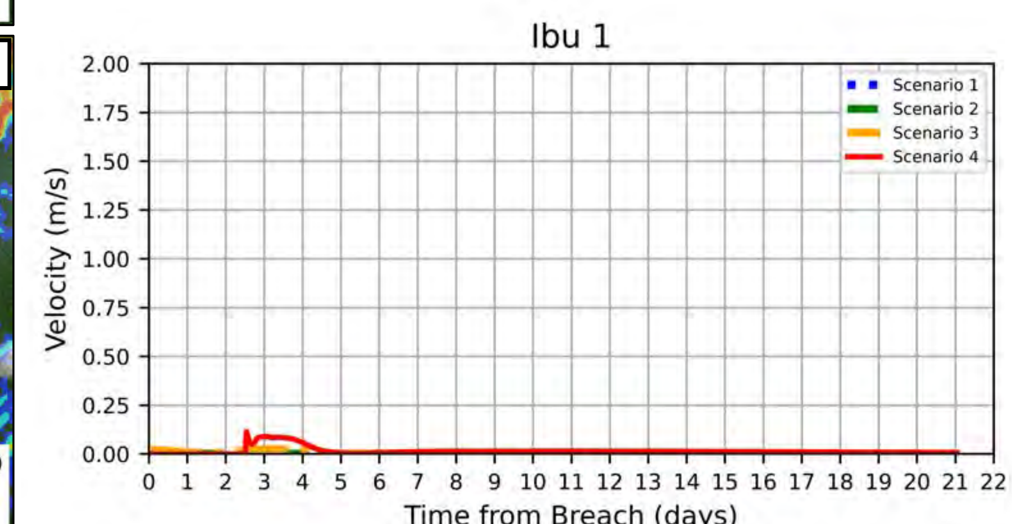


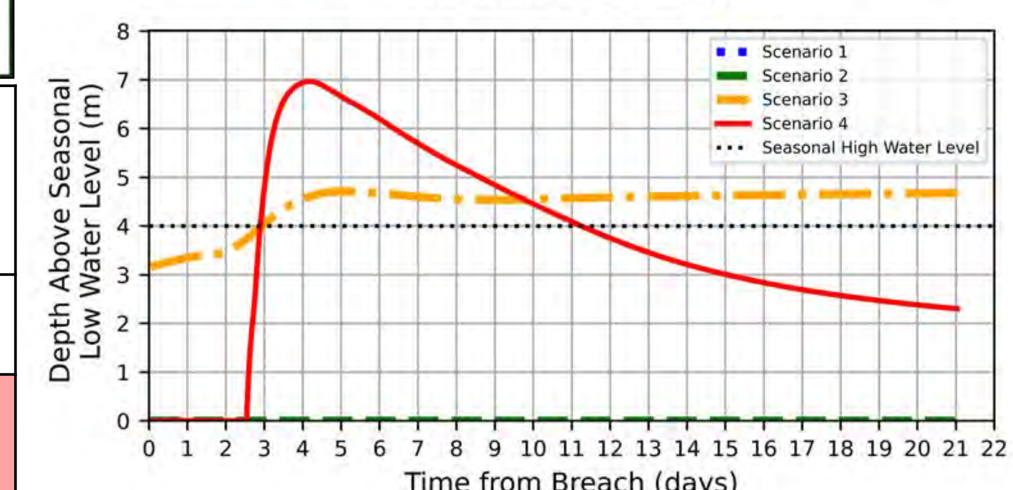


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | **0.26 $Bm^3$** | **1.24 $Bm^3$** | **1.24 $Bm^3$** | **6.37 $Bm^3$** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **Not Inundated** | **4.71, [0.71], 0.02** | **6.97, [2.97], 0.17** |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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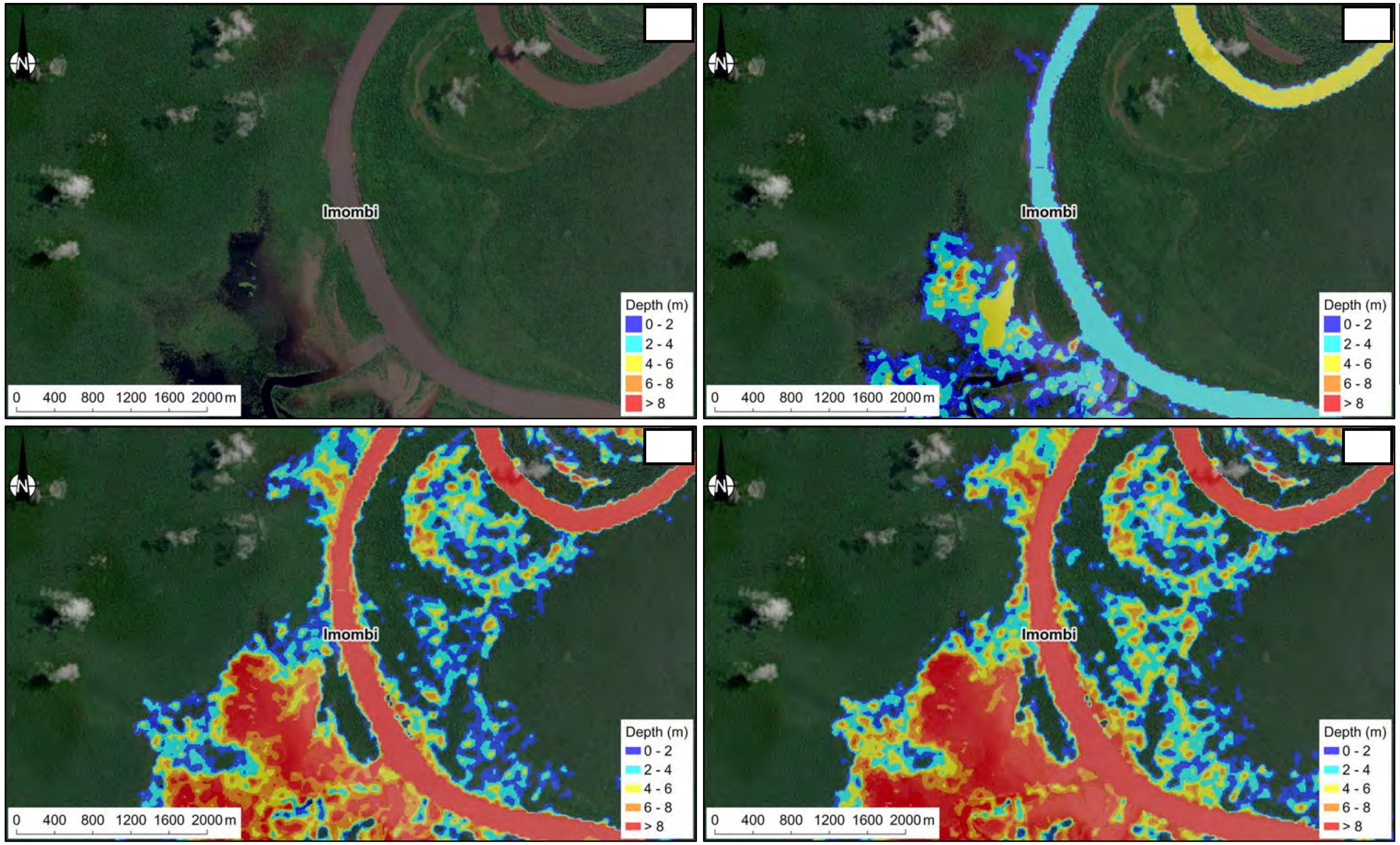



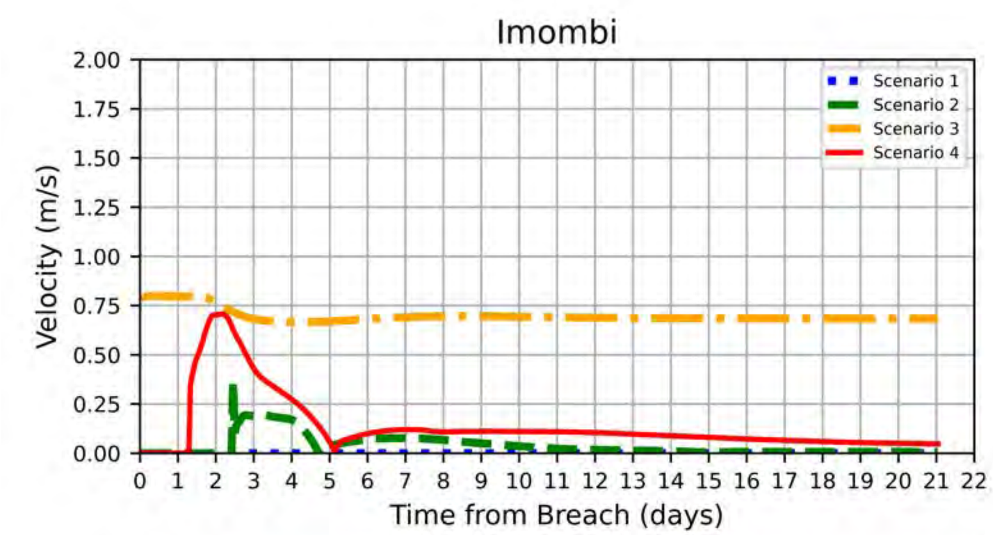


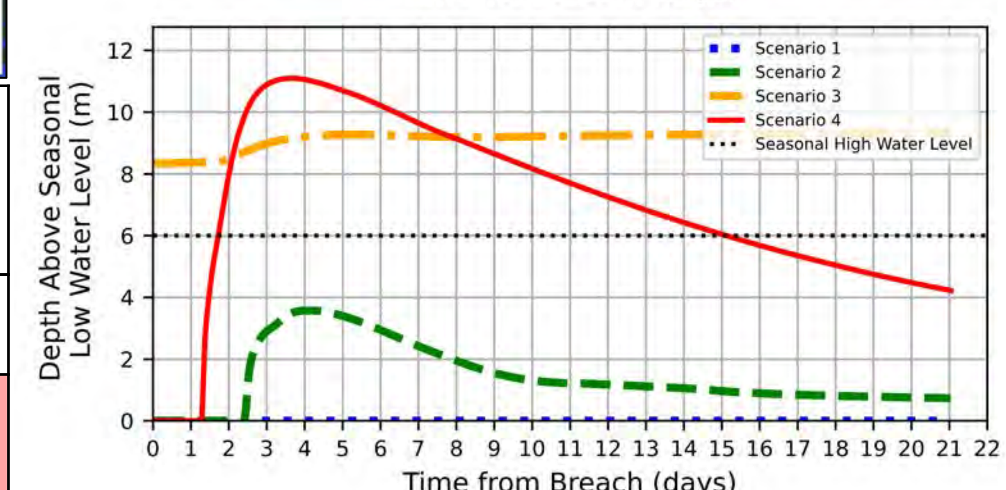


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | Not Inundated | 3.57, [-], 0.47 | 9.31, [3.31], 0.8 | 11.1, [5.1], 0.85 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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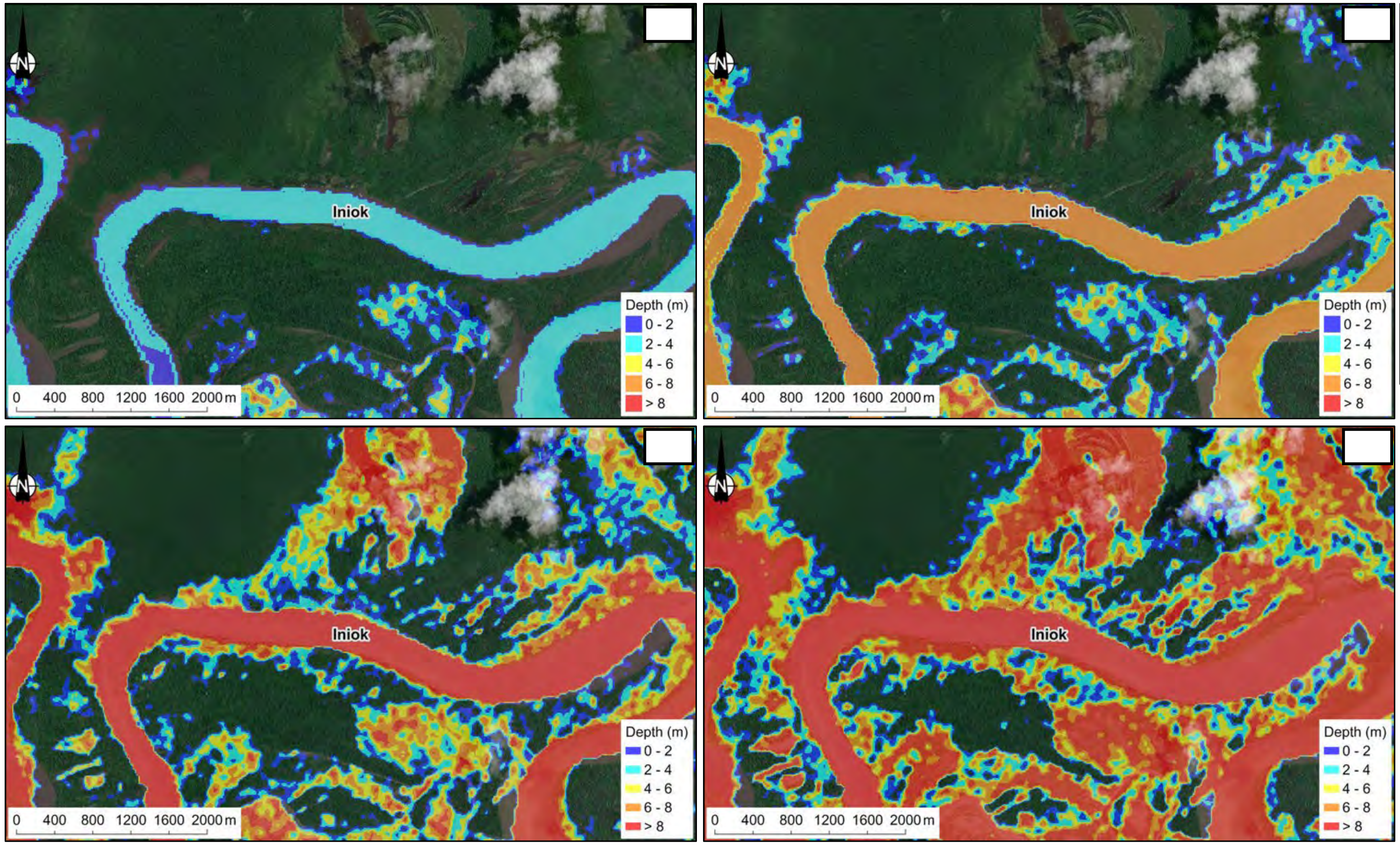



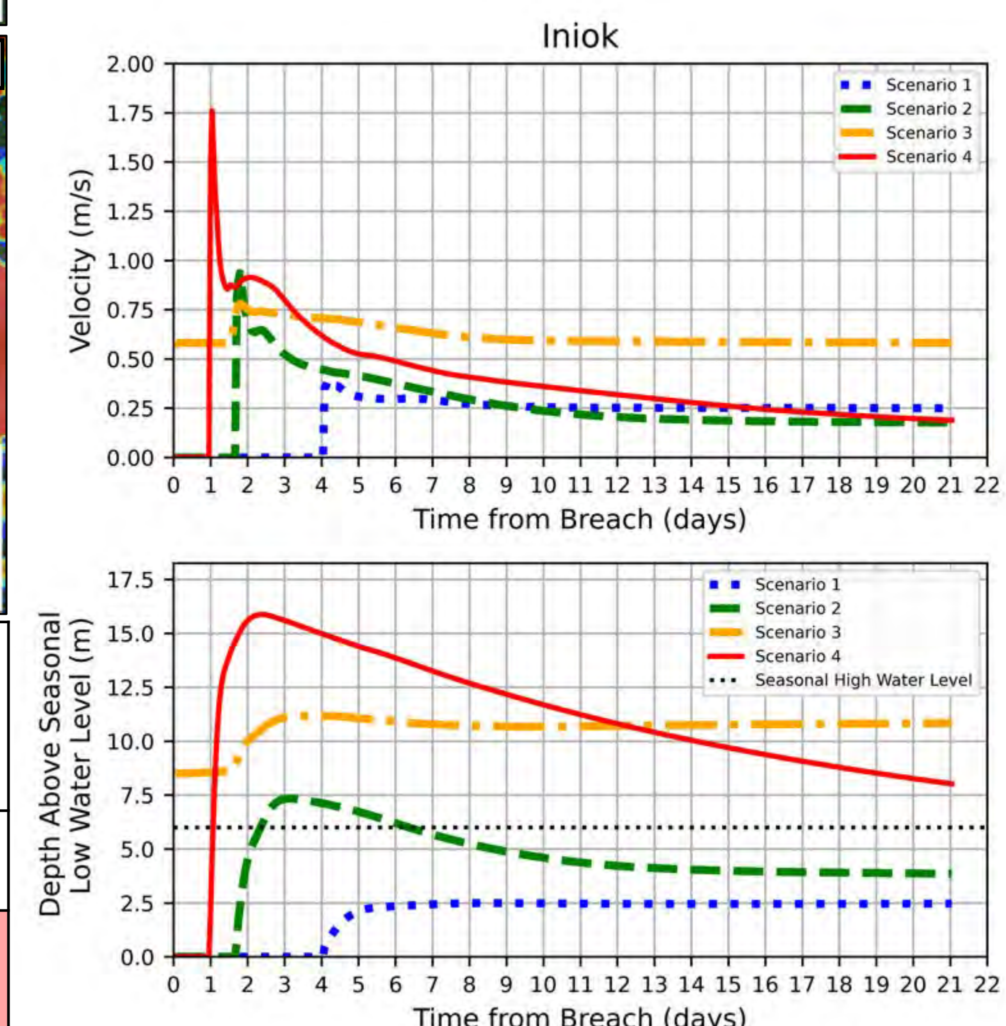


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 2.49, [-], 0.45 | 7.34, [1.34], 0.98 | 11.17, [5.17], 0.78 | 15.88, [9.88], 1.81 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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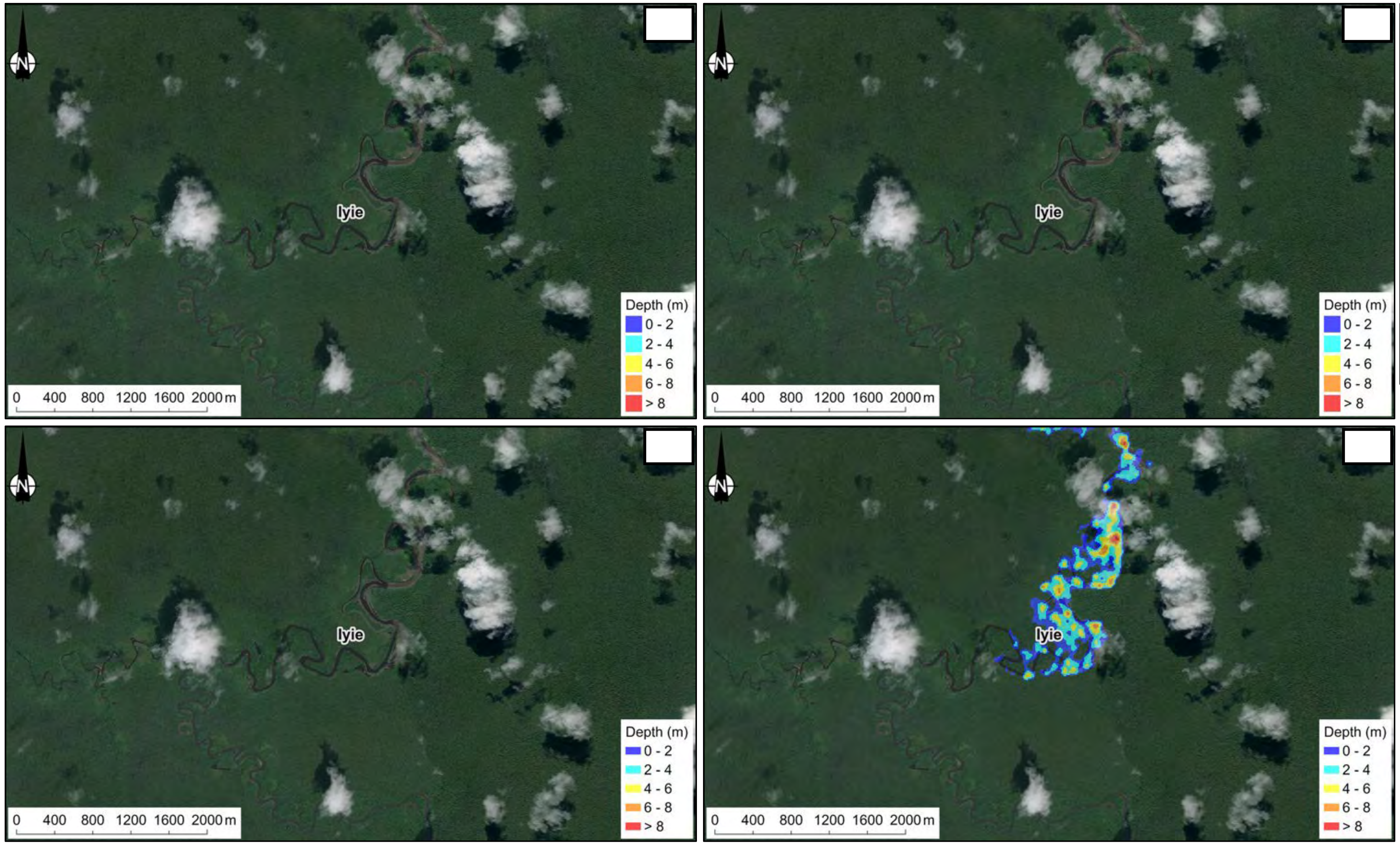



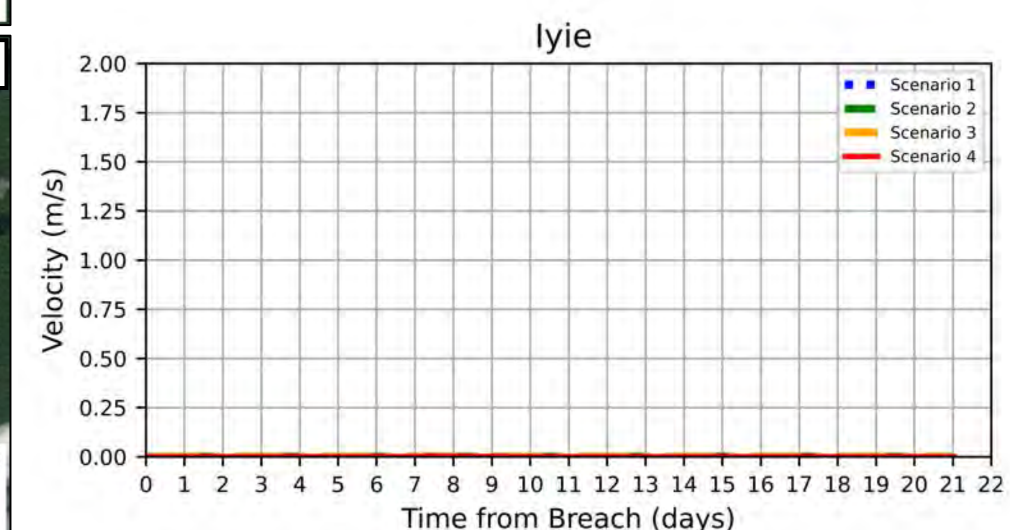


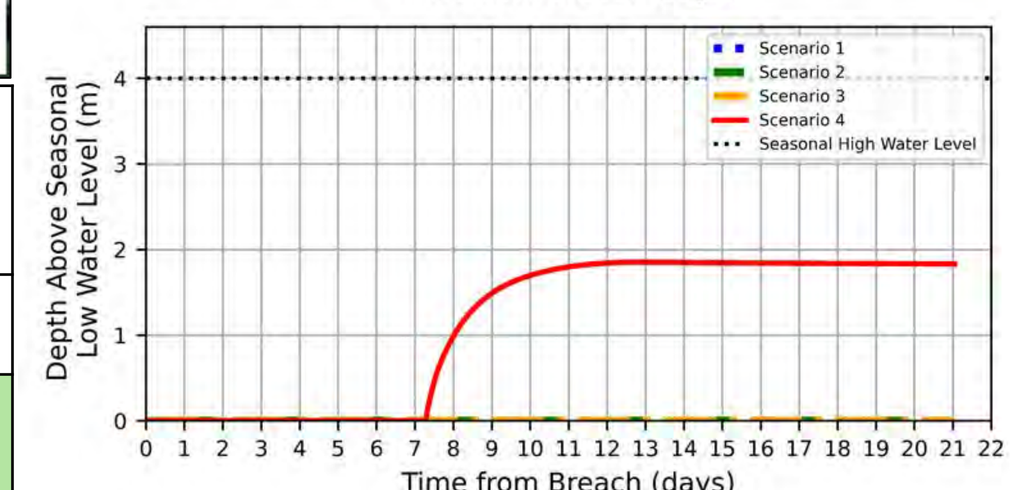


| **Scenario** | **(1)<br>2m Breach – Seasonal Low Initial Water Level** | **(2)<br>10m Breach - Seasonal Low Initial Water Level** | **(3)<br>10m Breach – Seasonal High Initial Water Level** | **(4)<br>66.7m Breach (Max) – Seasonal Low Initial Water Level** |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | **0.26 $Bm^3$** | **1.24 $Bm^3$** | **1.24 $Bm^3$** | **6.37 $Bm^3$** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **Not Inundated** | **Not Inundated** | **1.85, [-], 0.02** |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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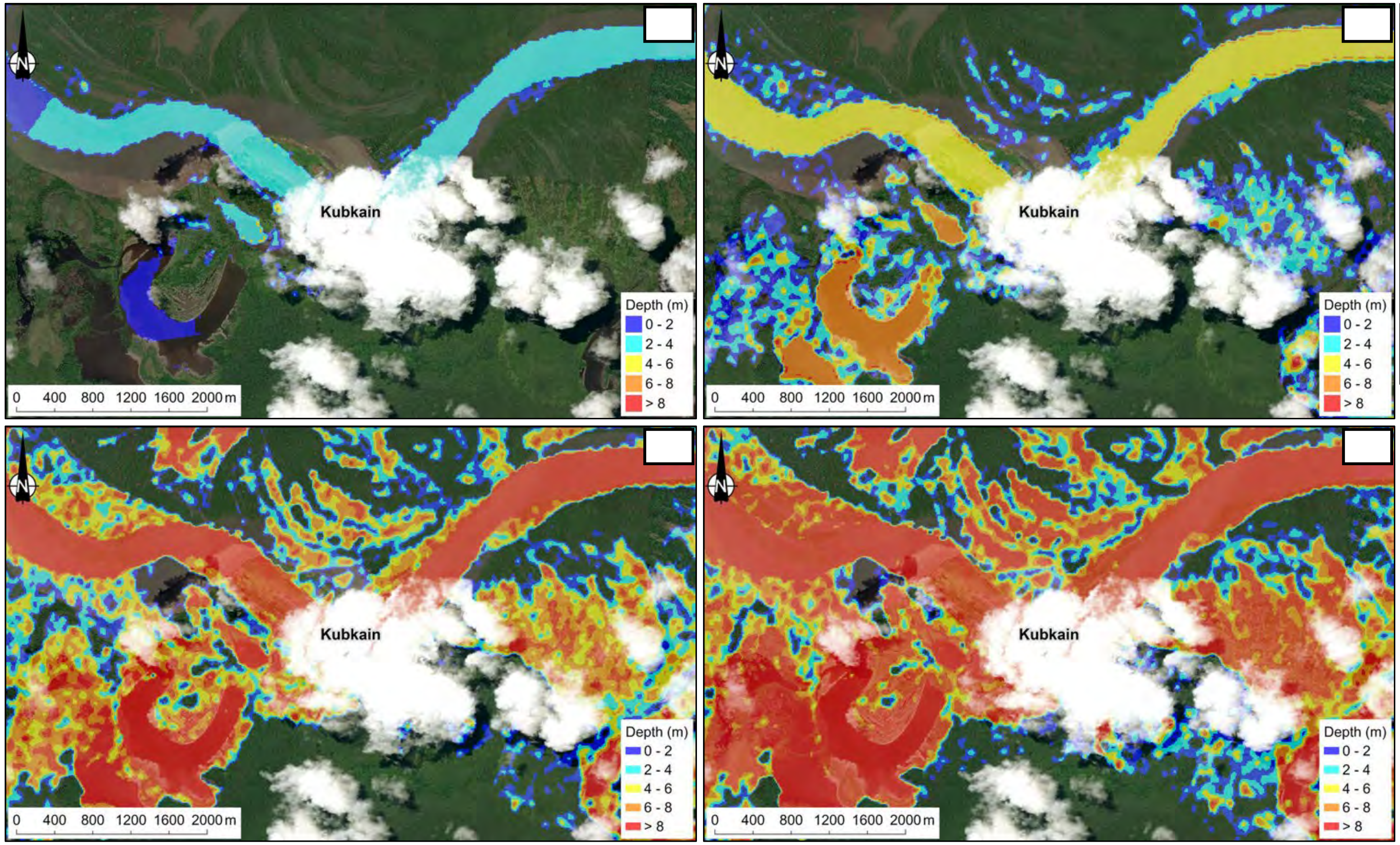
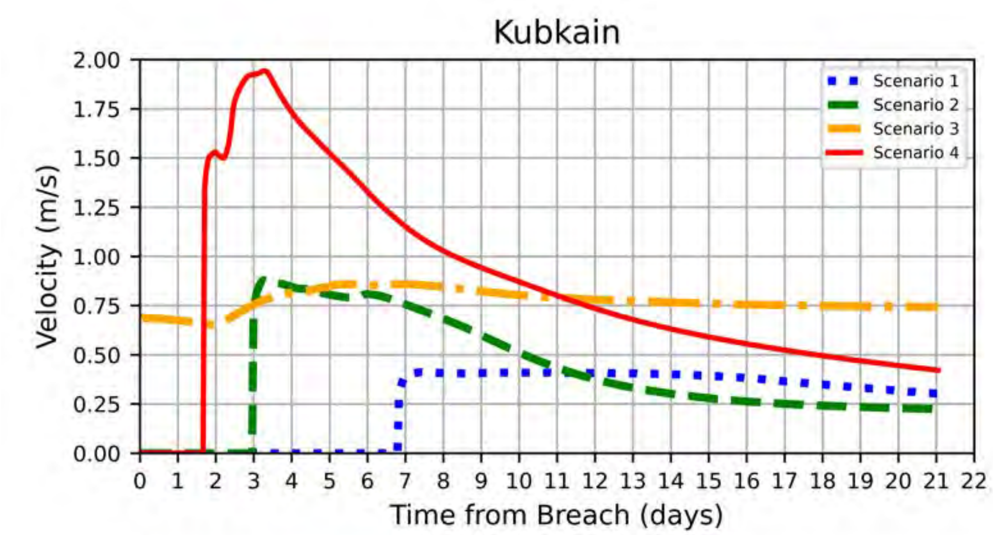
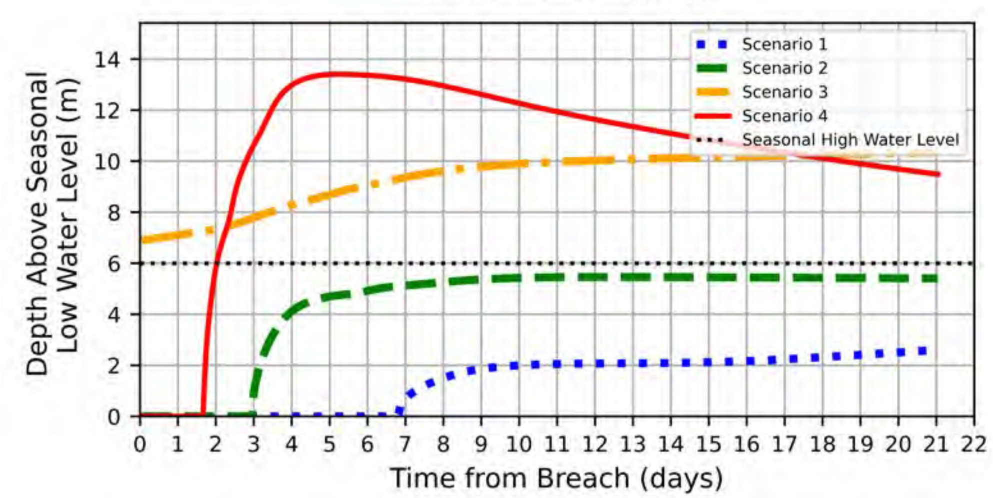
| Job No: CAPR003035<br>Filename: FriedaRiver_VillagebyVillageAssessment.pptx | Frieda River Dam | Date: May 2024 | Approved: ABL | Figure: **X.X** |








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 2.6, [-], 0.45 | 5.46, [-], 0.93 | 10.33, [4.33], 0.87 | 13.41, [7.41], 1.99 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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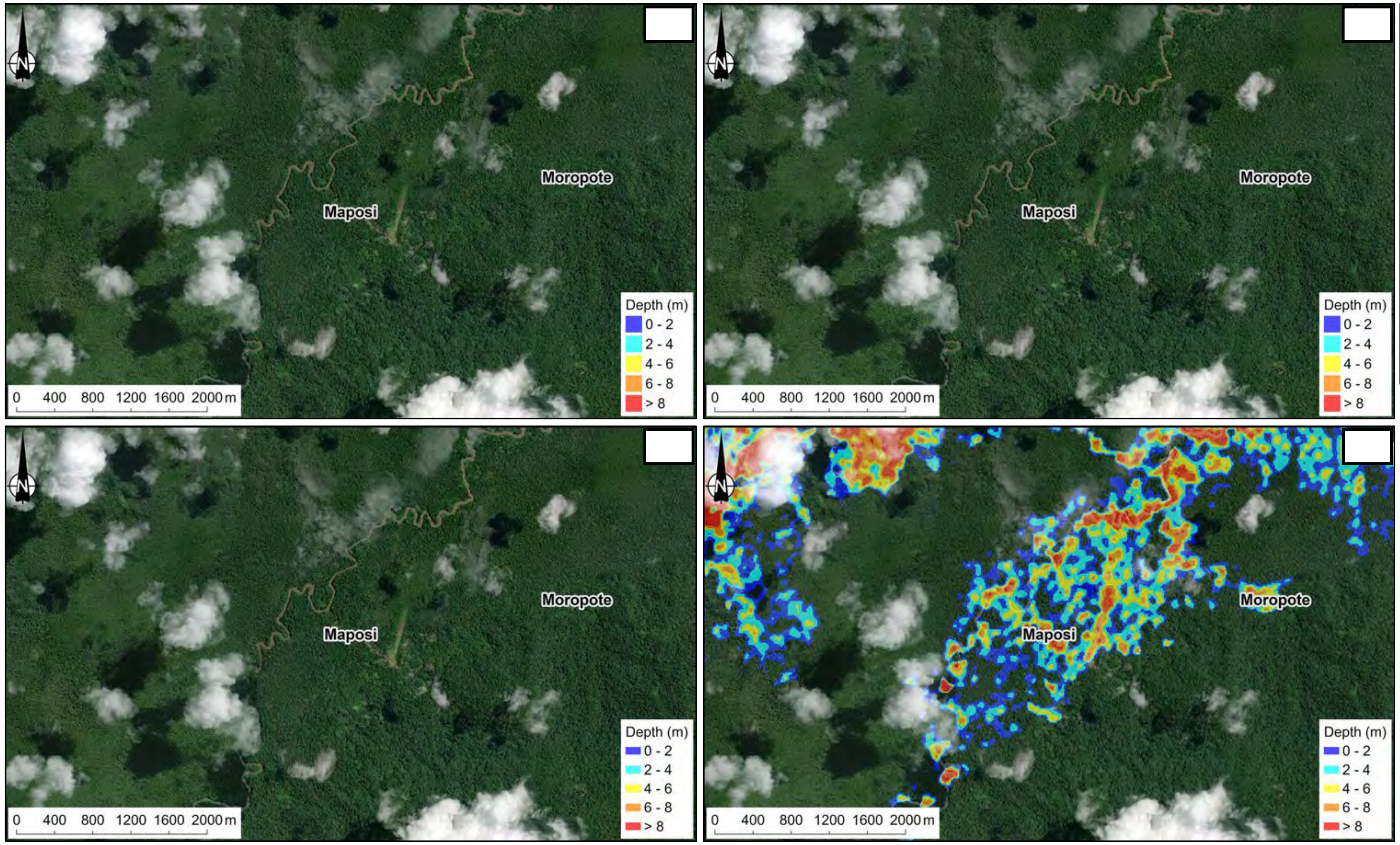



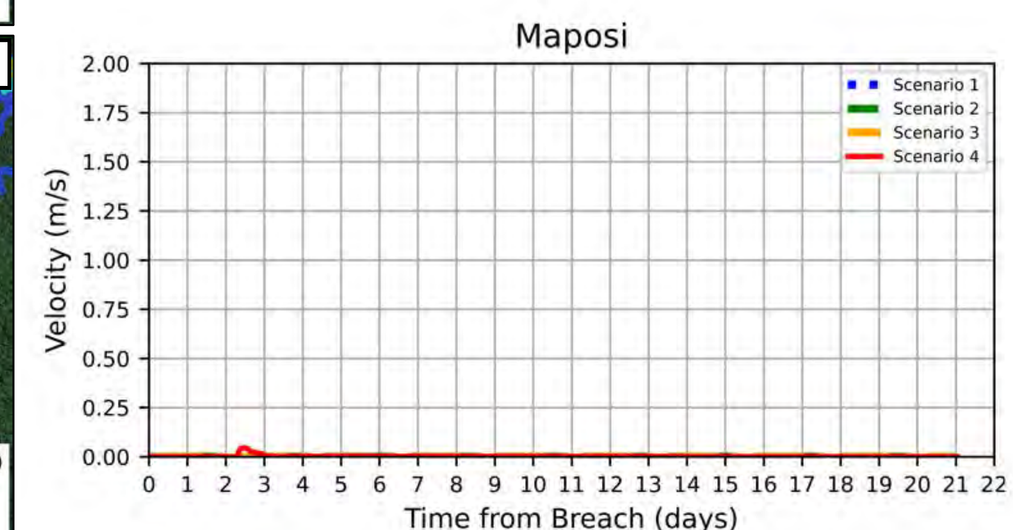


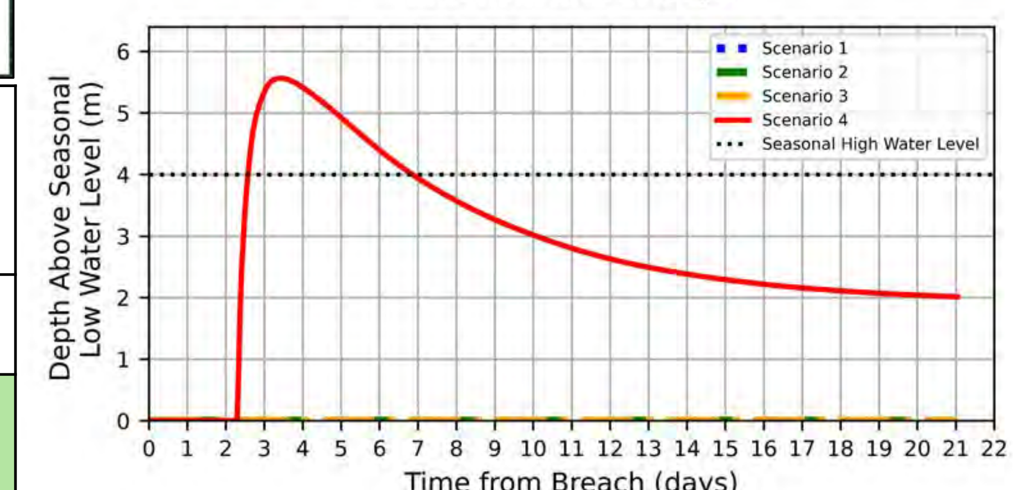


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | Not Inundated | 5.56, [1.56], 0.12 |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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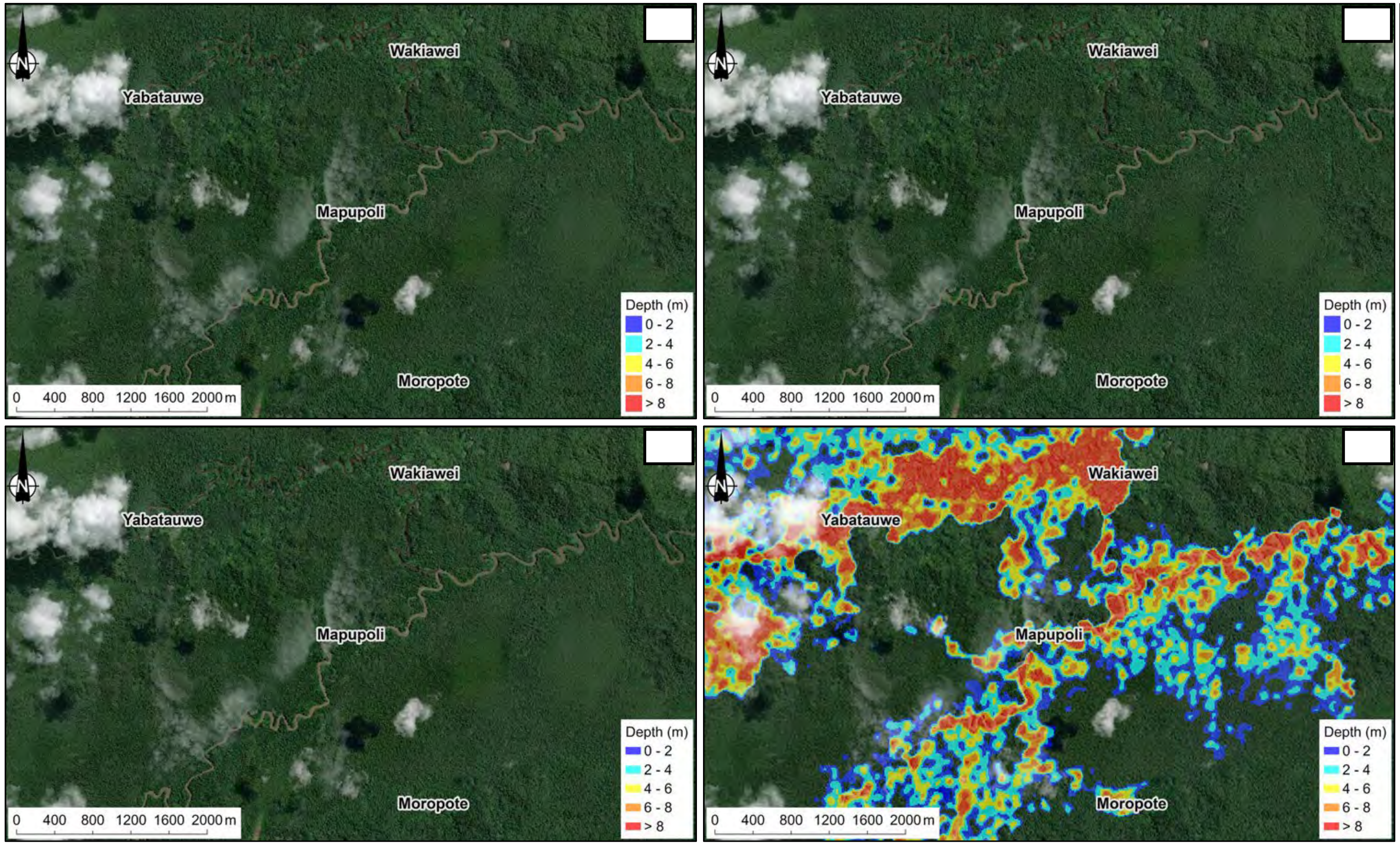
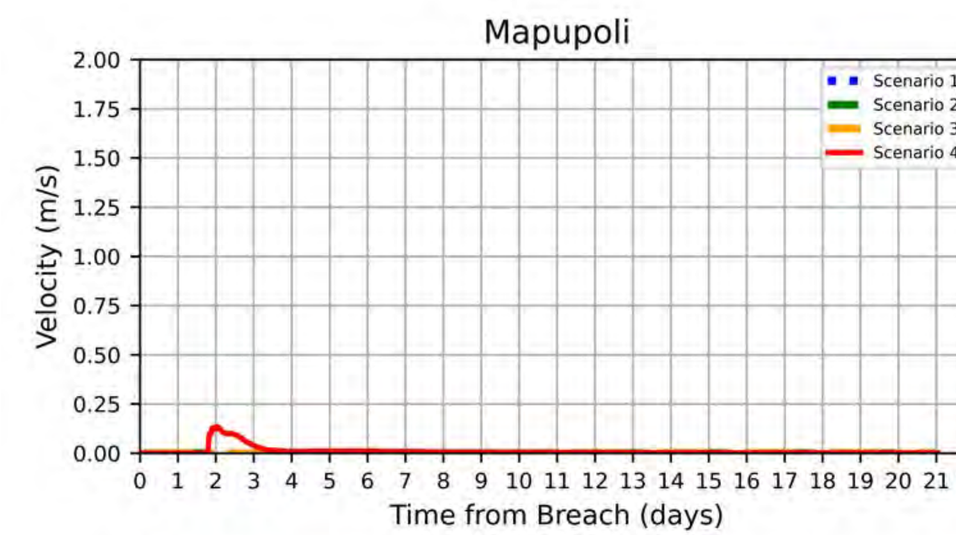
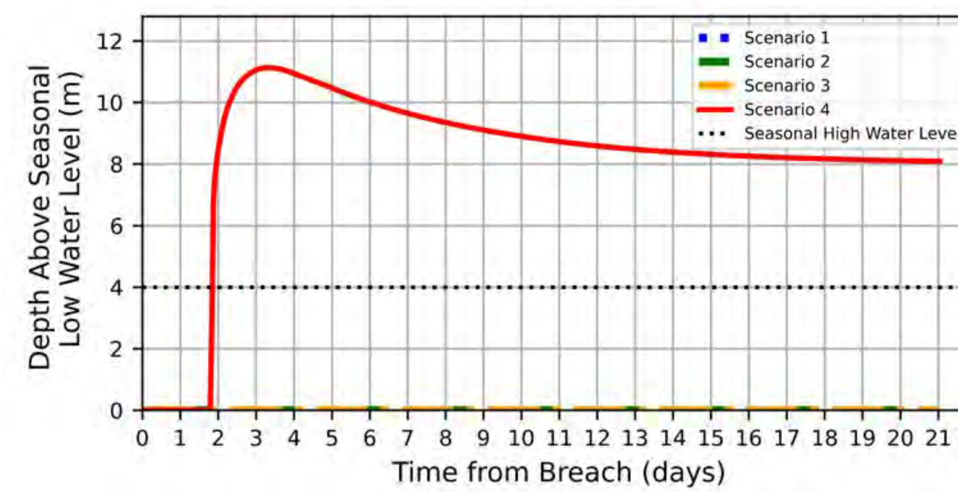
| Job No: CAPR003035<br>Filename: FriedaRiver_VillagebyVillageAssessment.pptx | Frieda River Dam | Date: May 2024 / Approved: ABL / Figure: X.X |








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume (m³) | 0.26 Bm³ | 1.24 Bm³ | 1.24 Bm³ | 6.37 Bm³ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | Not Inundated | 11.29, [7.29], 0.66 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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Filename: FriedaRiver_VillagebyVillageAssessment.pptx

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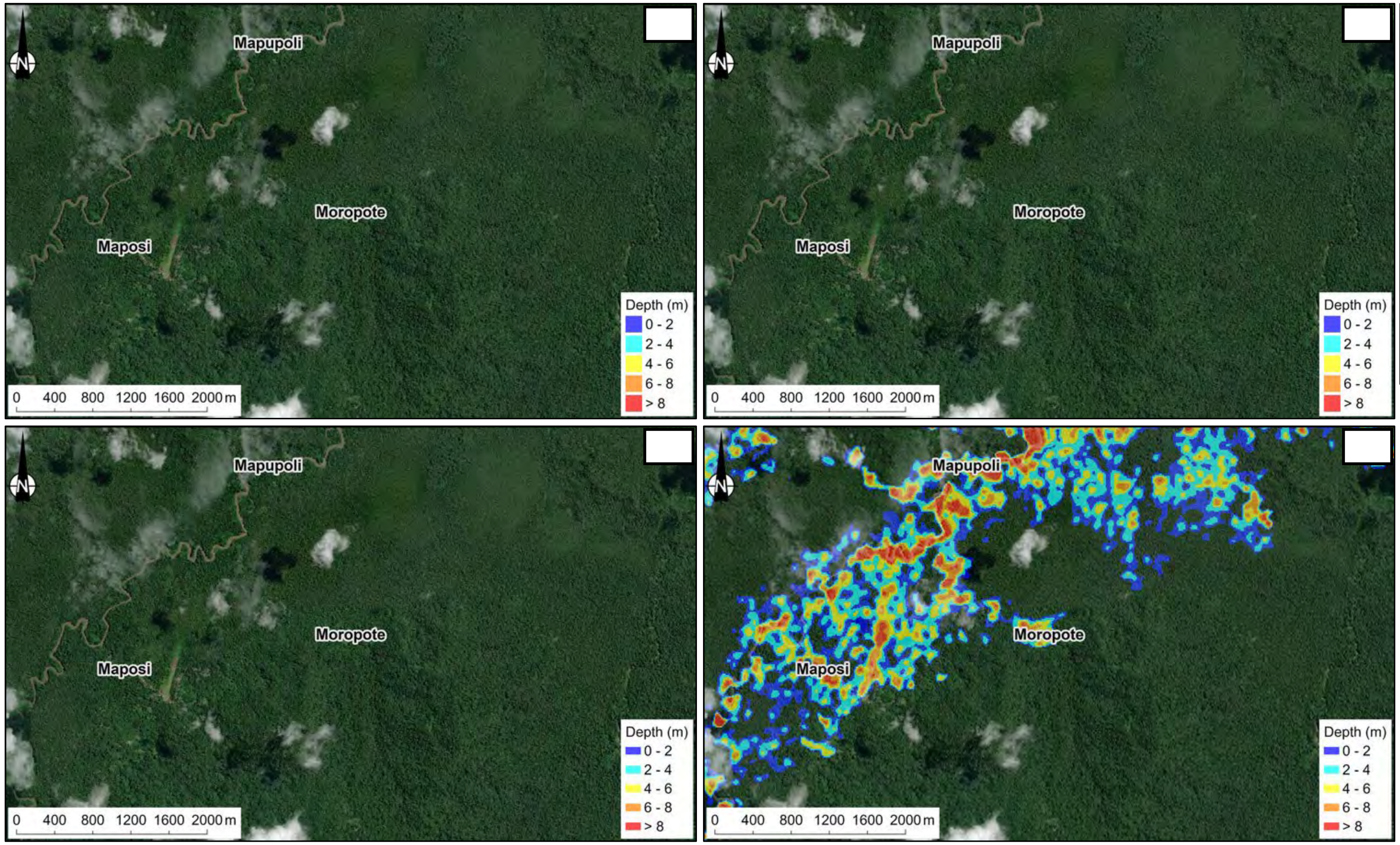
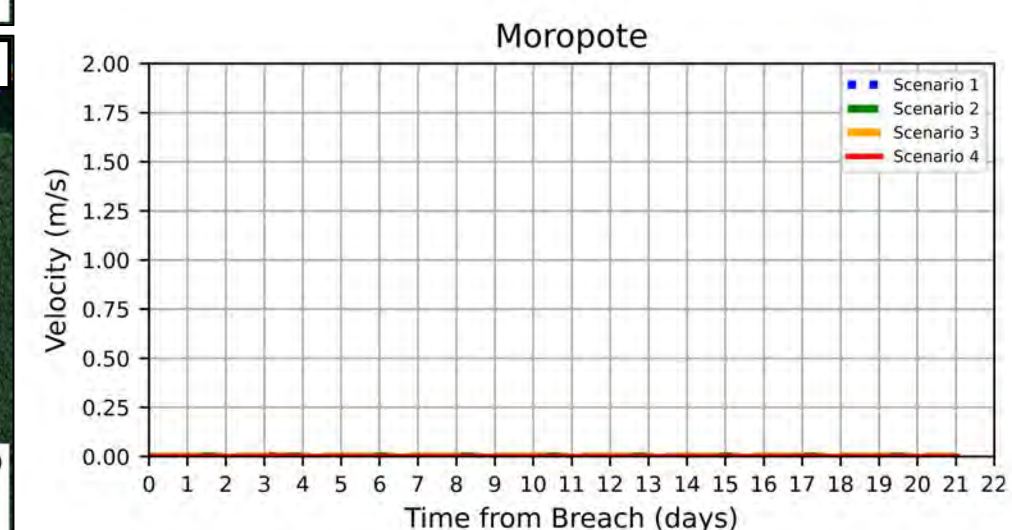
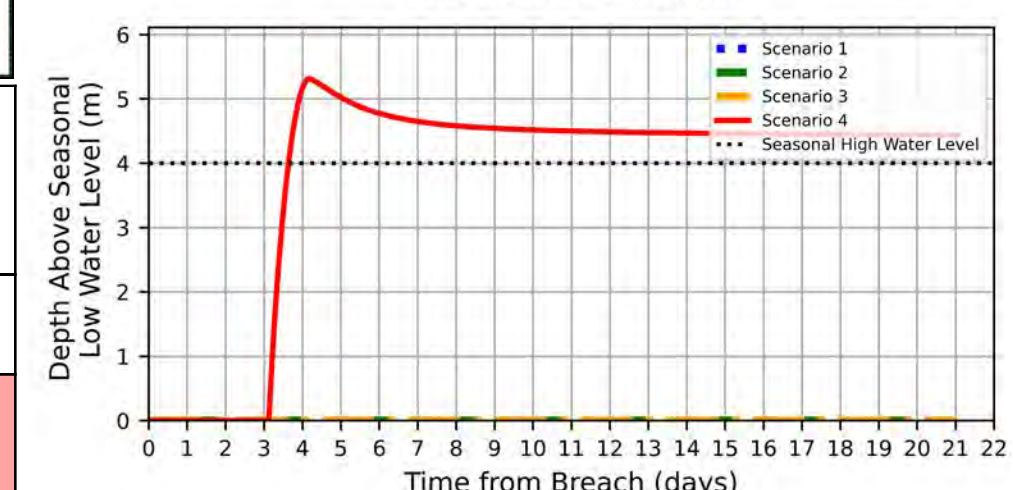
Date: May 2024 | Approved: ABL | Figure: X.X








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | **0.26 $Bm^3$** | **1.24 $Bm^3$** | **1.24 $Bm^3$** | **6.37 $Bm^3$** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **Not Inundated** | **Not Inundated** | **5.32, [-], 0.03** |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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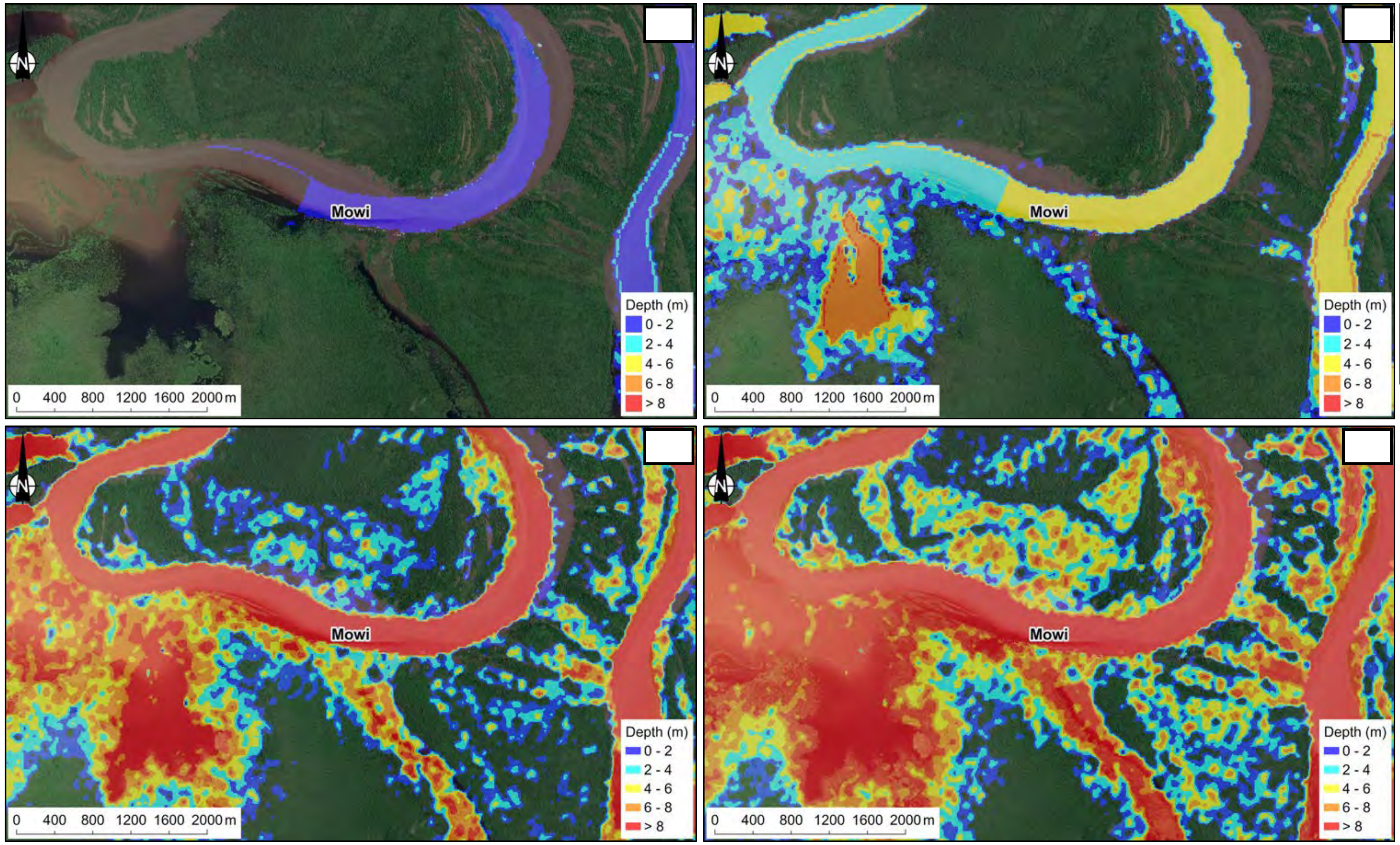
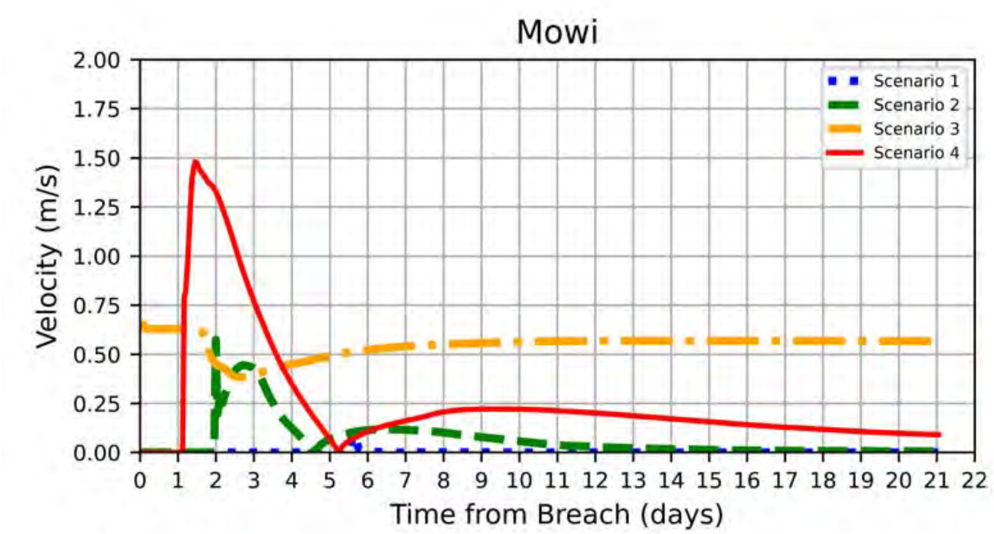
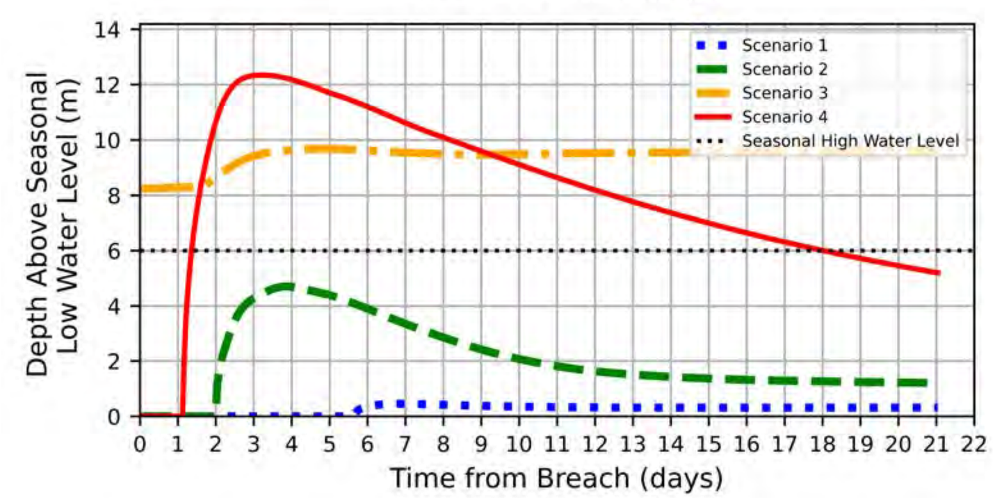
| Job No: CAPR003035<br>Filename: FriedaRiver_VillagebyVillageAssessment.pptx | Frieda River Dam | **Moropote**<br>Date: May 2024 · Approved: ABL · Figure: X.X |








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 0.45, [-], 0.33 | 4.7, [-], 0.93 | 9.68, [3.68], 0.66 | 12.35, [6.35], 1.49 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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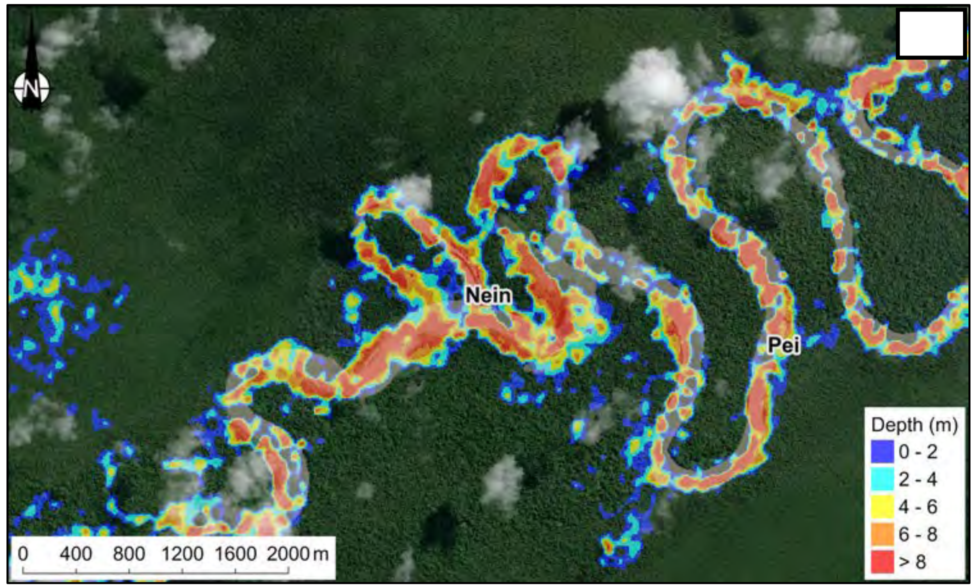
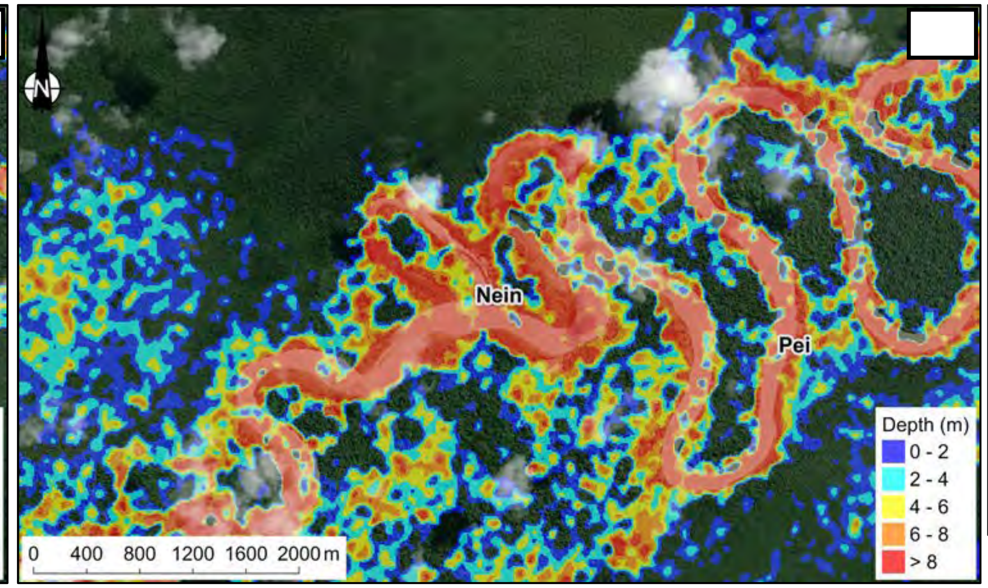
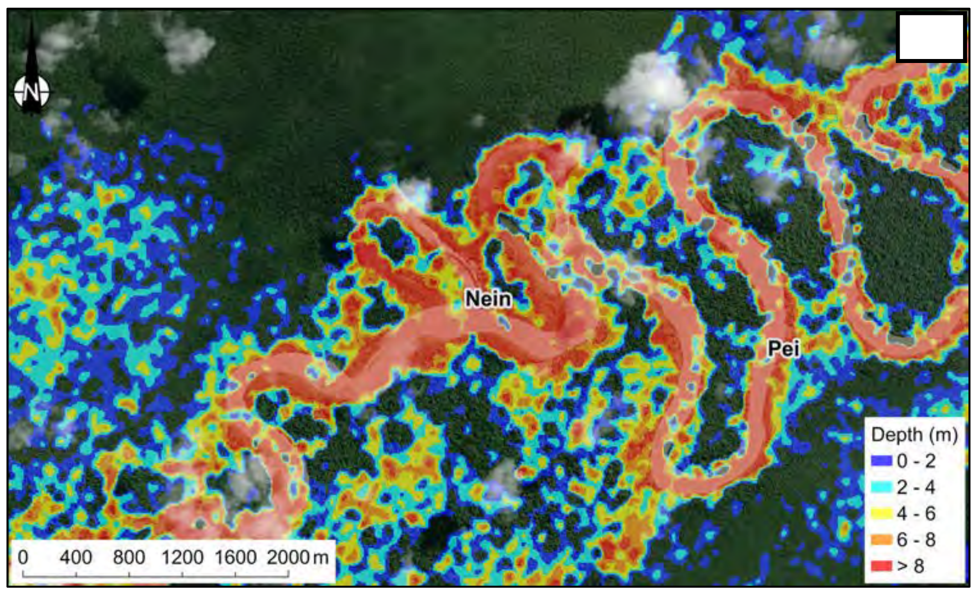
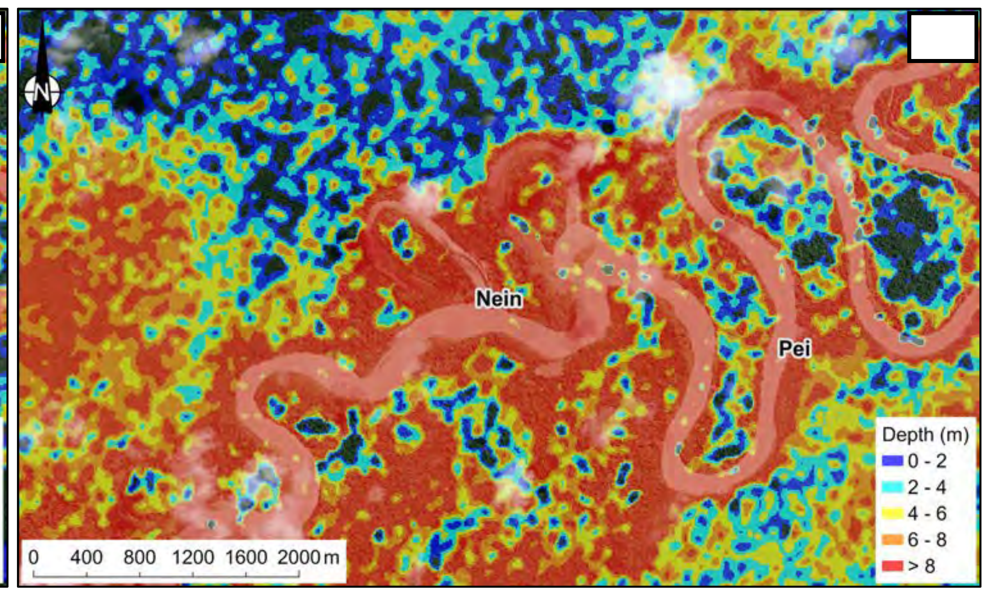
| Job No: CAPR003035<br>Filename: FriedaRiver_VillagebyVillageAssessment.pptx | Frieda River Dam | Date: May 2024 / Approved: ABL / Figure: X.X |

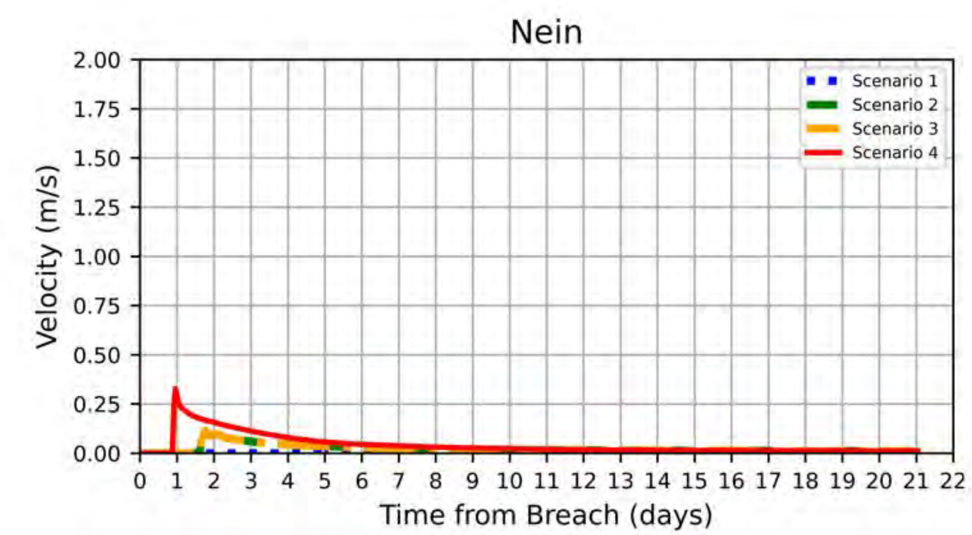


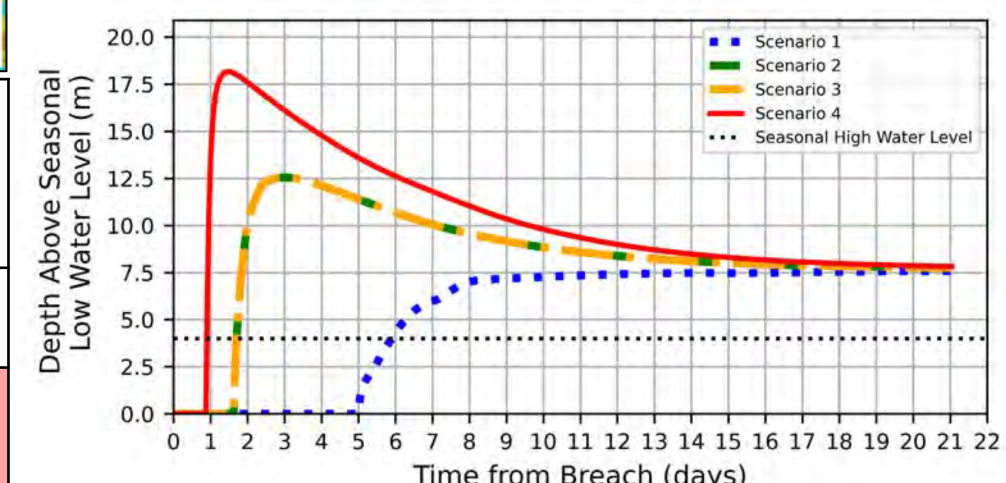








| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 7.58, [3.58], 0.1 | 12.55, [8.55], 0.23 | 12.55, [8.55], 0.22 | 18.16, [14.16], 0.47 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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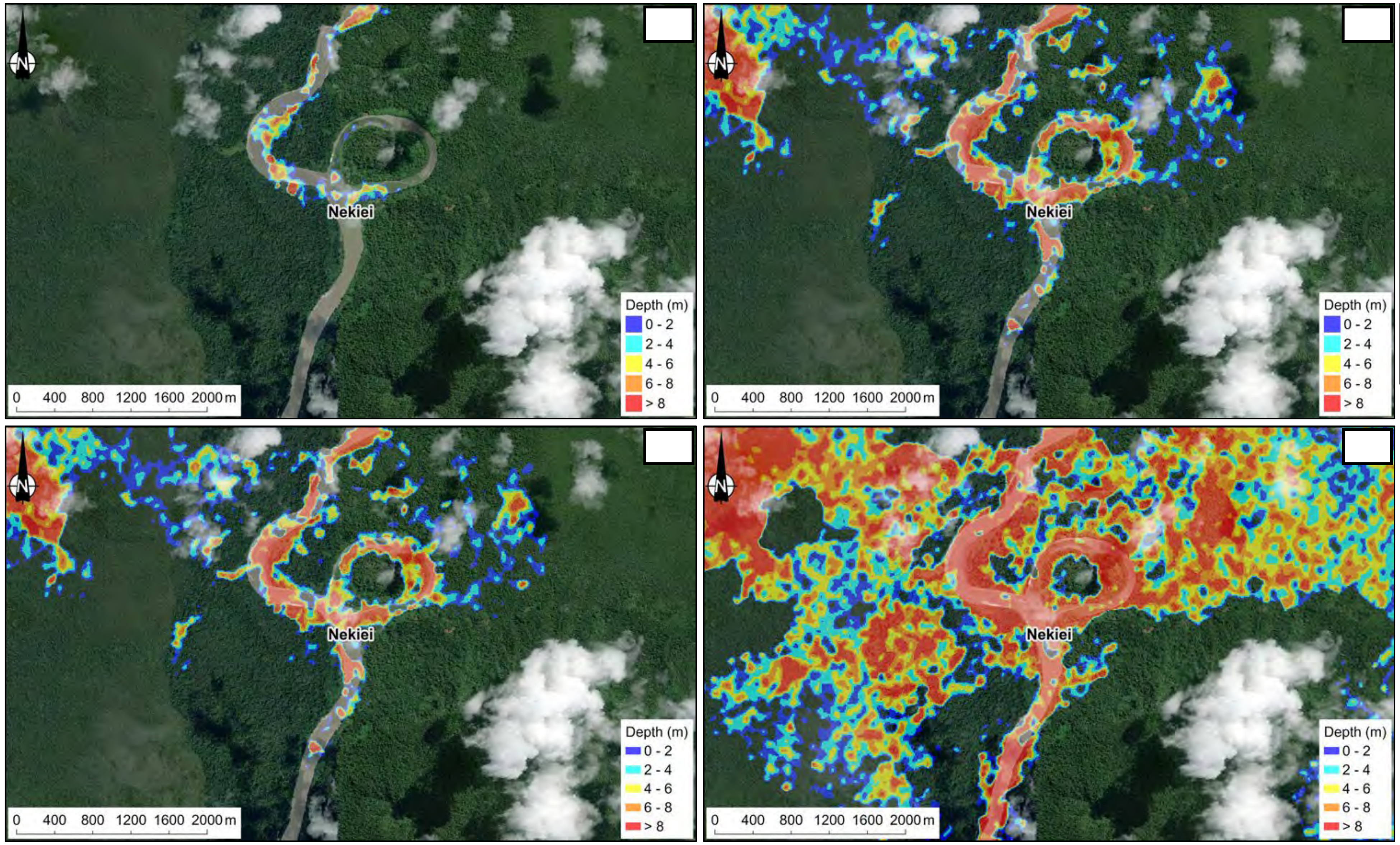
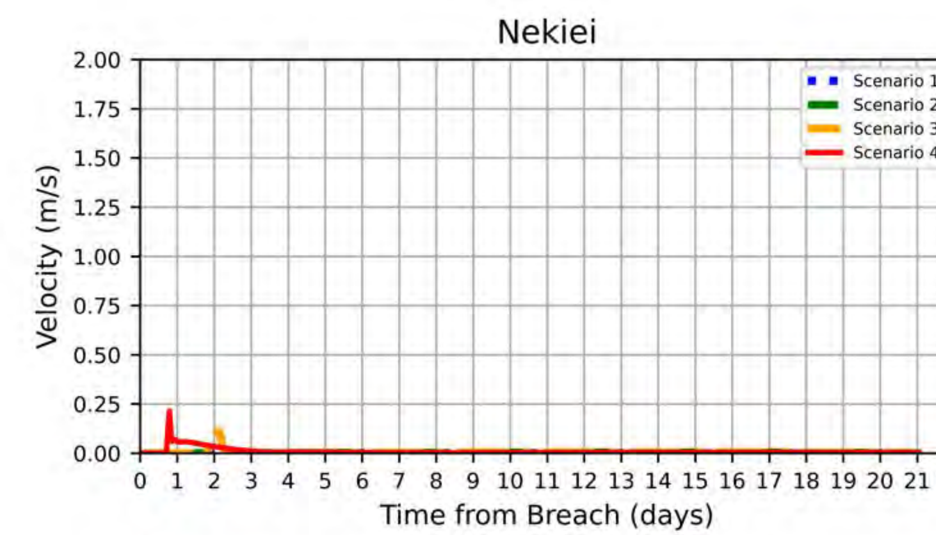
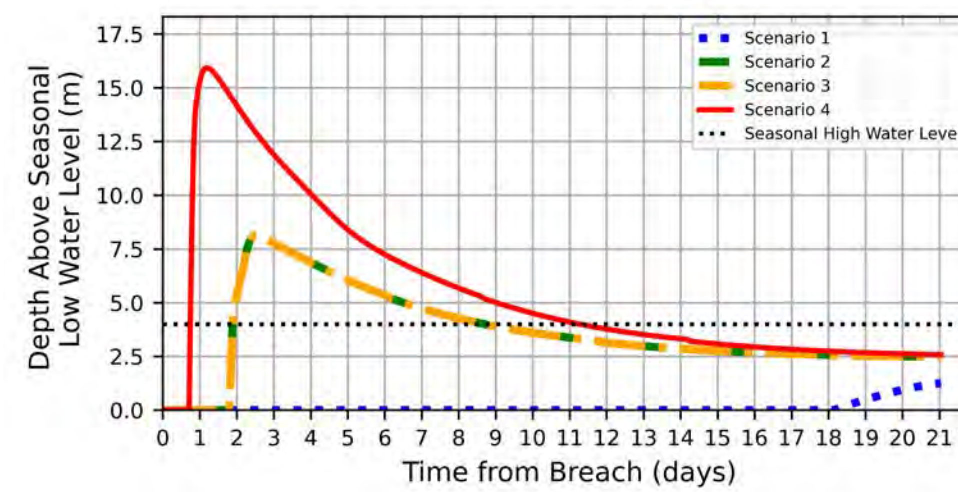
| Job No: CAPR003035<br>Filename: FriedaRiver_VillagebyVillageAssessment.pptx | Frieda River Dam | Date: May 2024 · Approved: ABL · Figure: X.X |








| Scenario | (1)<br>2m Breach – Seasonal Low Initial Water Level | (2)<br>10m Breach - Seasonal Low Initial Water Level | (3)<br>10m Breach – Seasonal High Initial Water Level | (4)<br>66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 1.26, [-], 0.07 | 8.14, [4.14], 0.17 | 8.14, [4.14], 0.17 | 15.93, [11.93], 0.36 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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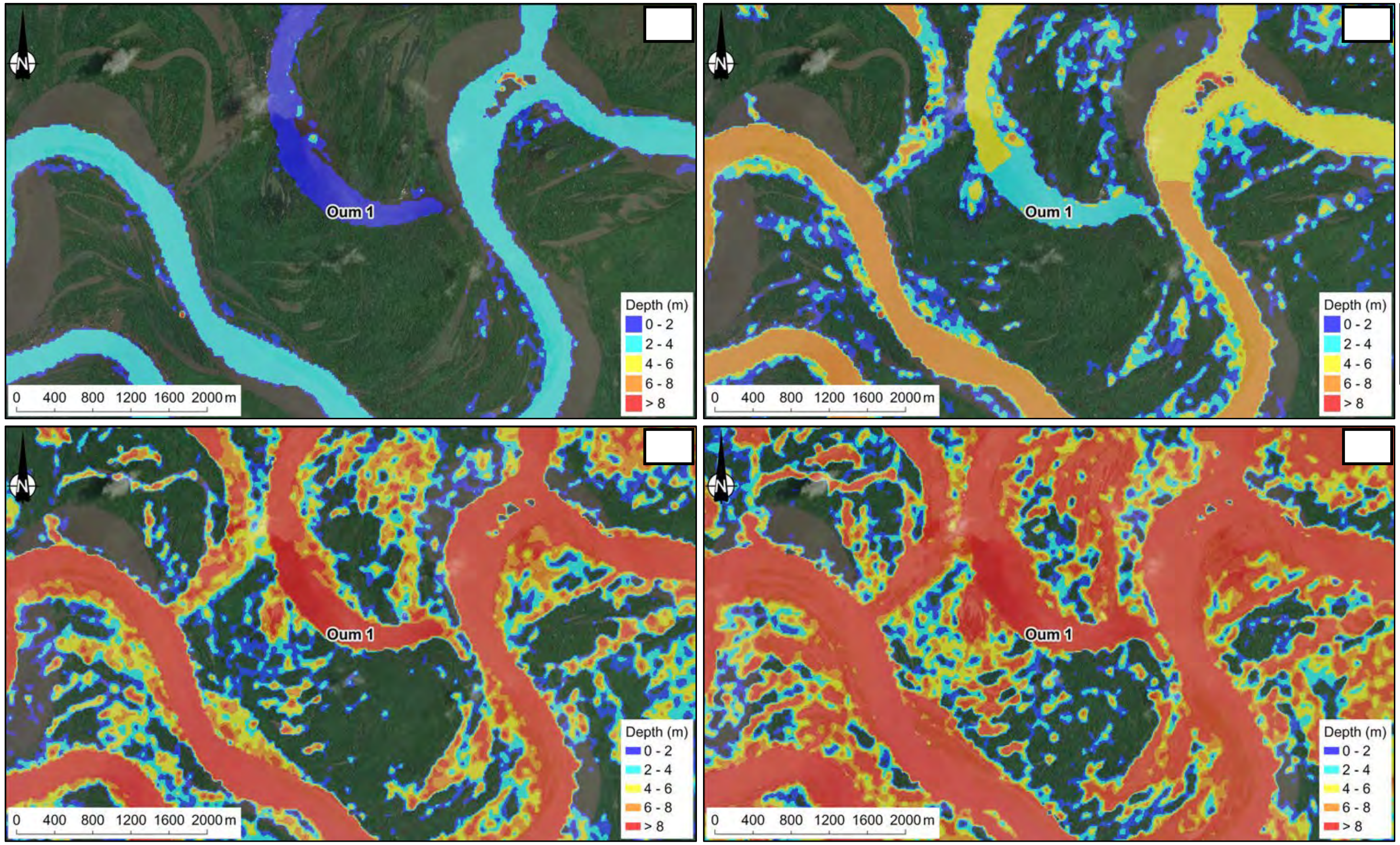



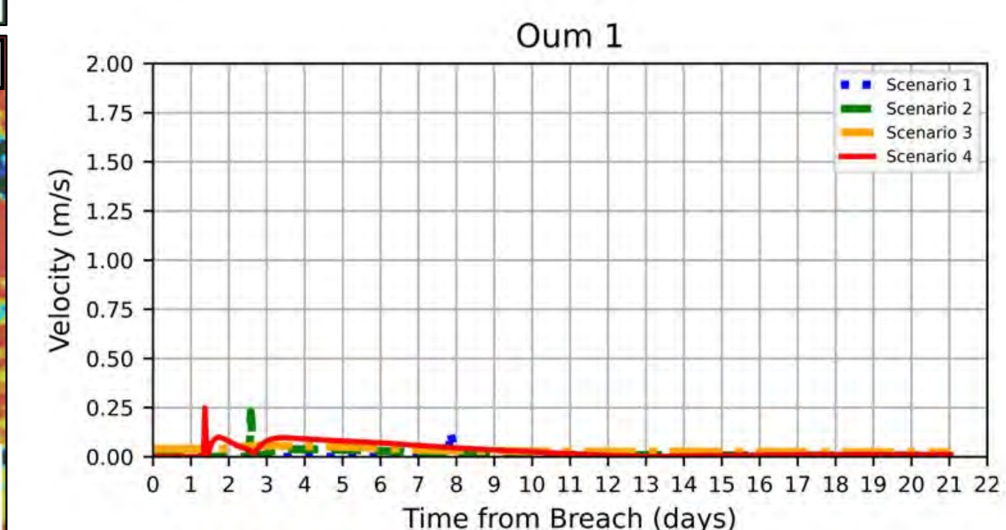


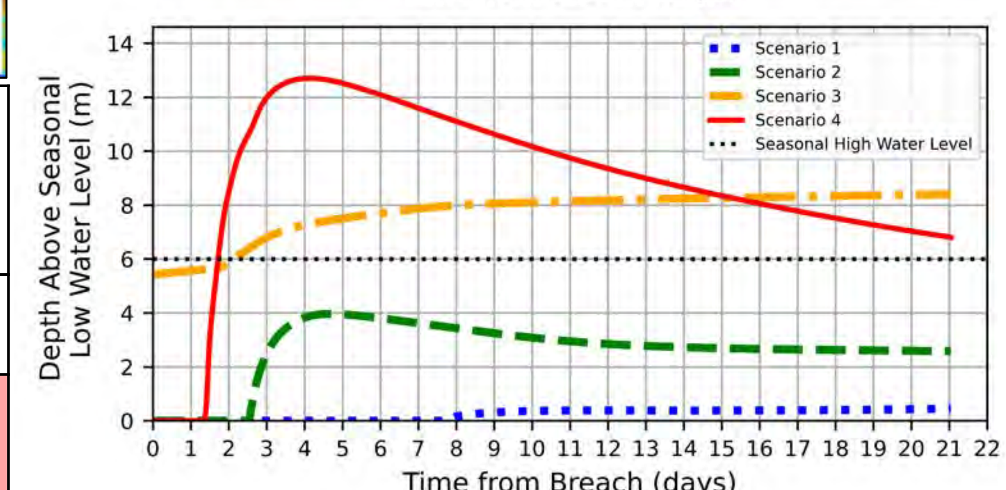


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 0.45, [-], 0.16 | 3.96, [-], 0.33 | 8.39, [2.39], 0.06 | 12.7, [6.7], 0.31 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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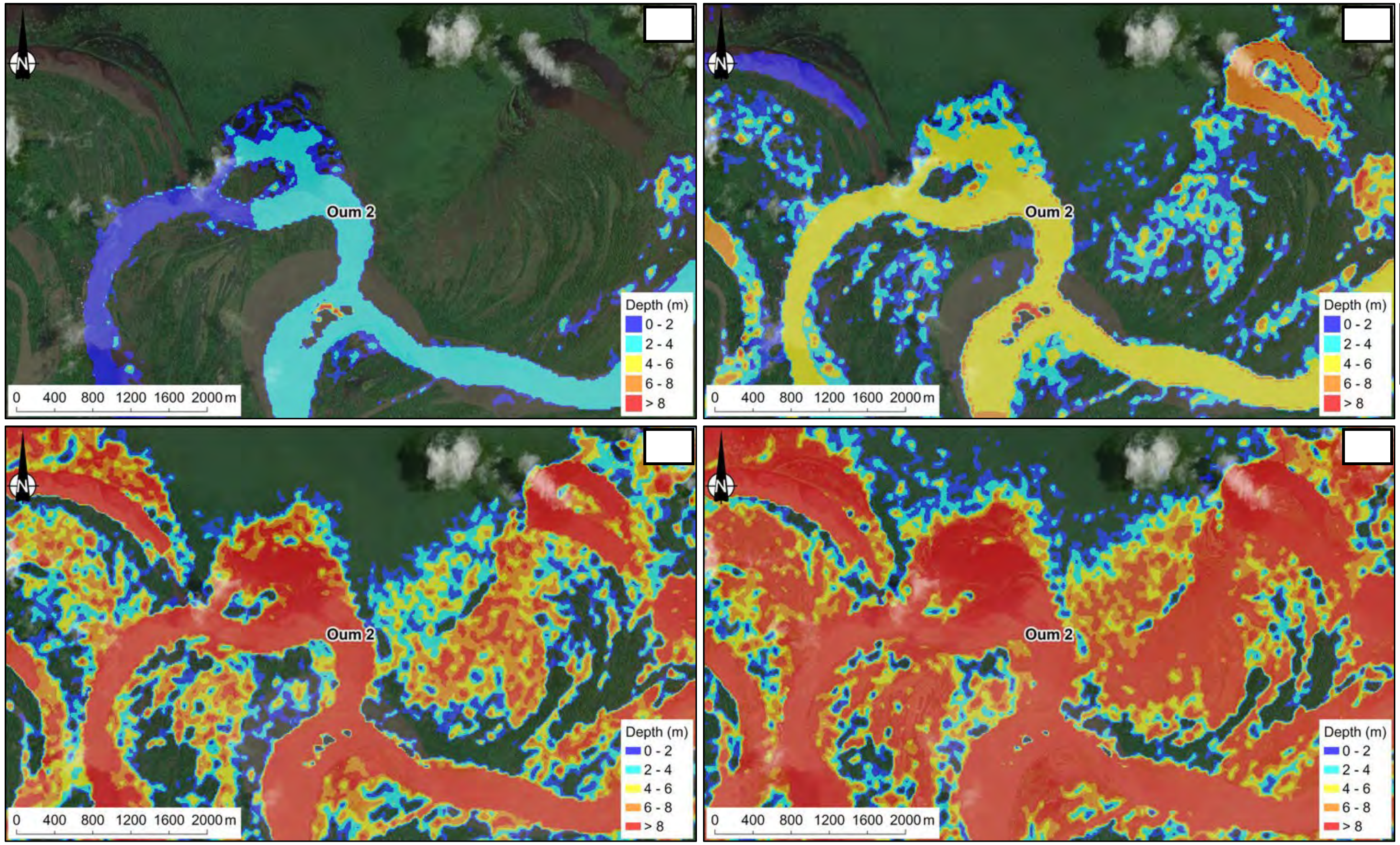



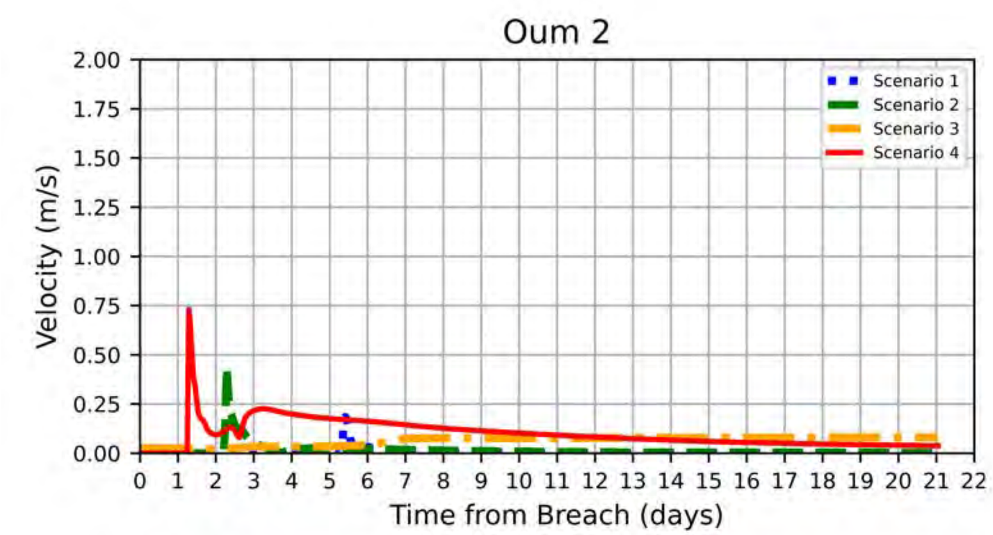


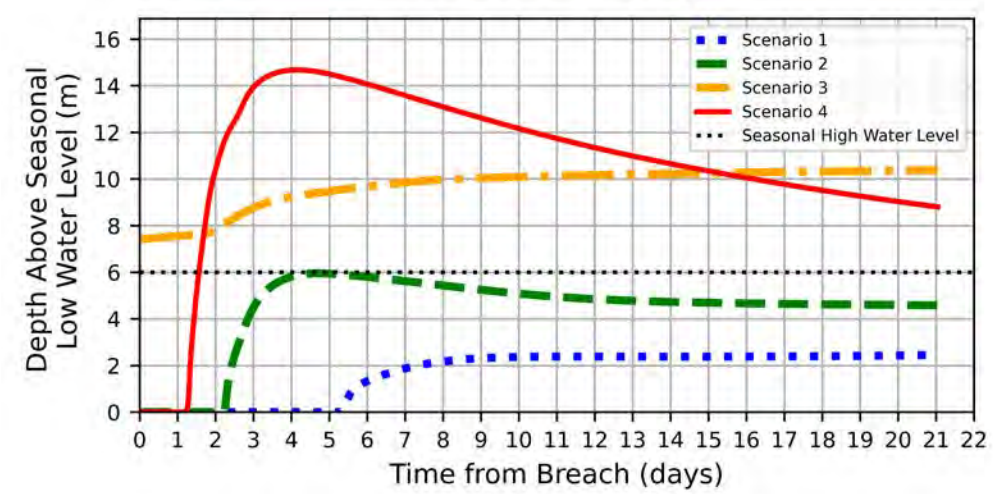


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 2.45, [-], 0.26 | 5.95, [-], 0.53 | 10.38, [4.38], 0.08 | 14.68, [8.68], 0.85 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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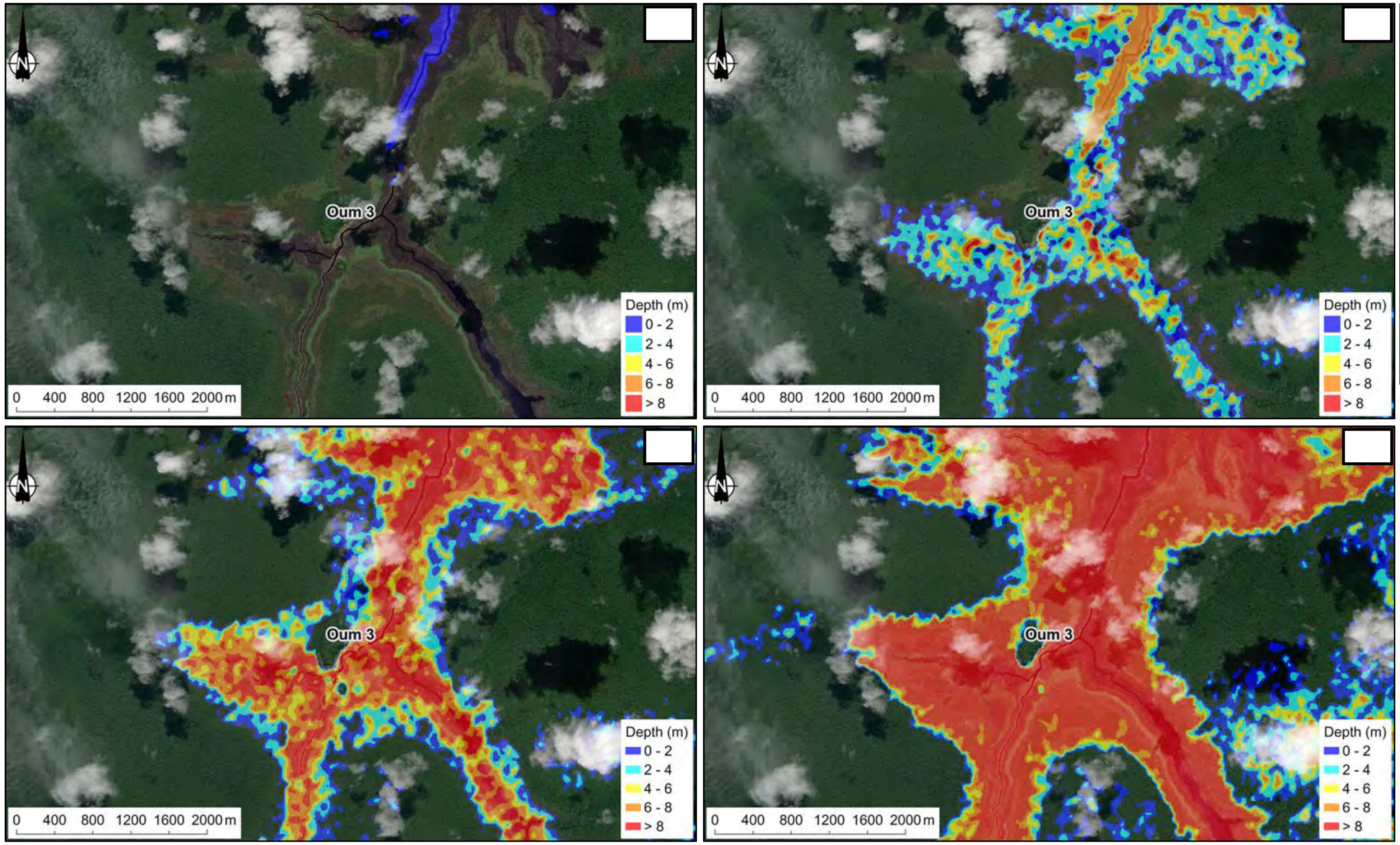



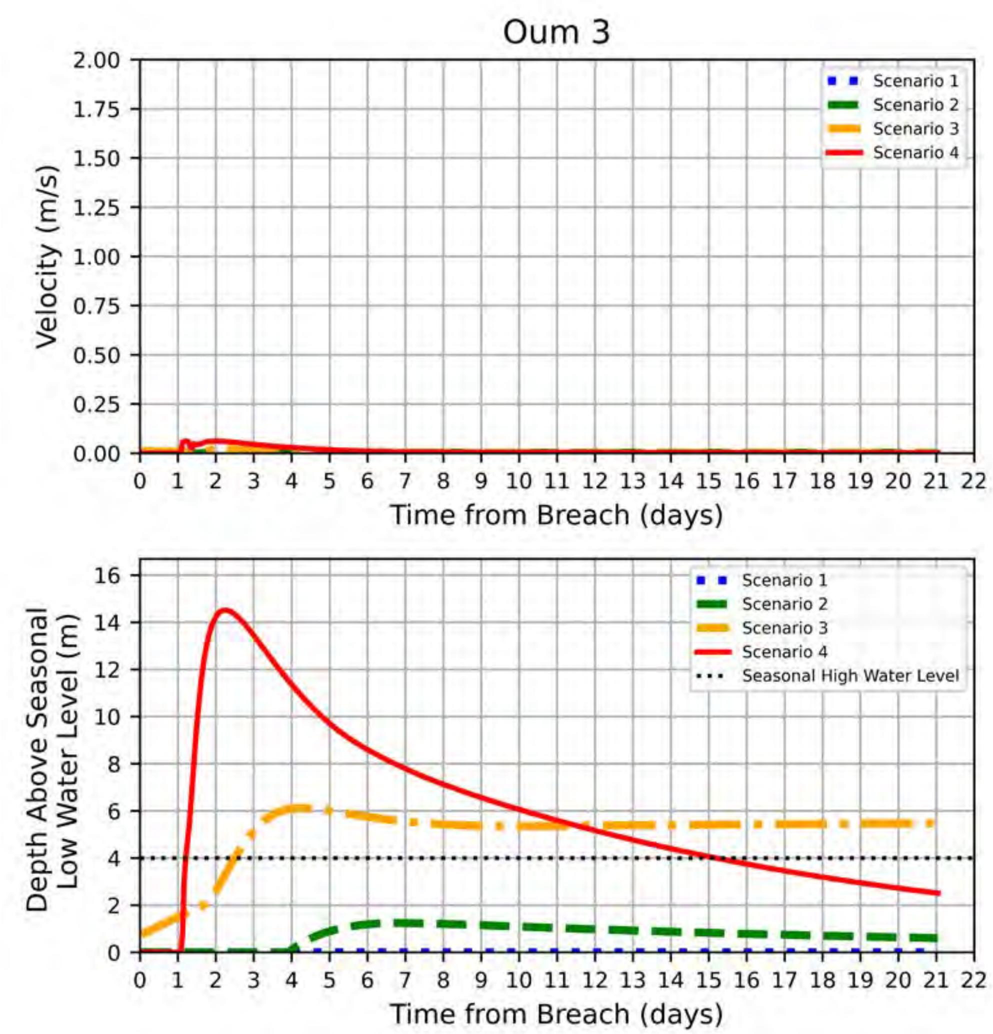


| Scenario | (1)<br>2m Breach – Seasonal Low Initial Water Level | (2)<br>10m Breach - Seasonal Low Initial Water Level | (3)<br>10m Breach – Seasonal High Initial Water Level | (4)<br>66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume (m³)** | **0.26 Bm³** | **1.24 Bm³** | **1.24 Bm³** | **6.37 Bm³** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **1.24, [-], 0.03** | **6.12, [2.12], 0.02** | **14.51, [10.51], 0.07** |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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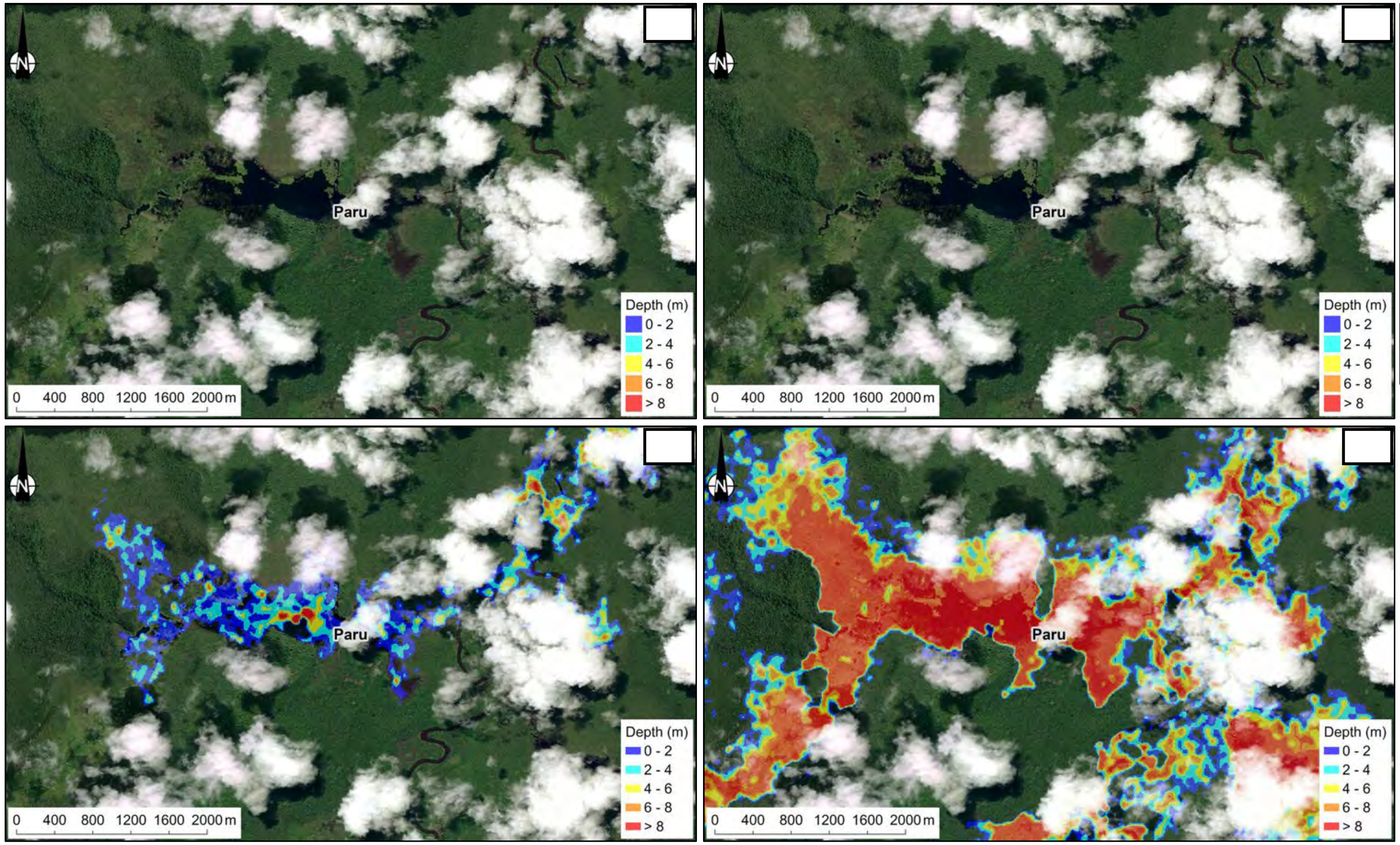



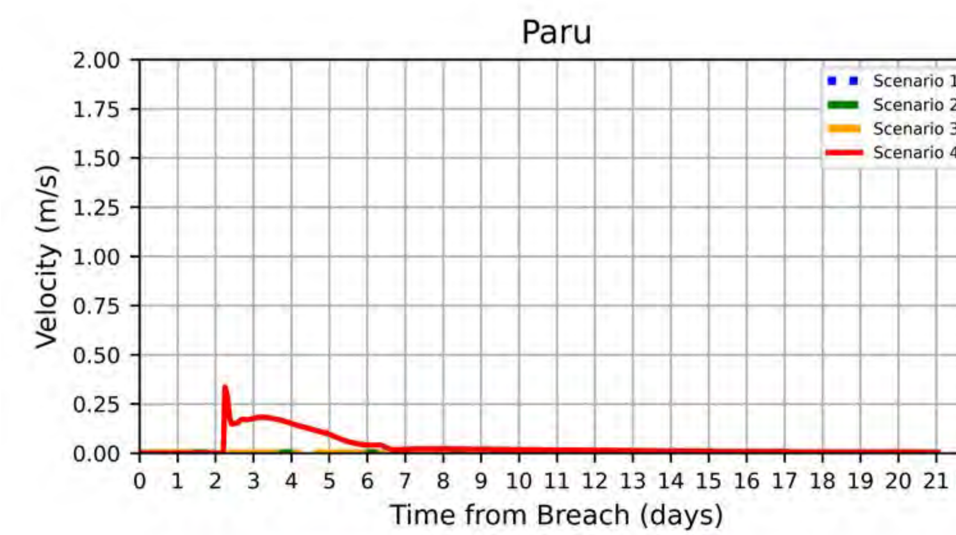


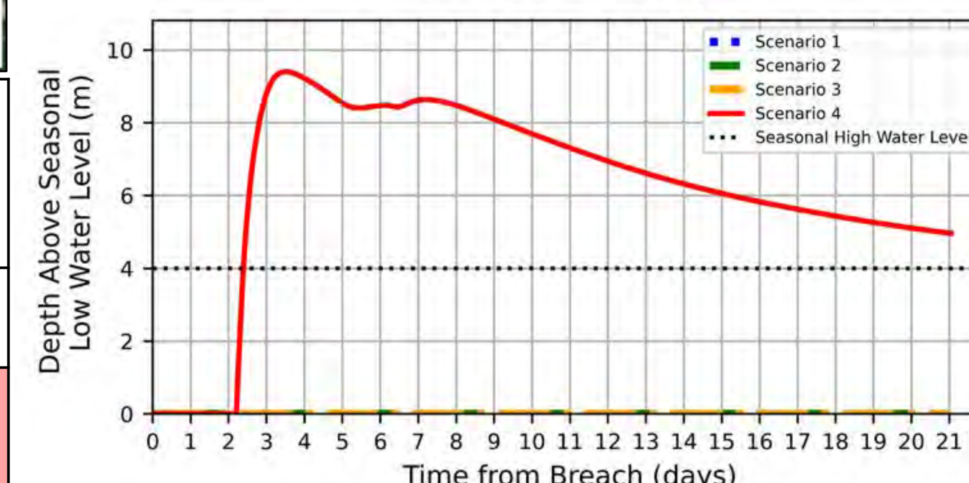


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | Not Inundated | 9.4, [5.4], 0.34 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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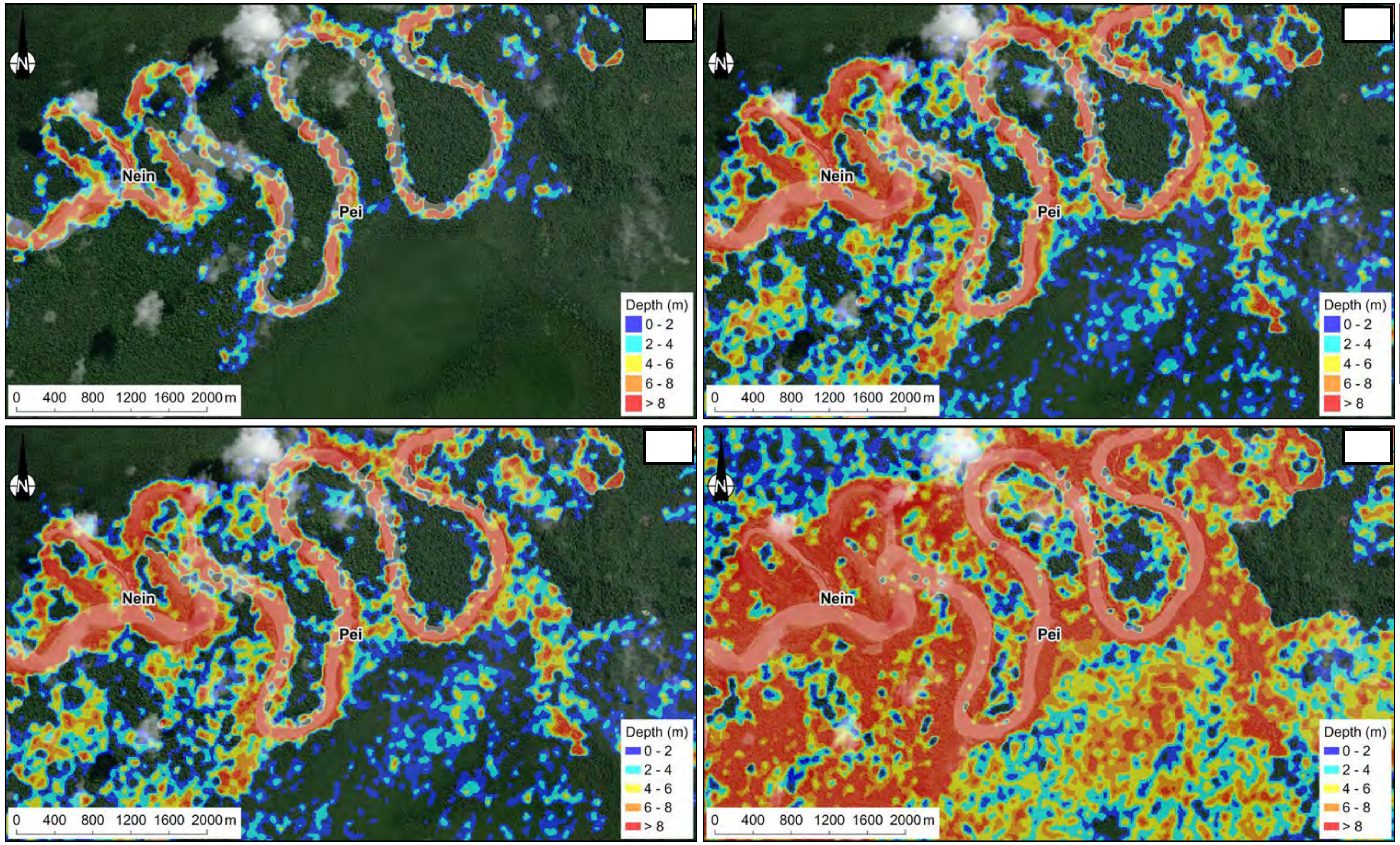



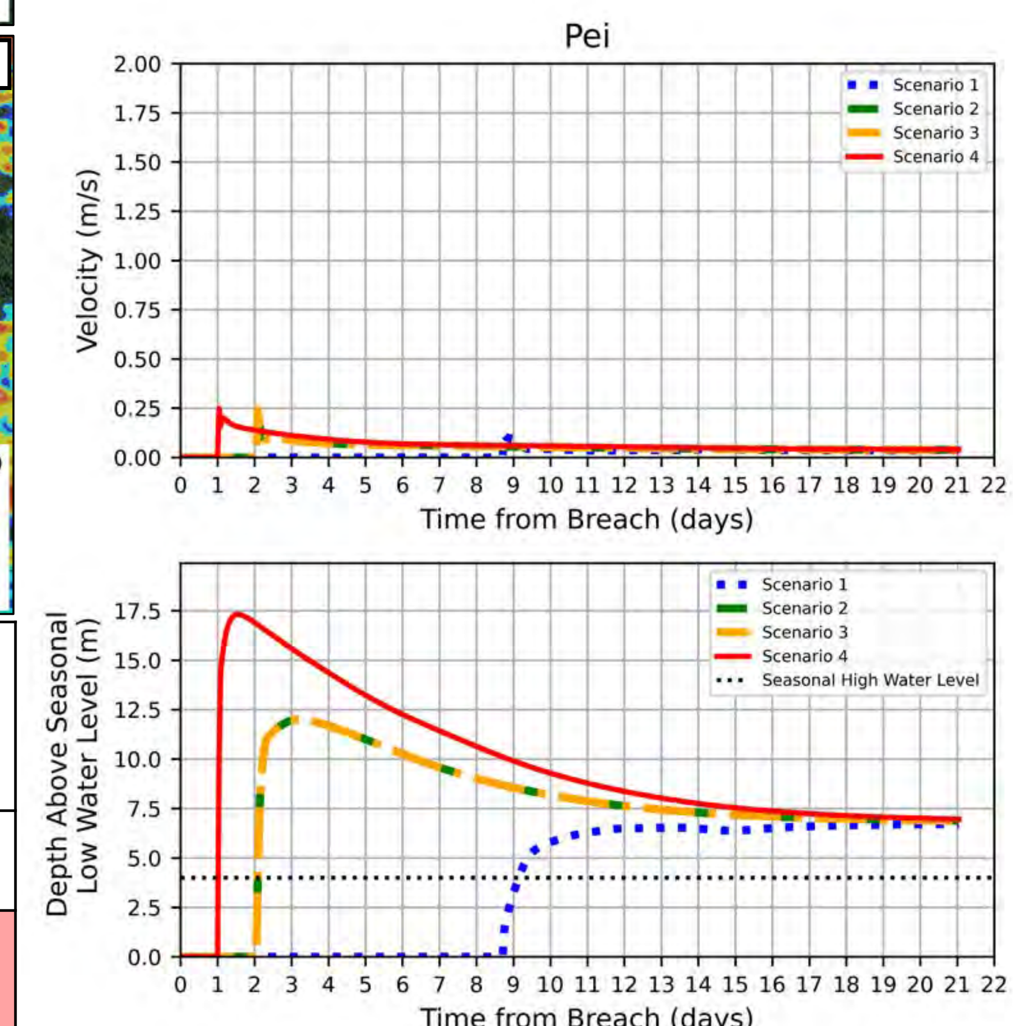


| Scenario | (1)<br>2m Breach – Seasonal Low Initial Water Level | (2)<br>10m Breach - Seasonal Low Initial Water Level | (3)<br>10m Breach – Seasonal High Initial Water Level | (4)<br>66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 6.71, [2.71], 0.21 | 11.99, [7.99], 0.3 | 11.99, [7.99], 0.3 | 17.31, [13.31], 0.51 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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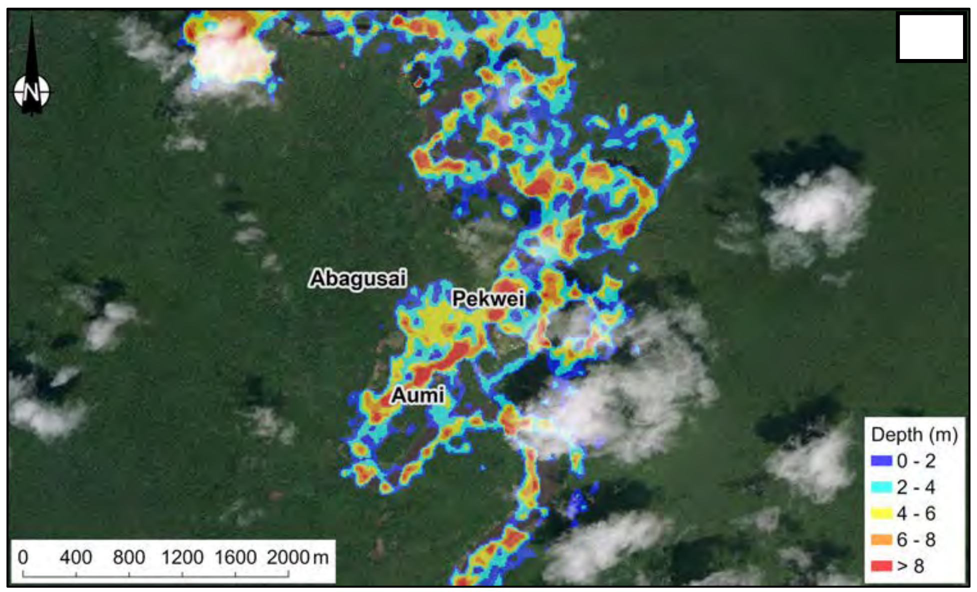


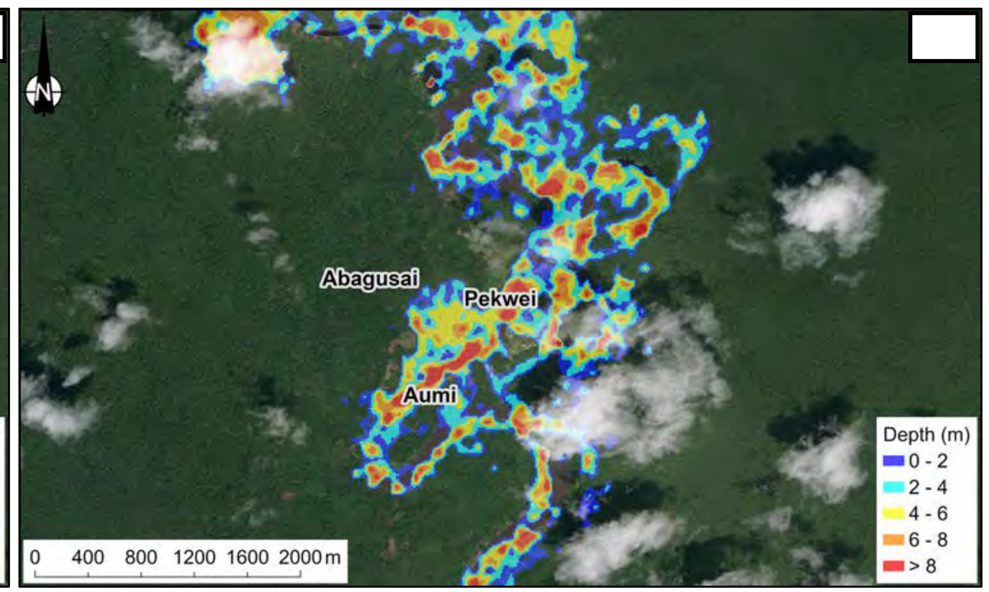


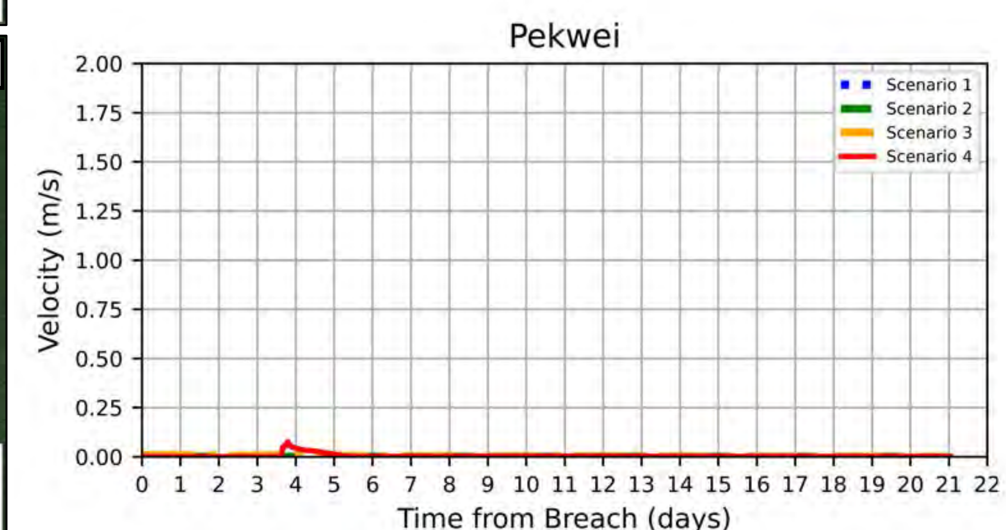


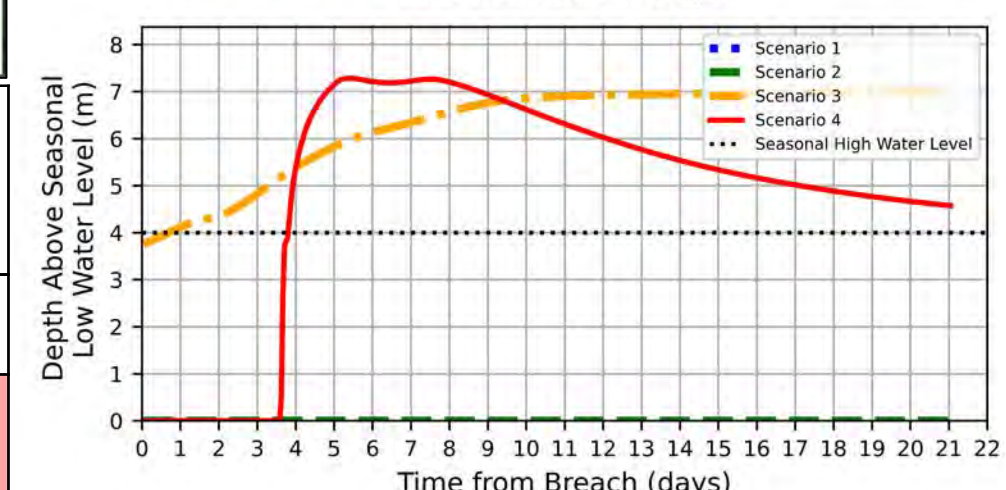


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | 7, [3], 0.01 | 7.28, [3.28], 0.14 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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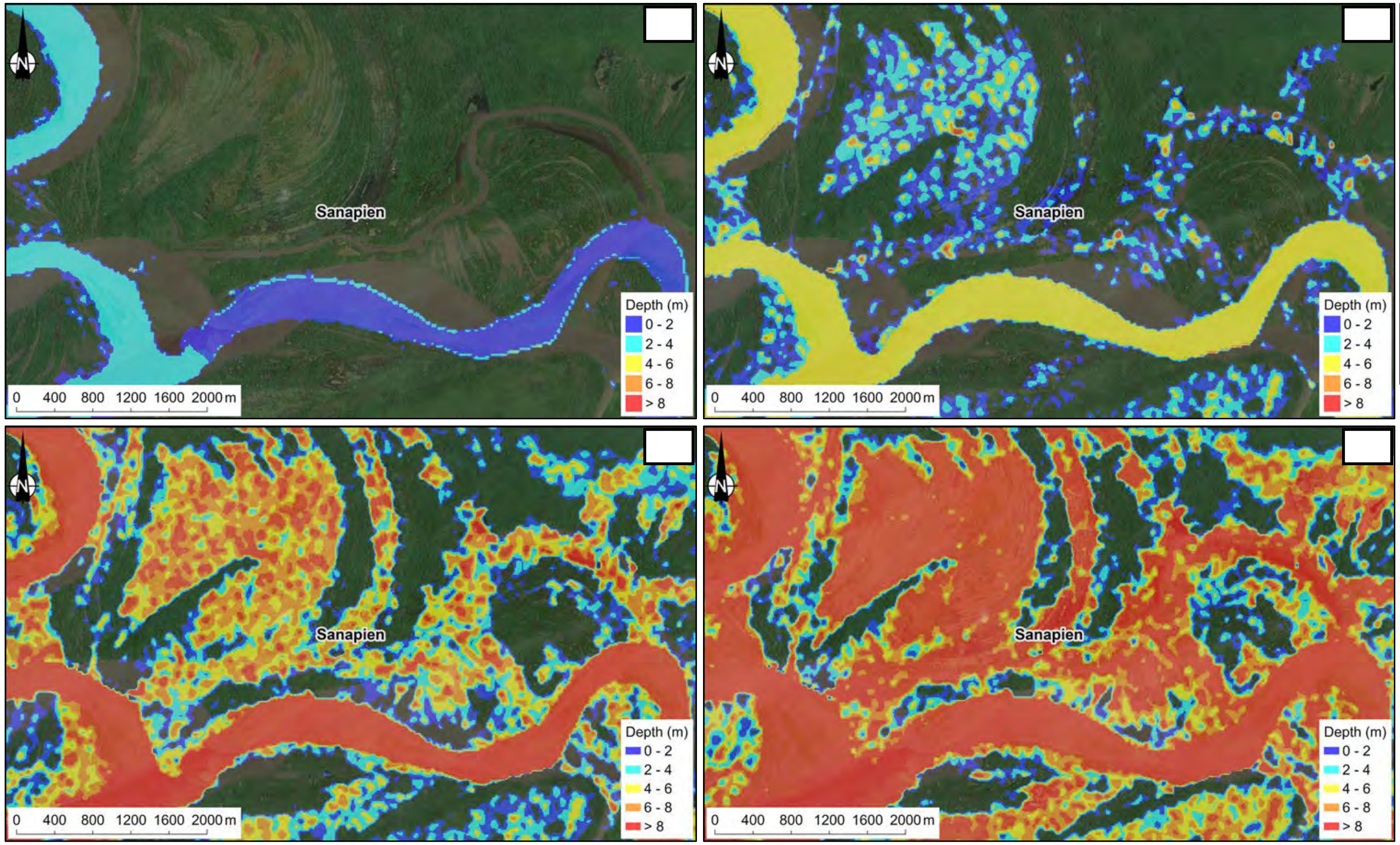



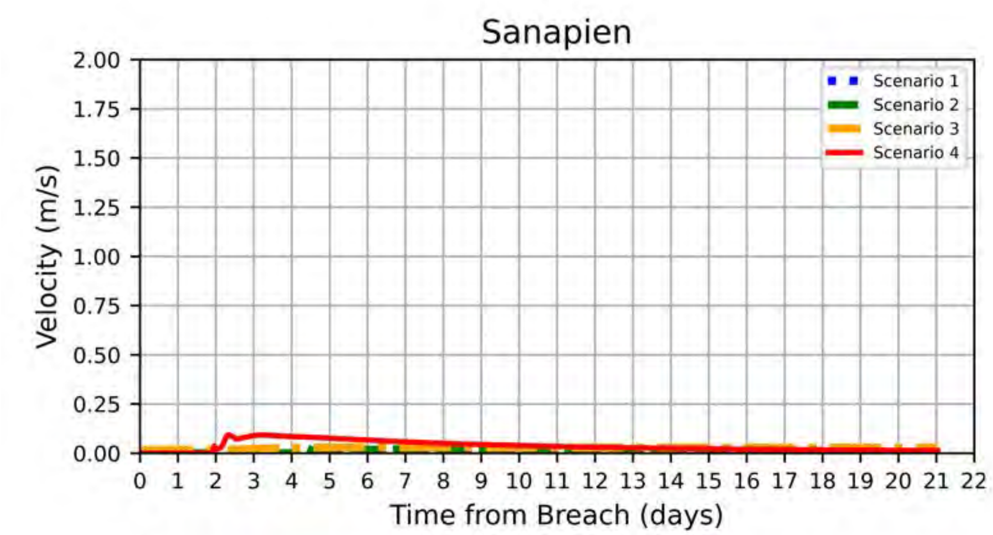


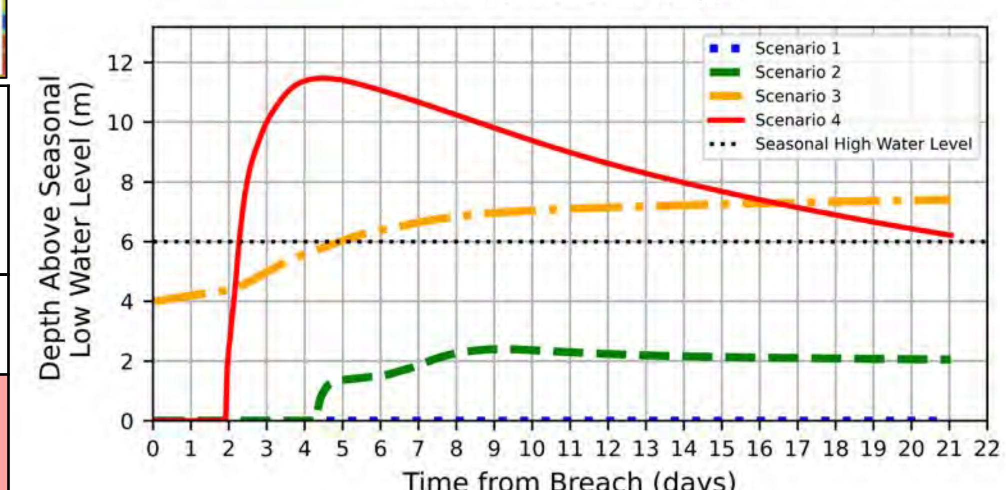


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | **0.26 $Bm^3$** | **1.24 $Bm^3$** | **1.24 $Bm^3$** | **6.37 $Bm^3$** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **2.4, [-], 0.06** | **7.4, [1.4], 0.04** | **11.47, [5.47], 0.14** |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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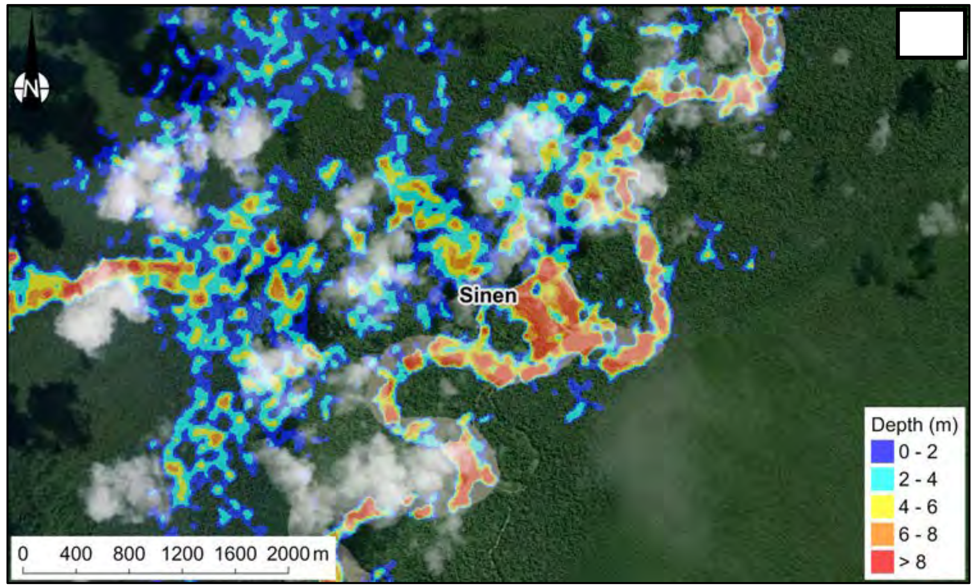


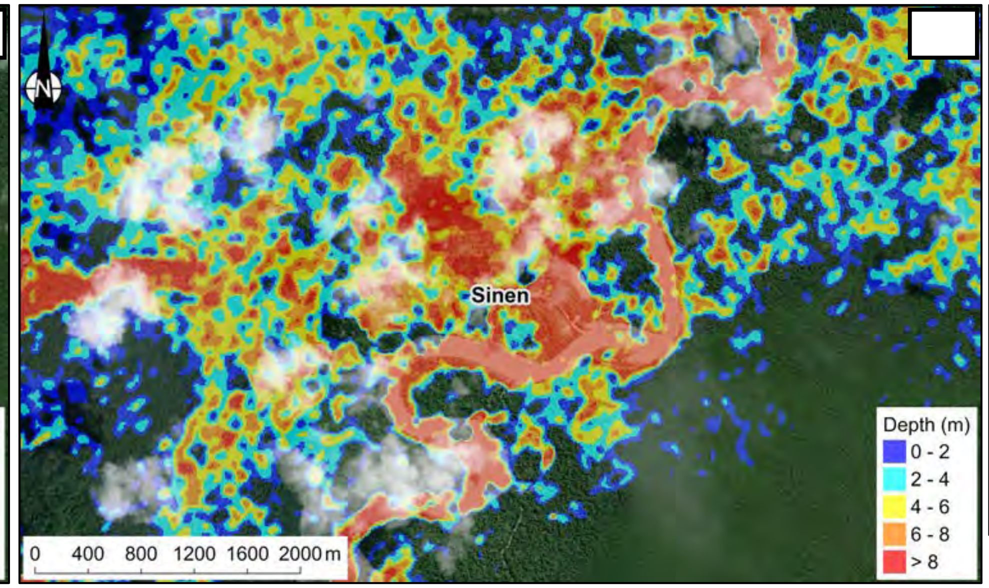



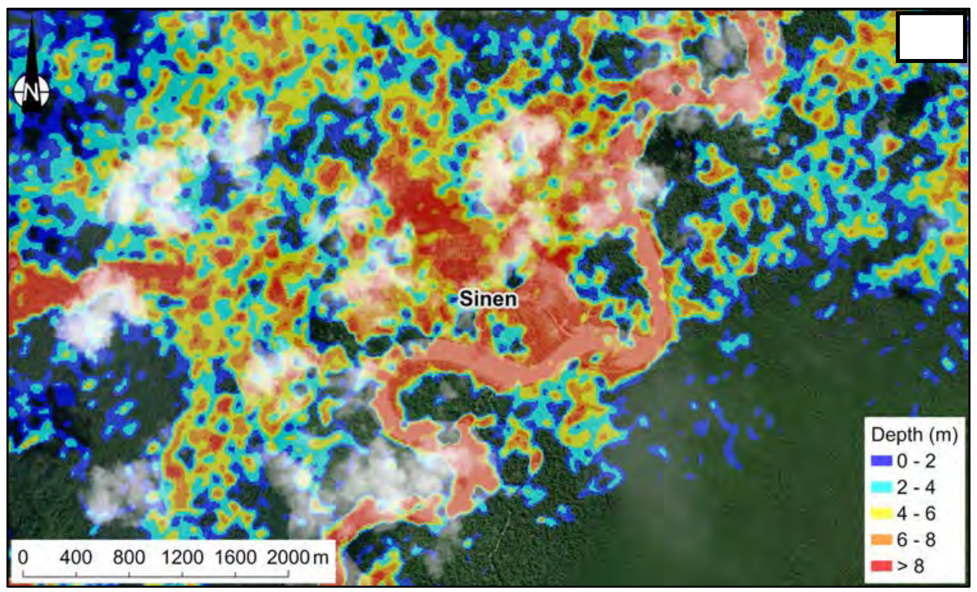


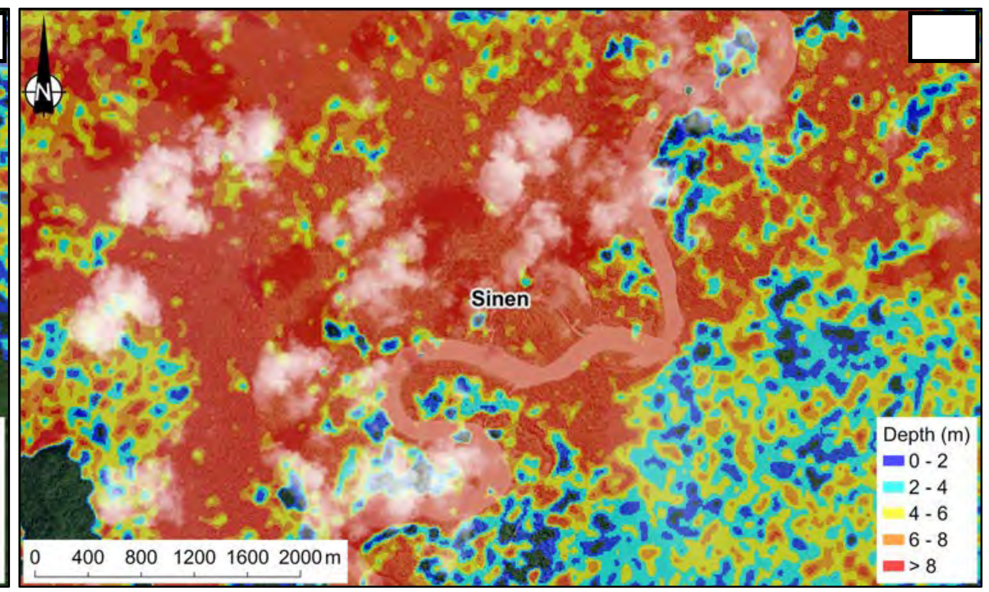


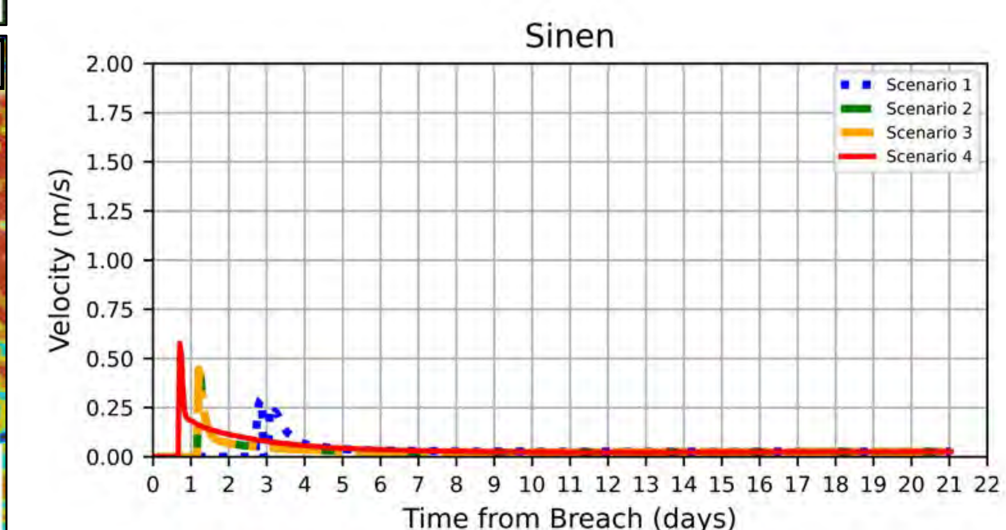


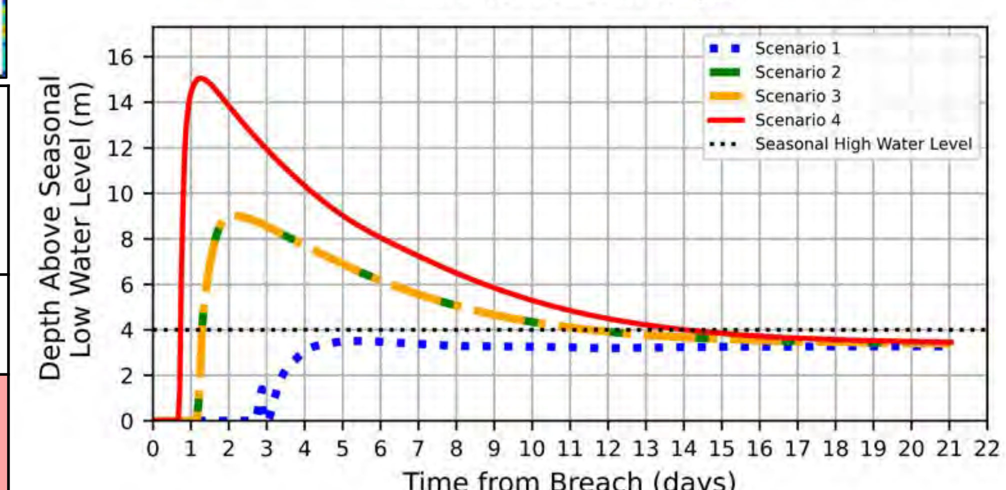


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 3.51, [-], 0.33 | 9.01, [5.01], 0.48 | 9.01, [5.01], 0.48 | 15.06, [11.06], 0.7 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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Frieda River Dam

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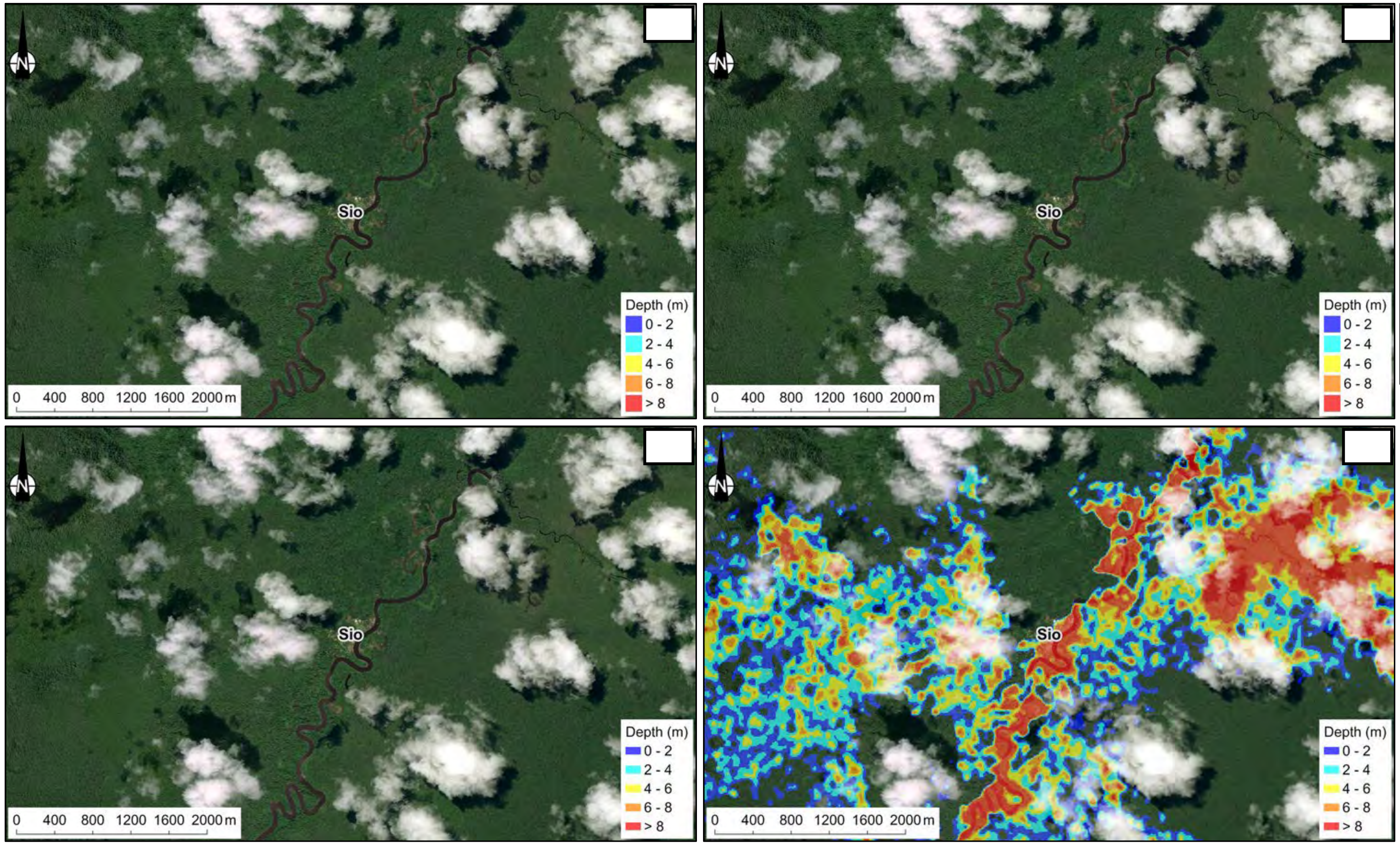



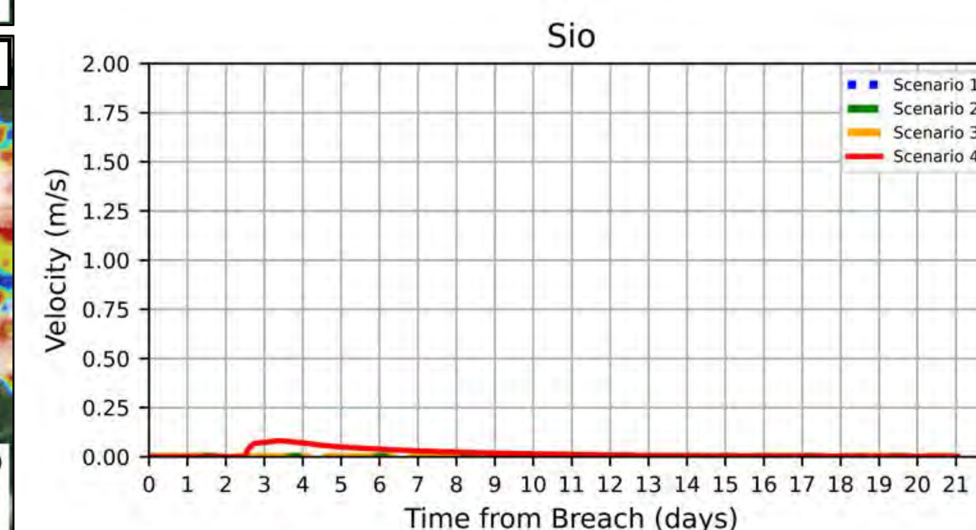


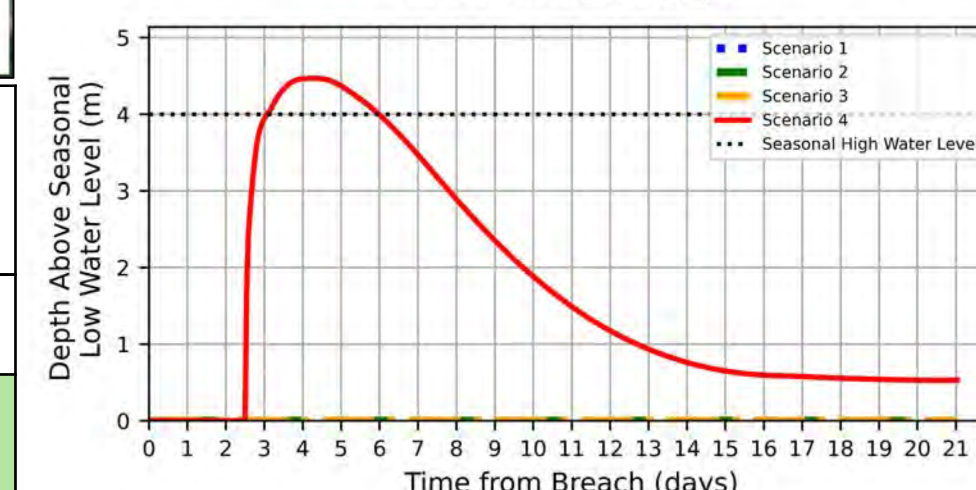


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| **Release Volume ($m^3$)** | **0.26 $Bm^3$** | **1.24 $Bm^3$** | **1.24 $Bm^3$** | **6.37 $Bm^3$** |
| **Maximum Depth, [depth above seasonal high-water level], velocity[1]** | **Not Inundated** | **Not Inundated** | **Not Inundated** | **4.47, [0.47], 0.13** |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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Frieda River Dam

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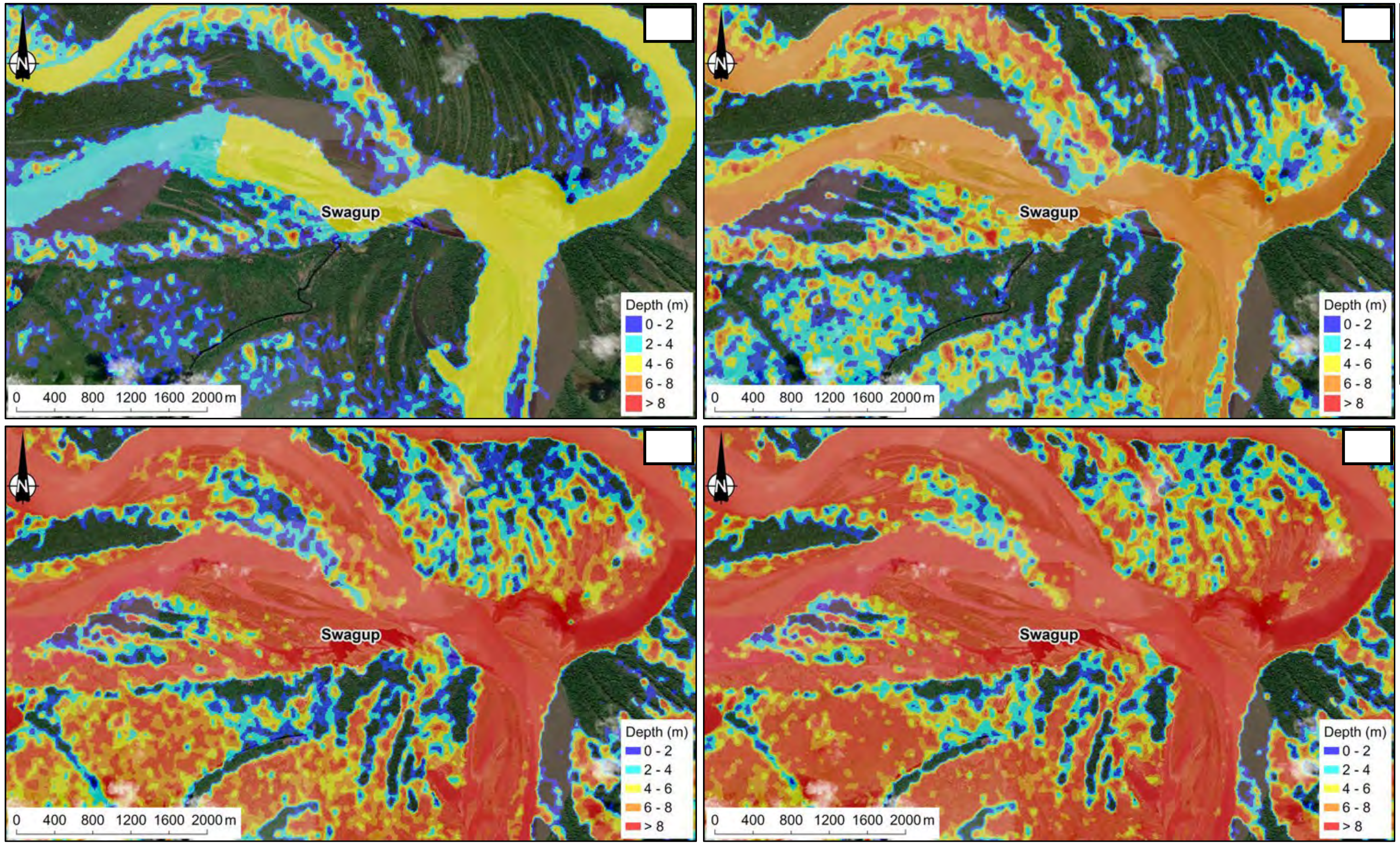



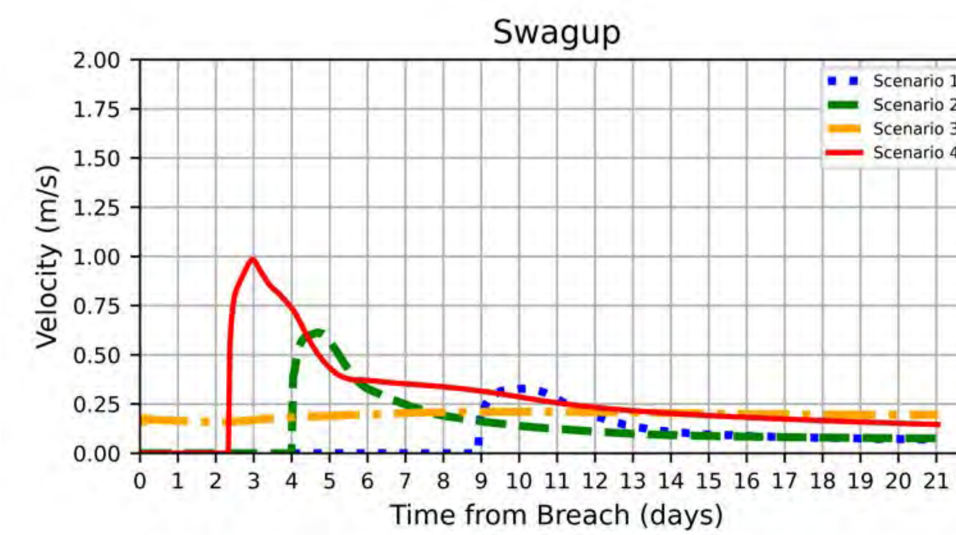


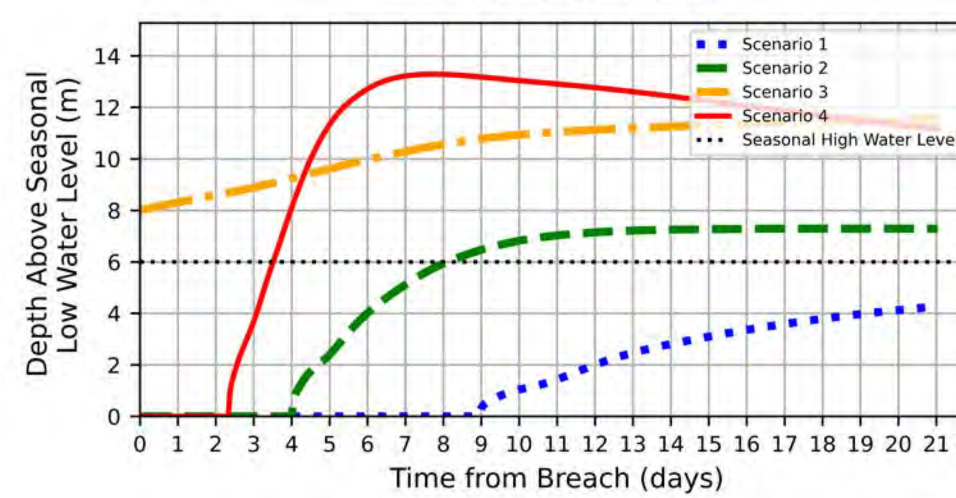


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 4.27, [-], 0.35 | 7.29, [1.29], 0.66 | 11.55, [5.55], 0.22 | 13.29, [7.29], 1.06 |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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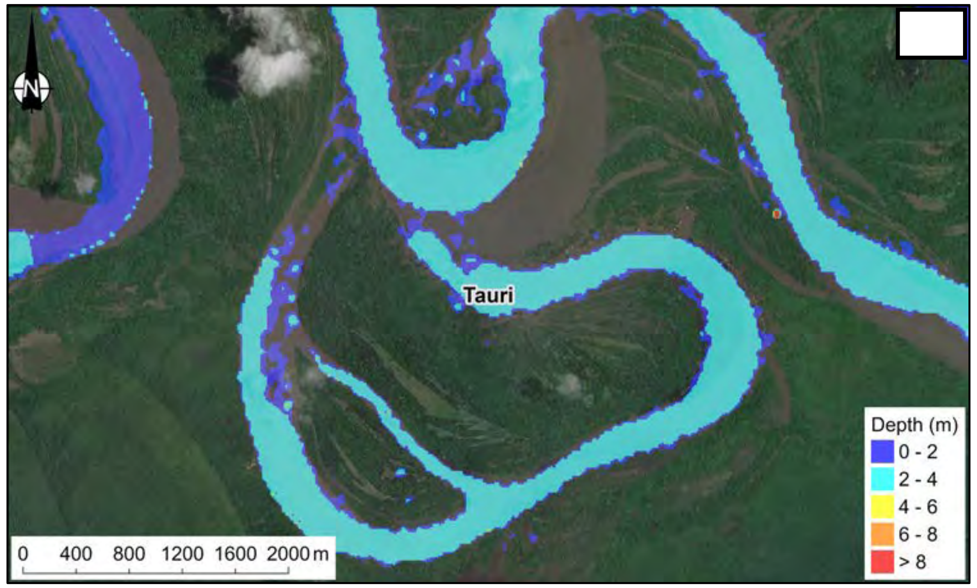


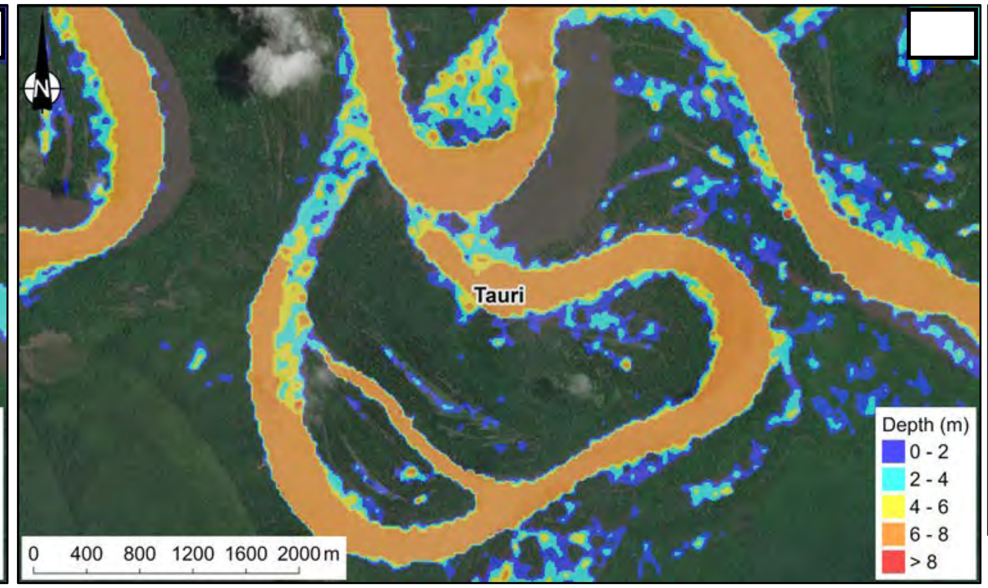



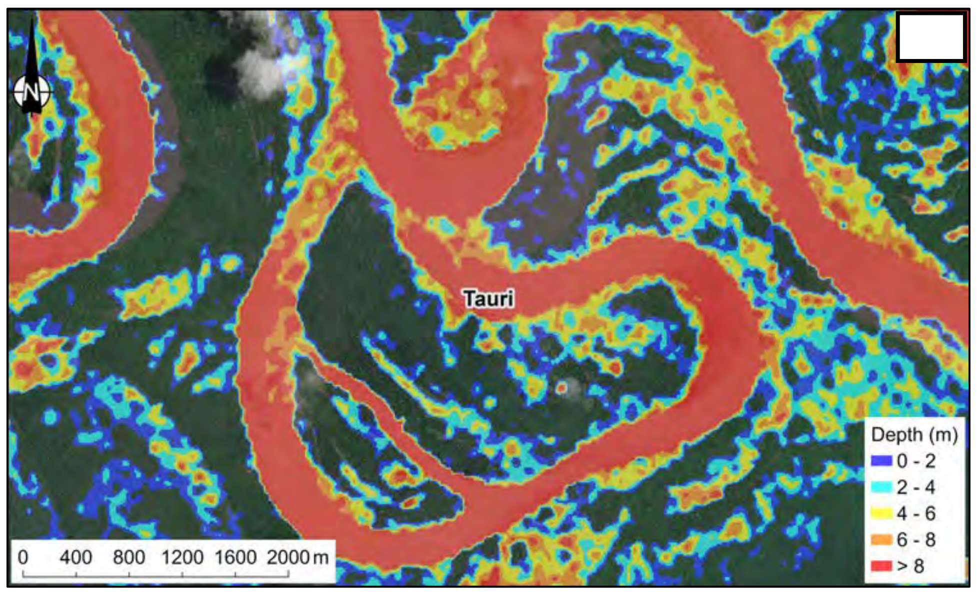


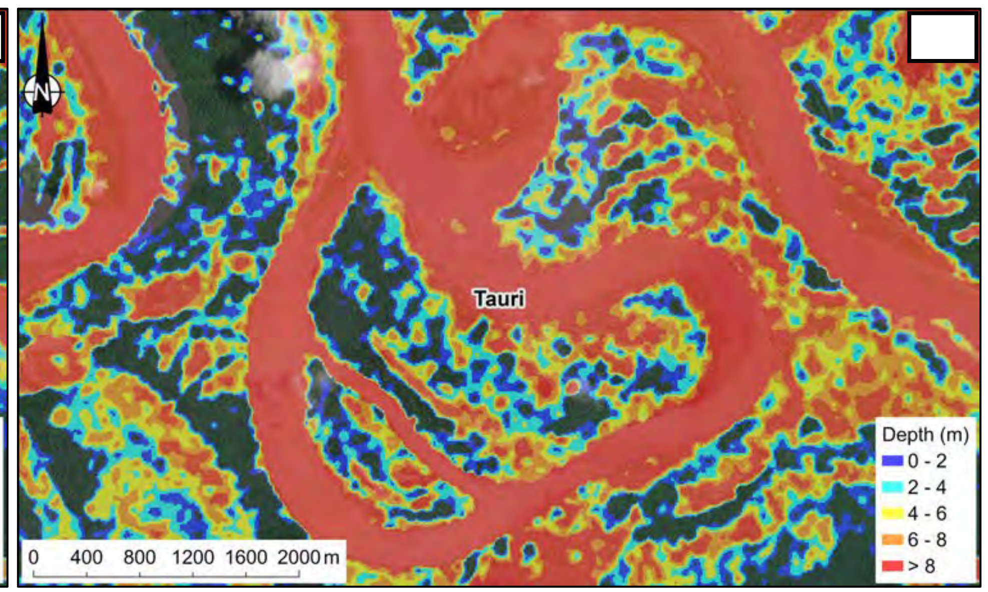


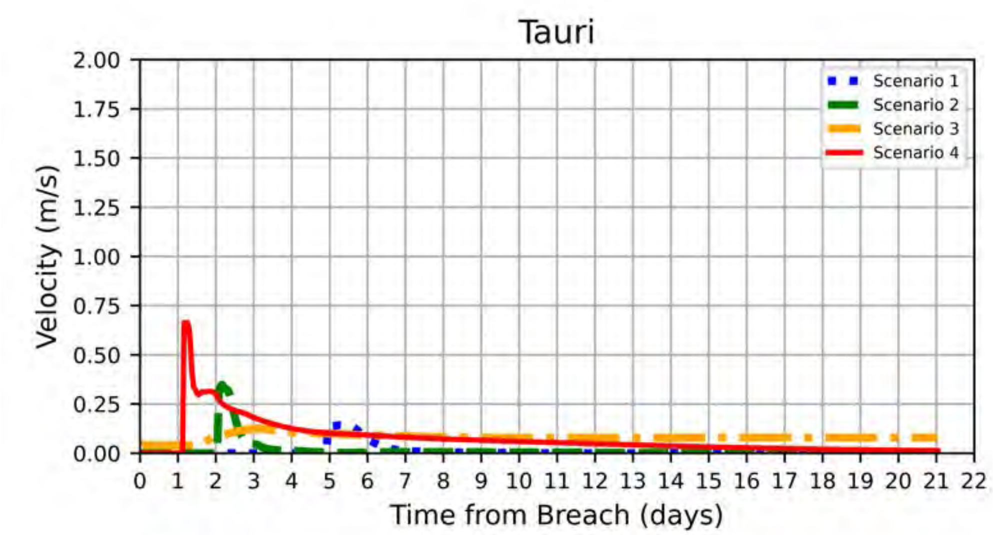


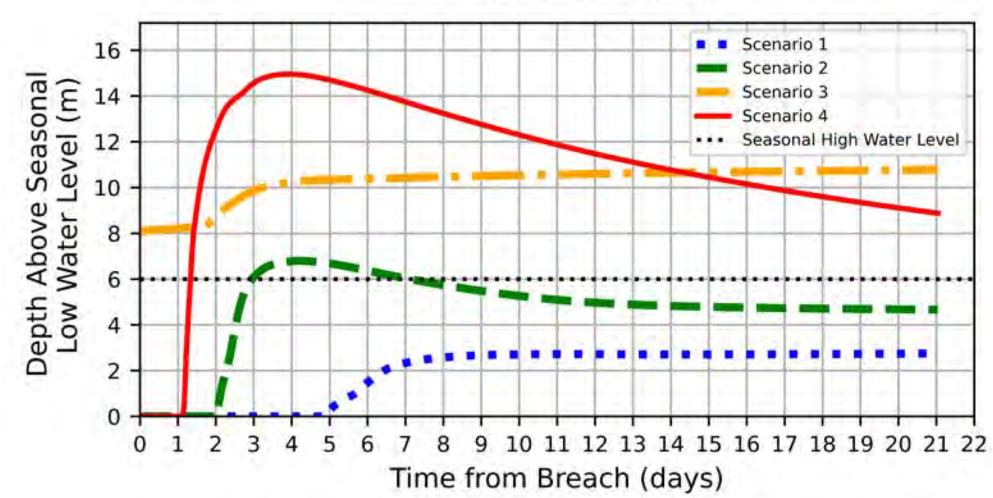


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume (m³) | 0.26 Bm³ | 1.24 Bm³ | 1.24 Bm³ | 6.37 Bm³ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 2.75, [-], 0.24 | 6.79, [0.79], 0.45 | 10.77, [4.77], 0.12 | 14.95, [8.95], 0.79 |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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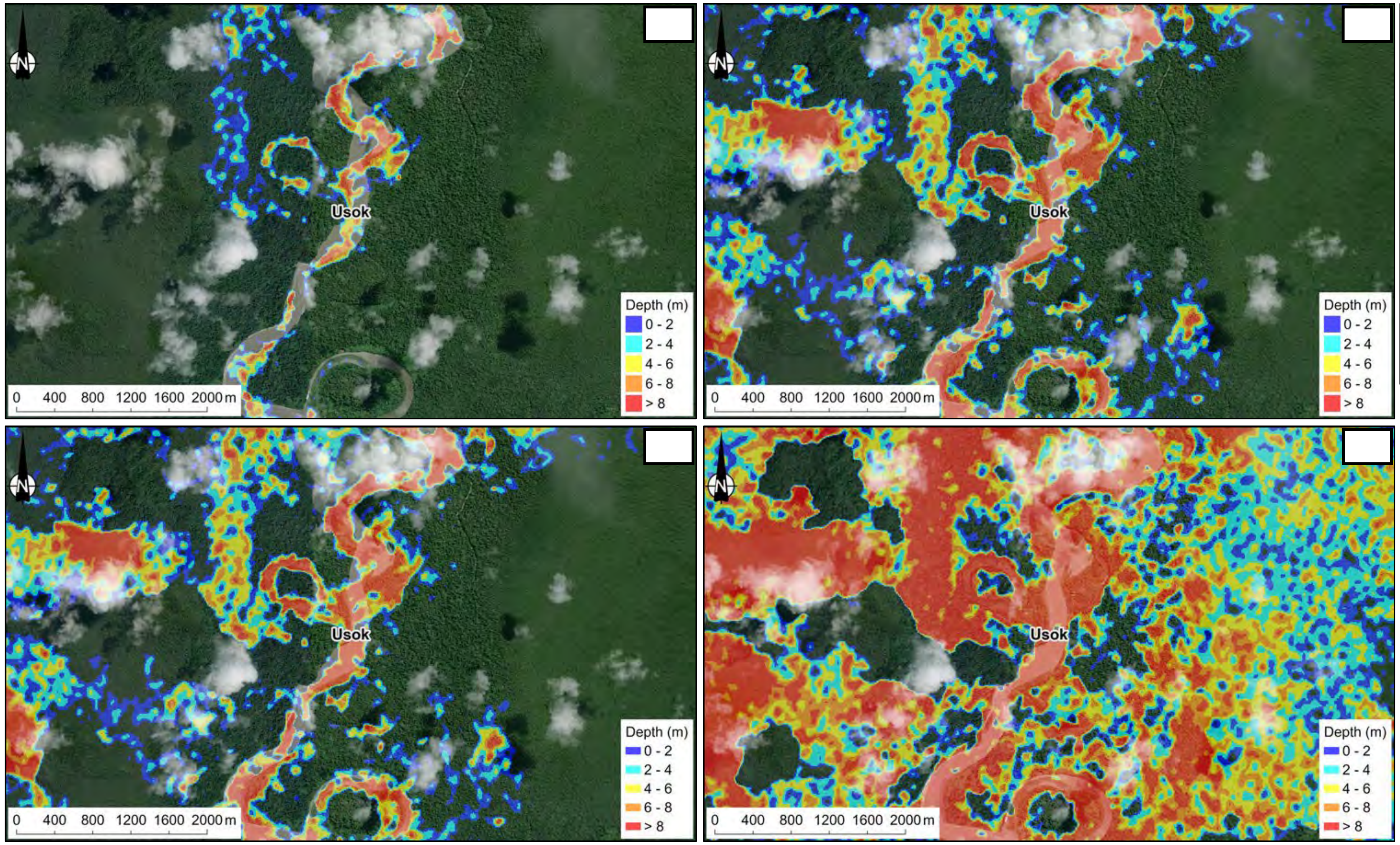



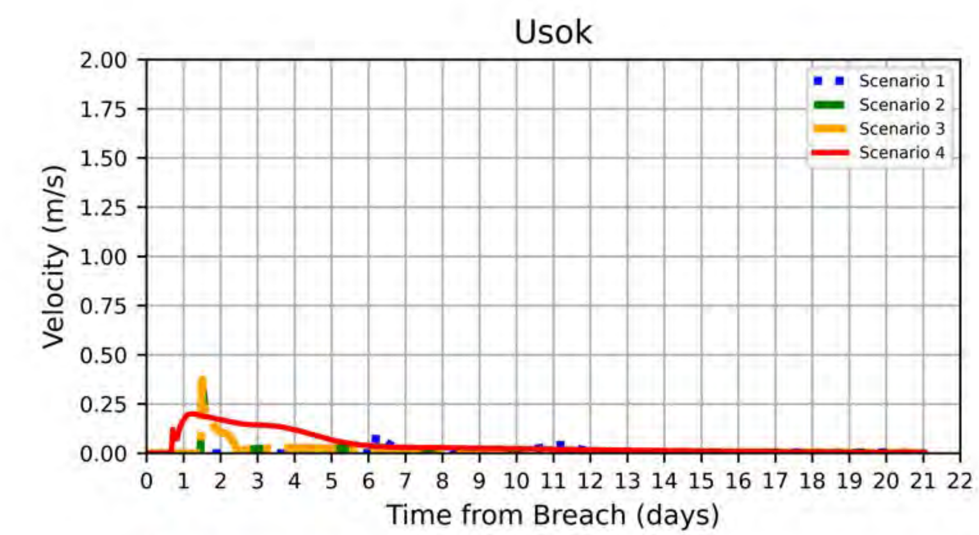


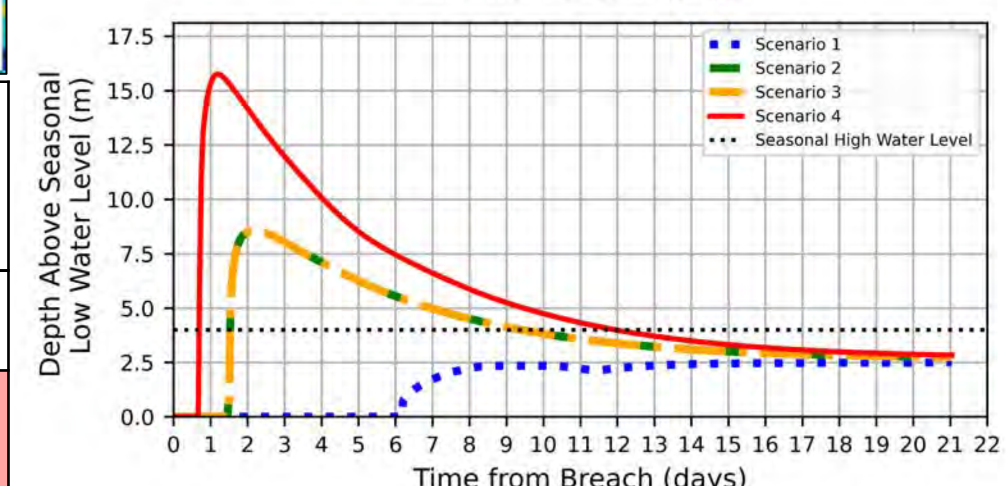


| Scenario | (1) 2m Breach – Seasonal Low Initial Water Level | (2) 10m Breach - Seasonal Low Initial Water Level | (3) 10m Breach – Seasonal High Initial Water Level | (4) 66.7m Breach (Max) – Seasonal Low Initial Water Level |
|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 2.5, [-], 0.21 | 8.52, [4.52], 0.95 | 8.52, [4.52], 0.49 | 15.76, [11.76], 0.37 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
2. Damage to infrastructure may still occur for areas considered potentially non-hazardous zones (shaded in green).
3. Community locations are based on coordinates provided in SRK (2018) and should be considered approximate.

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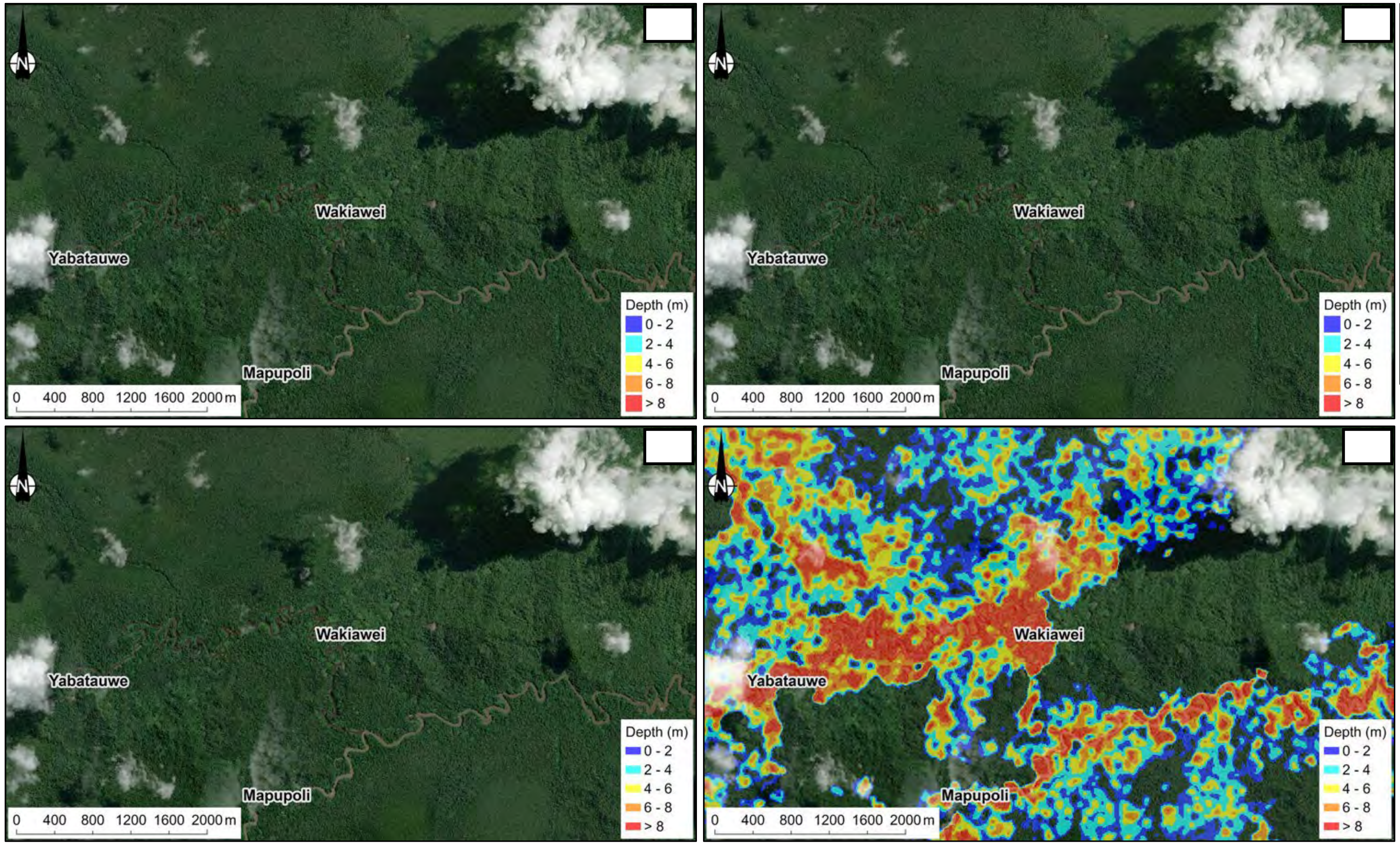



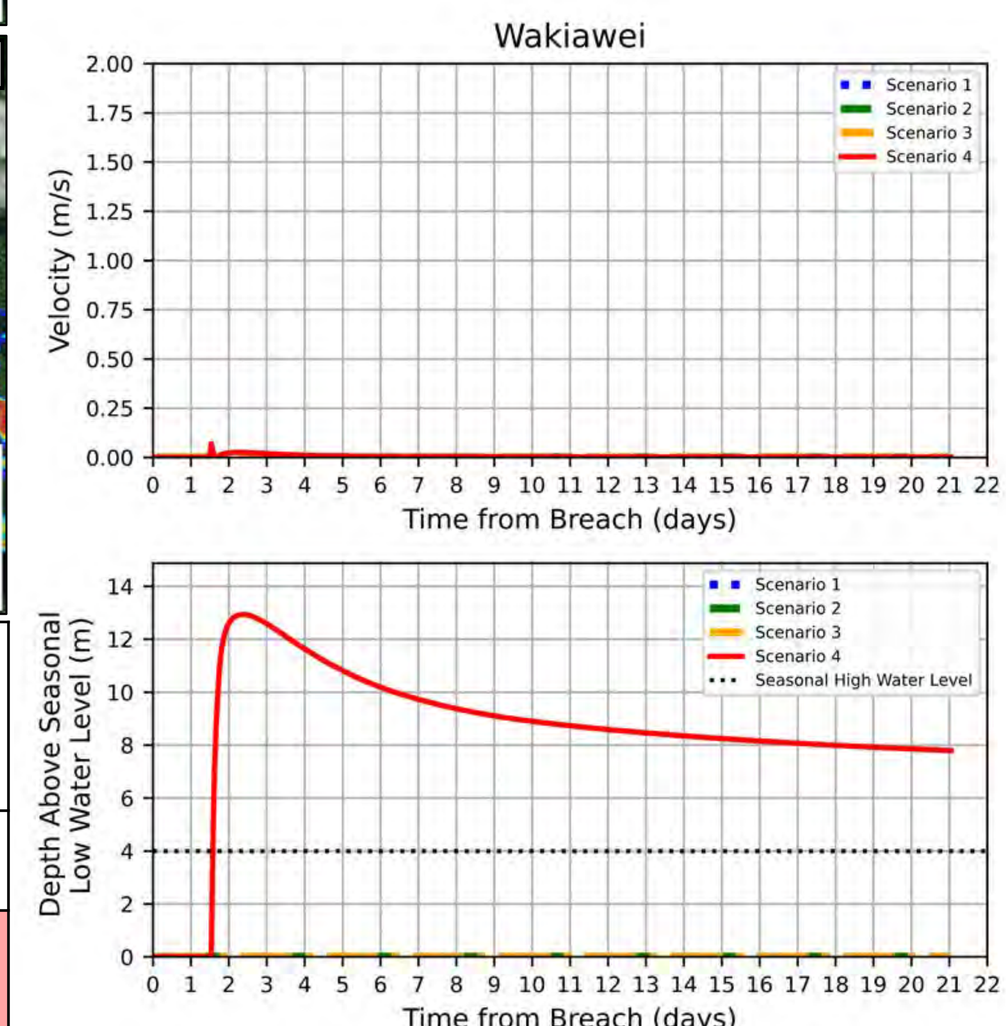


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|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | Not Inundated | 12.93, [8.93], 0.11 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
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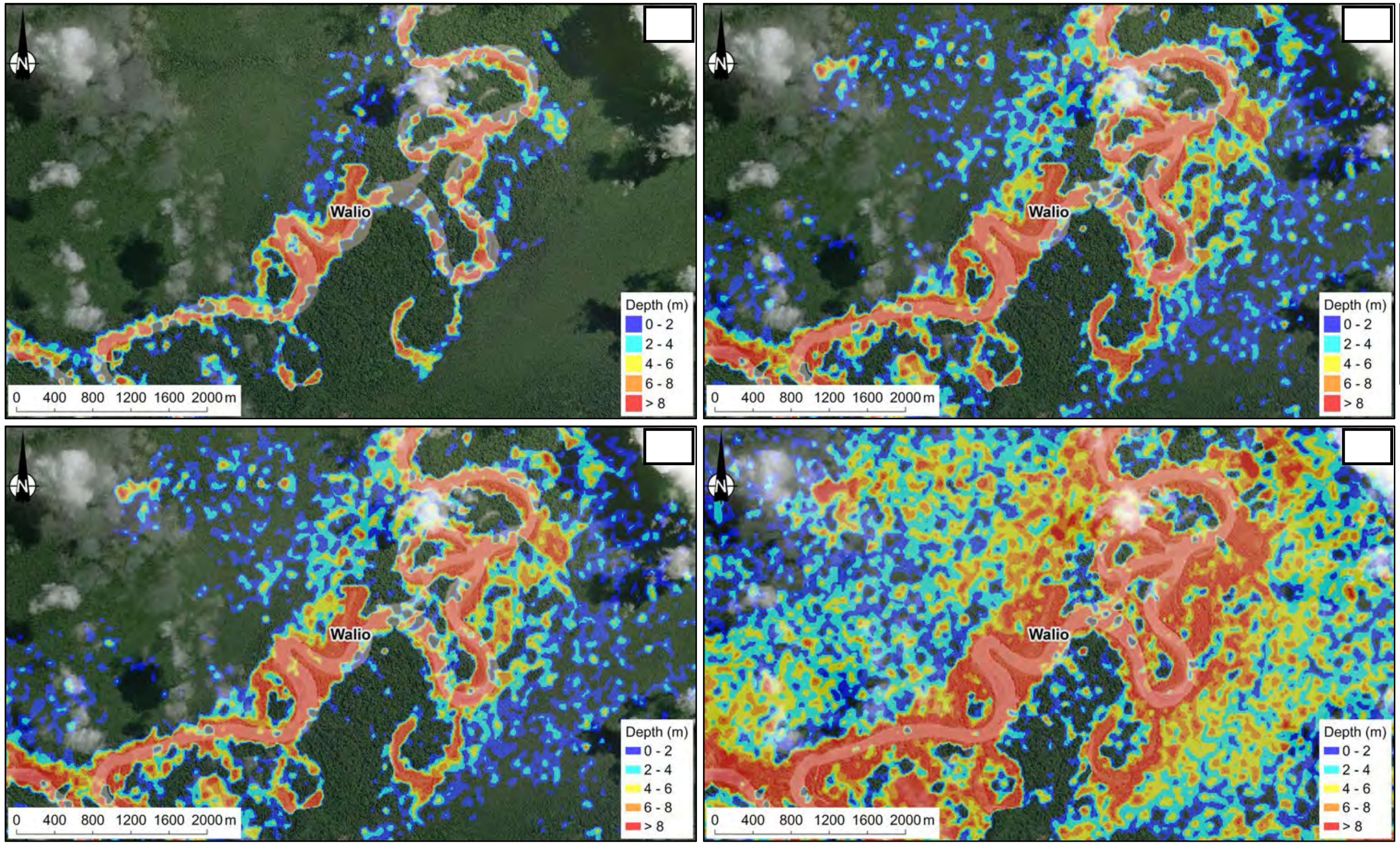



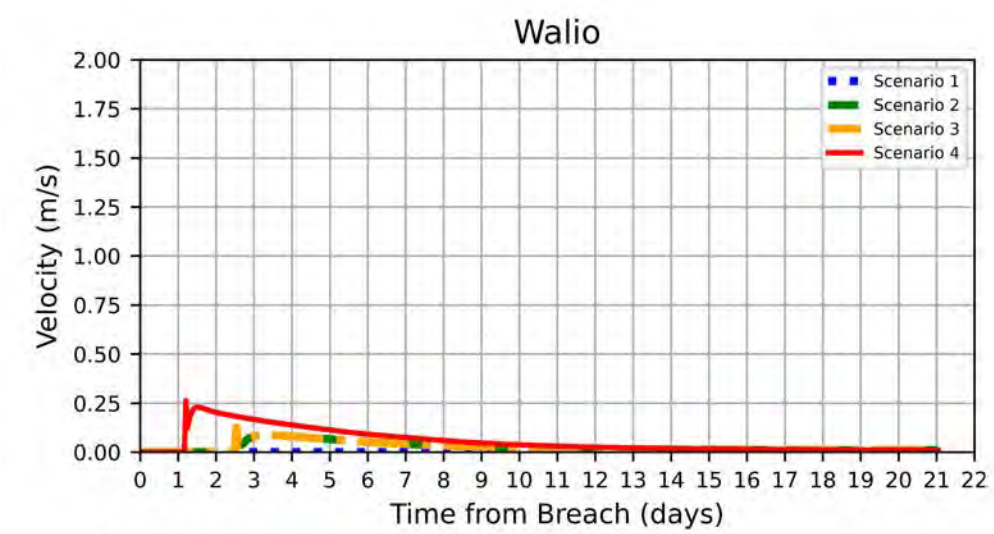


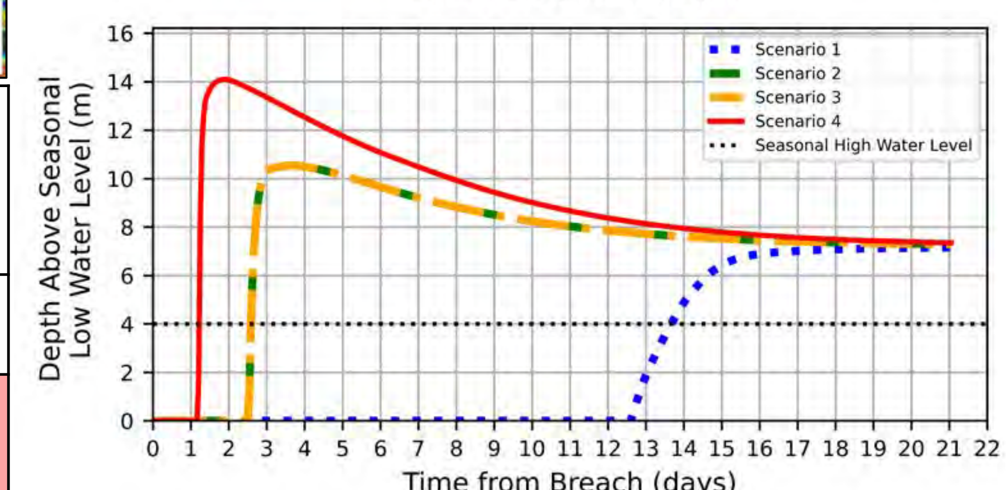


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|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | 7.15, [3.15], 0.12 | 10.54, [6.54], 0.23 | 10.54, [6.54], 0.23 | 14.1, [10.1], 0.29 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
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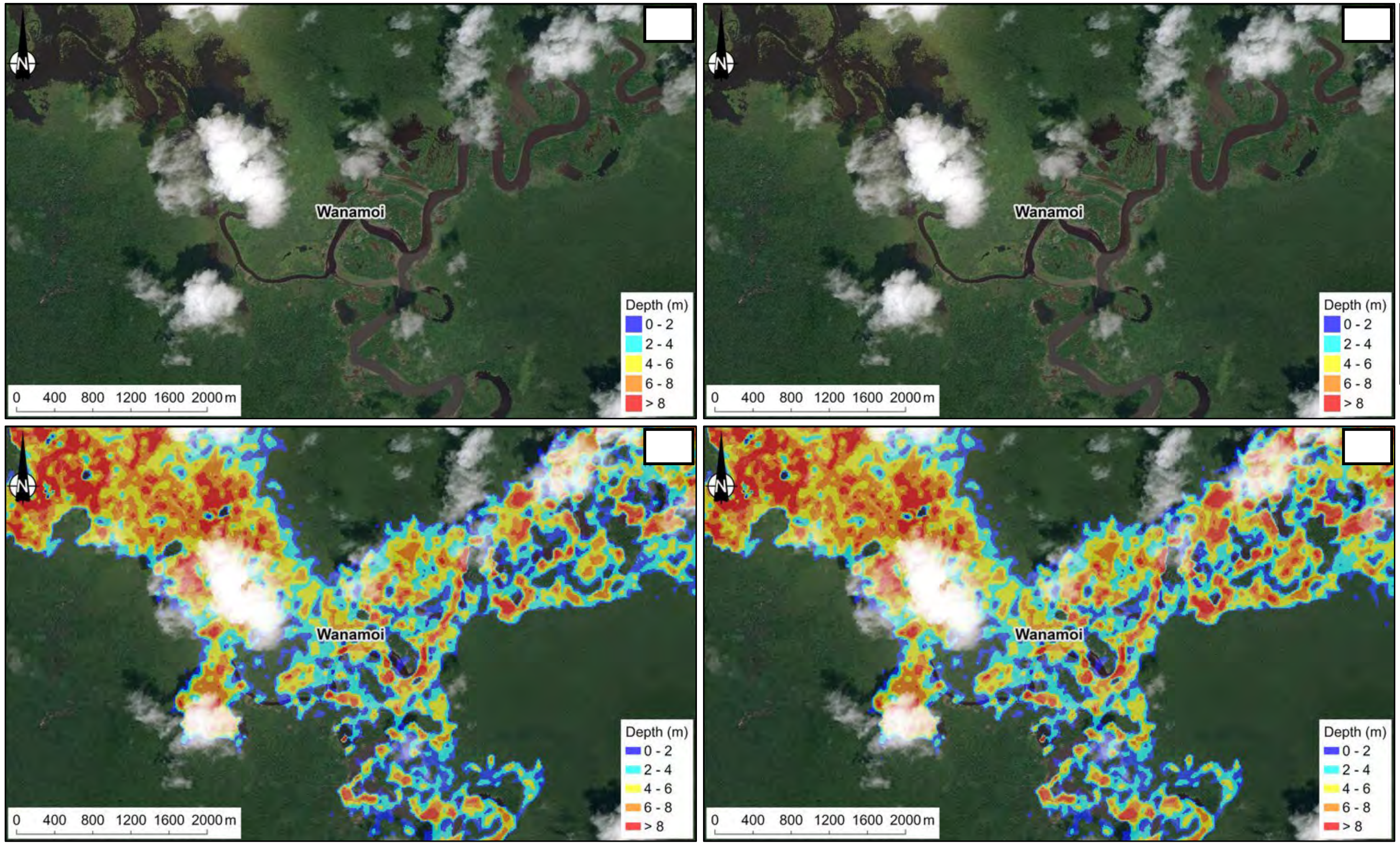



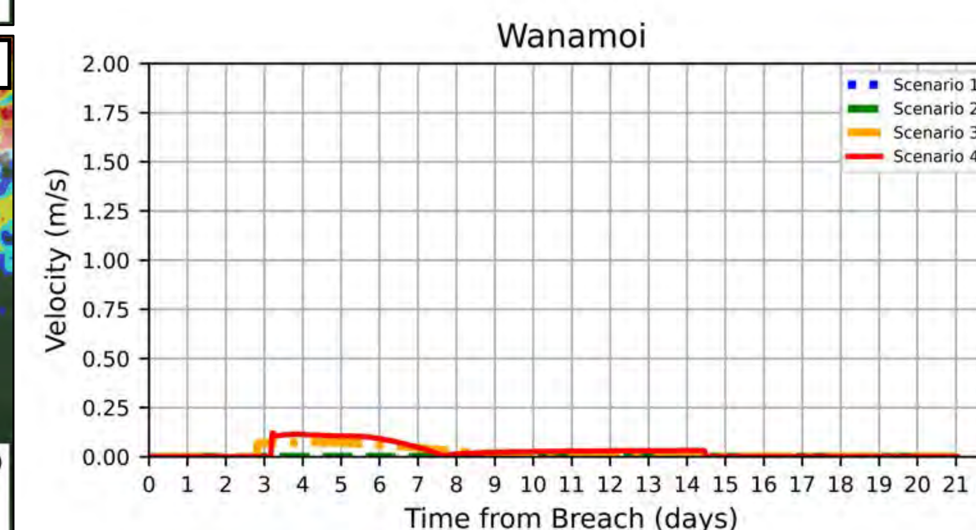


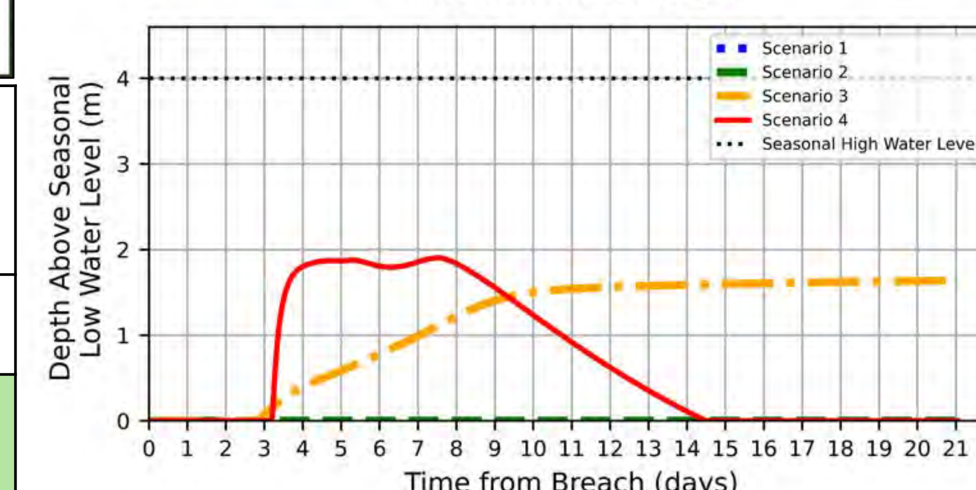


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|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | 1.64, [-], 0.07 | 1.9, [-], 0.22 |

Notes:
1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
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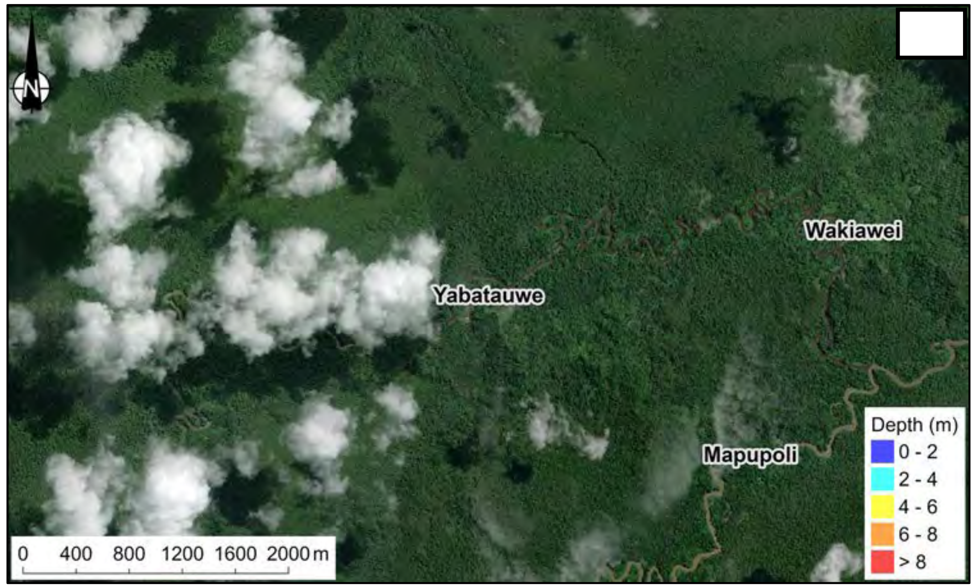


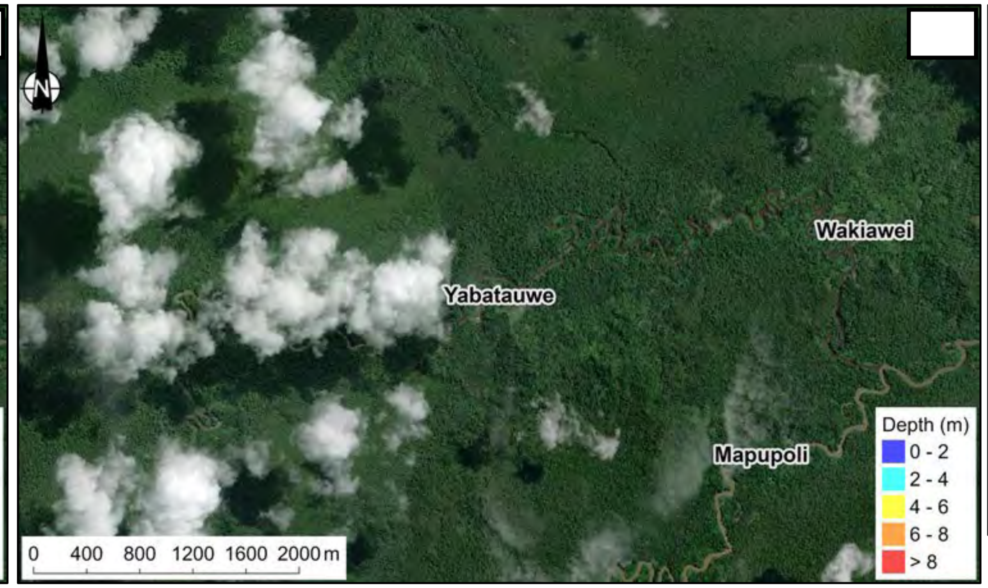


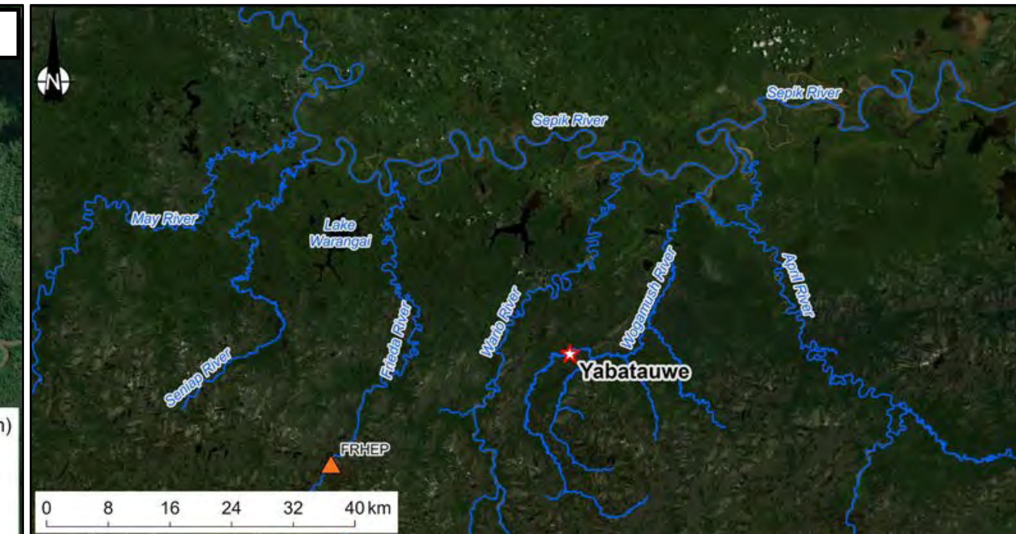


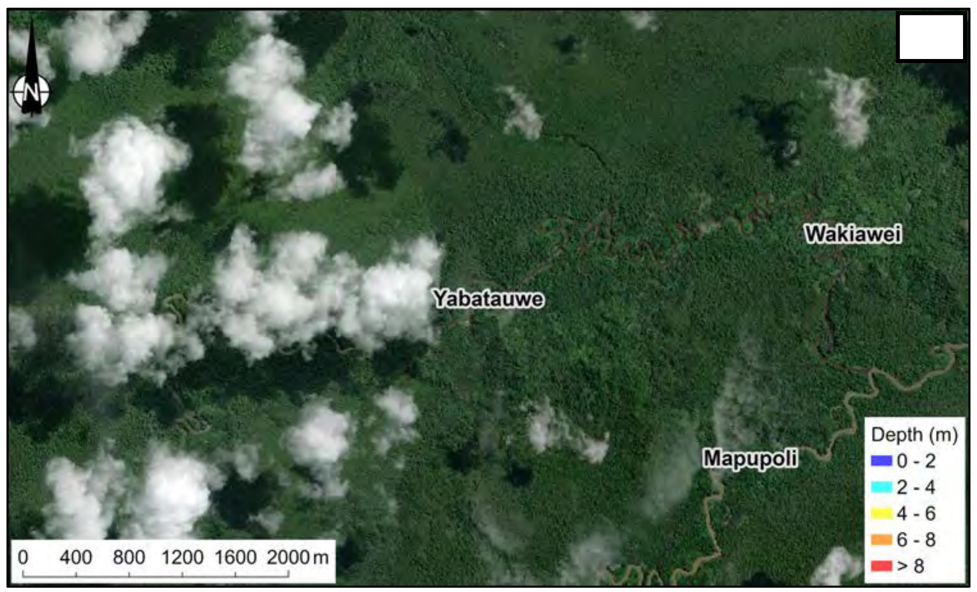


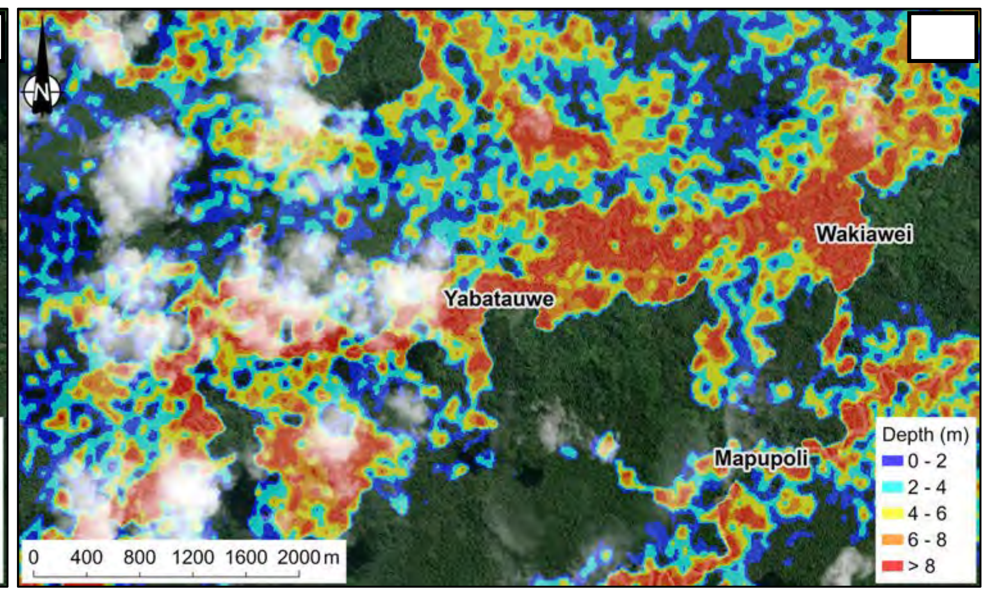


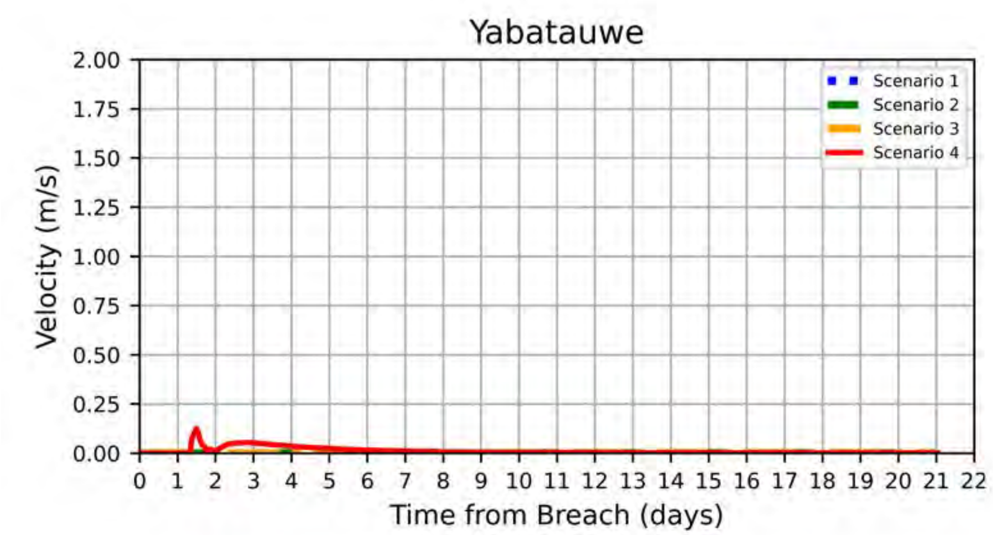


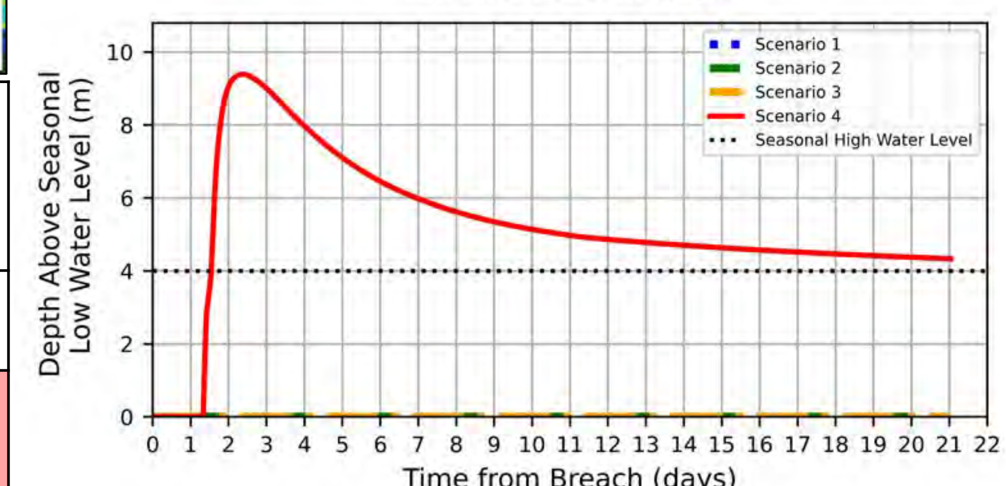


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|---|---|---|---|---|
| Release Volume ($m^3$) | 0.26 $Bm^3$ | 1.24 $Bm^3$ | 1.24 $Bm^3$ | 6.37 $Bm^3$ |
| Maximum Depth, [depth above seasonal high-water level], velocity[1] | Not Inundated | Not Inundated | Not Inundated | 9.39, [5.39], 0.25 |

Notes:

1. Red shading indicates community is in high danger zone for the breach event. Green shading indicates the community is in a potentially non-hazardous zone for the breach event.
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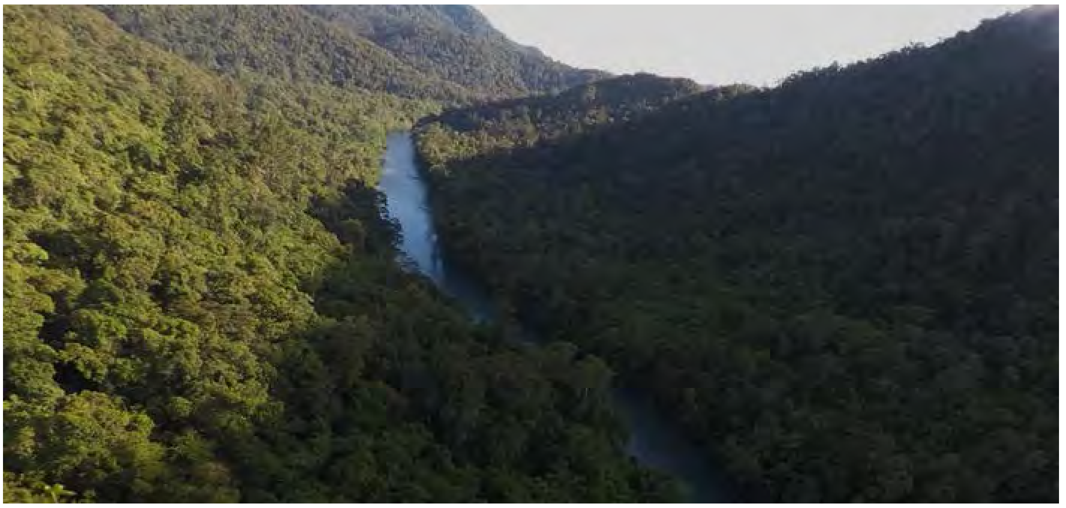
# Frieda River Hydroelectric Project Selection Phase Study

# Dam Break Assessment

**Report Prepared for**

## Frieda River Limited



**Report Prepared by**

SRK Consulting (Australasia) Pty Ltd

PNA009

July 2018

# Frieda River Hydroelectric Project

# Selection Phase Study

# Dam Break Assessment


## Frieda River Limited

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Brisbane QLD 4006

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West Perth WA 6005

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Tel: +61 08 9288 2000
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**PNA009**

**July 2018**


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Senior Consultant

Email: cprinsloo@srk.com.au

**Peer Reviewed by**

Pepe Moreno
Principal Consultant

Authors:

Dave Luppnow, Claude Prinsloo, Heather Thomson, Jeremy Brunet; Pepe Moreno; Rebecca Getty

# Executive Summary

SRK Consulting Limited (SRK) completed a dam break analysis to inform the design regarding the risks associated with the Frieda River Hydroelectric Project (FRHEP), develop inundation maps of potential flood extents and estimate flow at critical locations downstream of the FRHEP in the unlikely event of an FRHEP embankment failure.

This report presents SRK's evaluation of a hypothetical embankment failure. While failures of major dams are very uncommon, to develop appropriate dam classification and dam safety systems, it is necessary to consider the consequences of an embankment failure.

Three failure scenarios have been considered: Rainy Day, Sunny Day and spillway gates accidentally opened. The first two corresponds to the recommendations of International Committee if Large Dams (ICOLD) 1998, whilst the latter is a subset scenario of Sunny Day.

A HEC-RAS 5.0.3 model was developed to assess the possible consequences of the hypothetical breach scenarios. The results of the modelling were used to create inundation maps and predict flood wave arrival times, depths and maximum flow velocities. Based on the results of the analysis, SRK concludes as follows:

- Potential breach volumes have been estimated to be release in the order of 10,000 $Mm^3$ or 100% of stored volume as water.
- Within the extents of the modelling, dam breach flows are expected to impact 33 villages downstream of the breach. The most critical village affected is Paupe, which has a warning time of 2 hours from initiation of the breach or almost instantaneously after catastrophic failure. The first of the remaining villages will have a minimum of 6 hours before the breach flood arrives. All villages are expected to be impacted within 4 days of the initial breach.
- The flood wave is predicted to reach the Sepik River approximately 9 hours after the breach is initiated, and the wave height is expected to affect river traffic for some distance downstream on the Sepik River.
- A sudden opening of the spillway gates on a Sunny Day scenario will significantly affect at least 12 of the villages. The flood wave of at least 0.3 m is predicted to reach Paupe approximately 3 - 4 hours after the gates open, while the maximum water depth is expected to reach 14 m.

While a sediment transport analysis has not been undertaken, it should be noted that reactive tailings/ waste will be transported and deposited far downstream of the breached dam, posing a very significant long-term source of contamination throughout the region.

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Appendix A: Modelling Scenarios

# Disclaimer

# List of Abbreviations

| Abbreviation | Meaning |
|---|---|
| ACC | asphalt concrete core |
| ANCOLD | Australian National Committee on Large Dams |
| $Bm^3$ | billion metres cubed |
| CFRD | concrete faced rockfill dam |
| ERP | emergency response plan |
| FRCGP | Frieda River Copper Gold Project |
| FRHEP | Frieda River Hydroelectric Project |
| FRL | Frieda River Limited |
| GIS | Geographical Information System |
| ICOLD | International Committee on Large Dams |
| IPS | Investigation Phase Study |
| ISF | integrated storage facility |
| LAS | Left abutment spillway |
| LiDAR | Light Detection and Ranging (remote sensing) |
| LOM | life of mine |
| M | million |
| $m^3$ | cubic metres |
| MAA | Multiple Accounts Analysis |
| MW | megawatts |
| PMF | probable maximum flood |
| PNG | Papua New Guinea |
| RAS | Right abutment spillway |
| REL | Robinson Energy Limited |
| RL | reduced level |
| SDP | Sepik Development Project |
| SM | Sinohydro Midstream |
| SPS | Selection Phase Study |
| SRTM | Shuttle Radar Topography Mission |
| TIRP | Tailings Independent Review Panel |
| TSF | tailings storage facility |

# 1 Introduction

Frieda River Limited (FRL) commissioned SRK Consulting (Australasia) Pty Ltd (SRK) to undertake the Selection Phase Study (SPS) design for the Frieda River Hydroelectric Project (FRHEP). The FRHEP is primarily designed as a hydroelectric facility, with the secondary function to store the life of mine (LOM) tailings and waste production.

The Frieda River Project, as currently envisaged, will mine a large copper porphyry deposit in rugged jungle-covered upland terrain of West Sepik Province in Papua New Guinea. The location of the Frieda River Project is shown in Figure 1-1. The area is remote, and there is no road access or reliable communications.

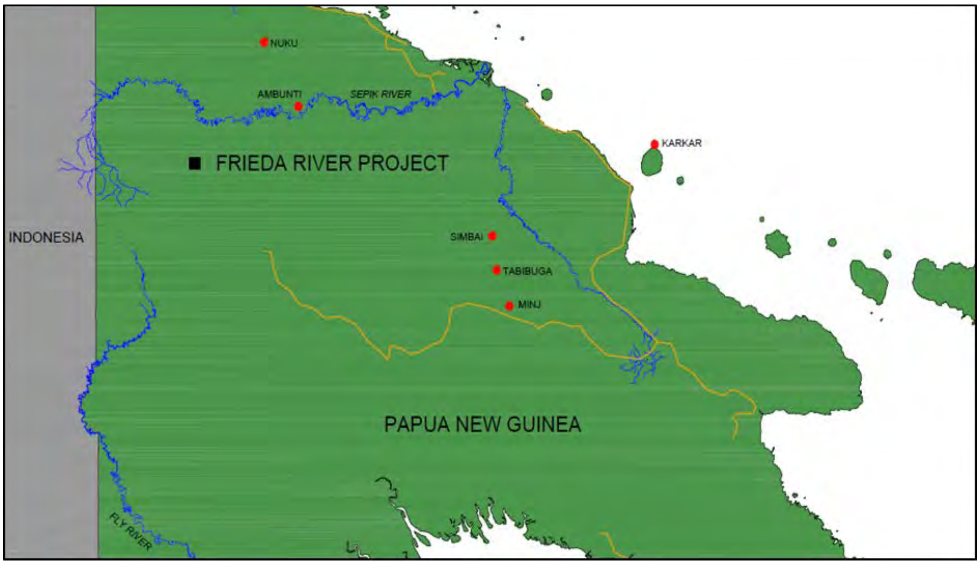



**Figure 1-1: Location of Frieda River Project**

As part of the SPS design, SRK completed a dam break analysis to inform the design regarding the risks associated with the FRHEP, develop inundation maps of potential flood extents and estimate flow at critical locations downstream of the FRHEP in the unlikely event of an FRHEP embankment failure.

This report presents SRK's evaluation of a hypothetical embankment failure. While failures of major dams are very uncommon, to develop appropriate dam classification and dam safety systems, it is necessary to consider the consequences of an embankment failure. The dam break analysis therefore assumes that a failure will occur, and calculates the speed and extent of the resulting flood wave travelling downstream. The calculations require simplifying assumptions to be made. The general approach is to adopt the assumption that will lead to conservatively short estimates of flood arrival times and conservatively high estimates of inundation depths.

As the FRHEP has a gated spillway, SRK also evaluated the likely flooding should the spillway gates malfunction and inadvertently open, without a flood occurring. This modelling was undertaken to estimate flood arrival times and inundation depths for this event.

The following items are considered beyond the scope of work of the current study:

- Emergency response plans (ERPs). The results of this study should be used to develop an ERP for the facility to define responsibilities and provide procedures to be followed in the event of a flood, potential failure, or actual failure of the FRHEP embankment.
- Potential flood mitigation measures. Design of control structures to minimise or divert tailings flood inundation has not been undertaken as part of this work scope.

## 1.1 Model inputs

### 1.1.1 Key inputs and assumptions

The key inputs and assumptions used in the dam break analysis are summarised in Table 1-1. It should be noted that a 72-hour probable maximum flood (PMF) inflow hydrograph was developed as part of the Hydrology Study developed for the FRHEP[1], which was shown to have the largest peak flow (approximately 30,000 $m^3/s$). However, the 12-hour PMF peak flow occurs much earlier, during the critical period of the embankment breach outflow. The estimated dam break outflows are 20 to 30 times greater than the PMF, therefore the selection of the PMF is not likely to be material to the results of this study.

**Table 1-1: Input parameters**

| | Parameter | Value | Comment |
|---|---|---|---|
| **Storage** | Total FRHEP storage capacity | 10,761 $Mm^3$ | At FRHEP crest (RL 235 m) |
| | Maximum storage volume | 9,640 $Mm^3$ | At maximum operating level (RL 226.1 m) |
| | Percentage solids storage | 0% - 22% | Tailings, waste and sediment percentage of total storage volume – range over the 33-year operating life |
| **Embankment and Foundation** | Embankment crest level | RL 235 m | |
| | Downstream embankment slope | 1V: 2H | Assumed for breach modelling. Subject to ongoing design activities to optimise. |
| | Embankment fill properties<br>- Density<br>- Friction angle | 22 $kN/m^3$<br>40° - 46° | Assumed for compacted rockfill |
| **Design Storm** | 12-hour PMF peak flow | 26,000 $m^3/s$ | Source: SRK, 2018[1] |
| | 12-hour PMF volume | 703 $Mm^3$ | |

## 1.2 Flood model

The flood extents were determined with a two-dimensional (2D) unsteady flow-model, with dam breach outflows applied as inflow hydrographs to the 2D flow area. The model was developed using the US Army Corps of Engineers' Hydraulic Engineering Centre River Analysis System (HEC-RAS) Version 5.0.3 software. The model inputs are summarised in the following subsections.

### 1.2.1 Model controls

The 2D computational mesh for the flow area was built using a grid size of 300 m x 300 m which was found adequate for characterising the terrain, the water surface slope and its changes. It should be noted that HEC-RAS Version 5.0.3 uses a different approach to other models, in which each grid cell is not a simple plane, but is assigned a detailed elevation-volume/ area relationship that represents the underlying surface topography. The grid was refined and oriented using break lines along the river banks. A time-step of 60 seconds was selected, which was appropriate to provide numerically stable and accurate solutions for the mesh size and the maximum velocities modelled.

[1] SRK 2018, Memorandum - Hydrology for FRHEP design, Document number PNA009_MEMO_Hydrology FRHEP_Rev3

### 1.2.2 Topography and points of interest

FRL provided LiDAR topography for parts of the FRHEP and areas immediately downstream of the FRHEP. The LiDAR data is high quality and is considered suitable for the dam break assessment. However, it is necessary to extend the dam break analysis beyond the LiDAR topography provided in order to assess downstream impacts to the Sepik River. Additional topography has therefore been sourced from publicly available sources (e.g. Shuttle Radar Topography Mission - SRTM), as necessary. The LiDAR and SRTM topography extents are shown in Figure 6-1.

FRL supplied GIS data that includes rivers (blue) and local potentially affected communities (orange dots) as shown in Figure 1-2.

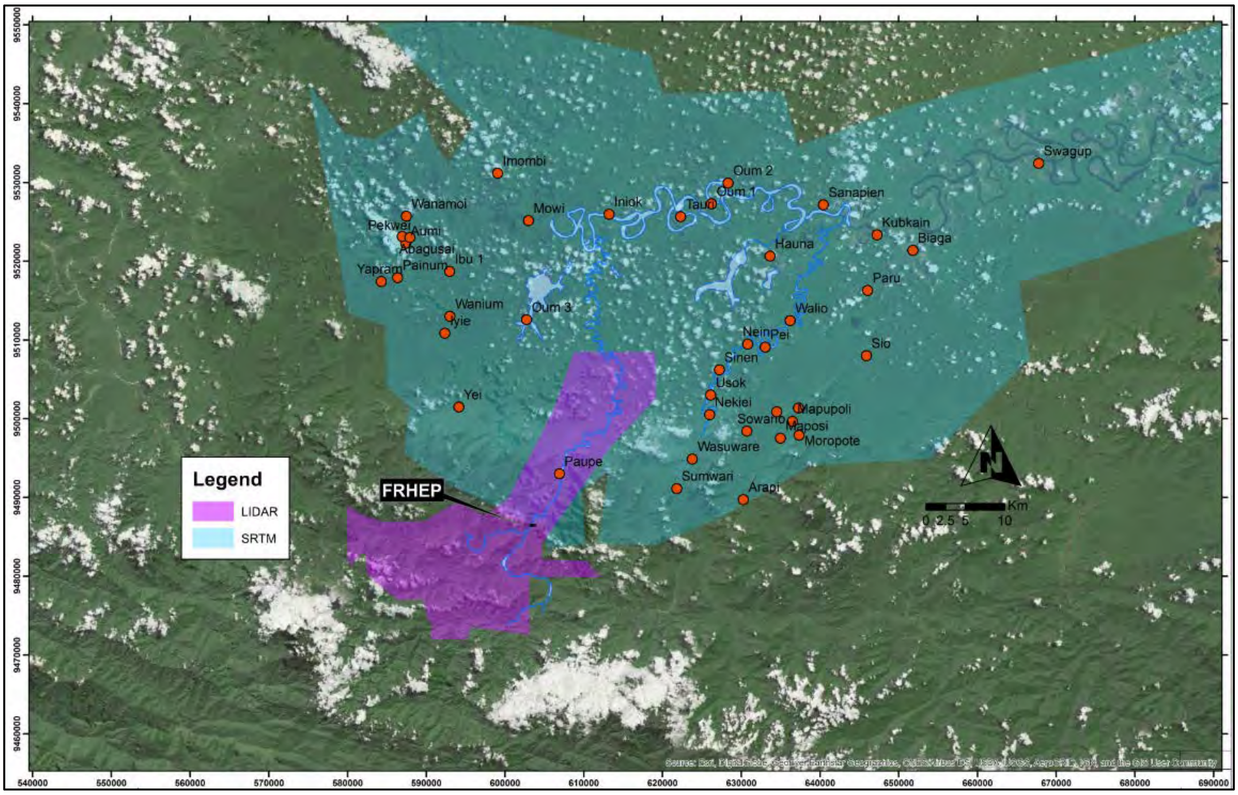



**Figure 1-2: Topography and GIS data**

It is noted that the SRTM survey data has considerably less accuracy and resolution than LiDAR data. SRK has therefore split the dam break modelling into multiple models; the first using the LiDAR data extents, and the second using the outputs of the first model as inputs to the SRTM model. The dam break outputs from the SRTM model are indicative only.

The villages and their locations in terms of topography type and downstream distance, are listed in Table 1-2. Paupe is the only village located within the extent of the LiDAR topography provided that is affected, whereas the remaining 37 villages are within the expanded SRTM extents. The SRTM dataset has a considerably lower resolution than the LiDAR data and the results should be regarded as indicative.

**Table 1-2: List of villages**

| Village/ Point of Interest | Approximate distance downstream (km) | Topography type |
|---|---|---|
| Oum 3 | 31 | SRTM |
| Iniok | 47 | SRTM |
| Paupe | 8 | LiDAR |
| Tauri | 50 | SRTM |
| Mowi | 48 | SRTM |
| Hauna | 56 | SRTM |
| Imombi | 56 | SRTM |
| Kubkain | 69 | SRTM |
| Nekiei | 36 | SRTM |
| Usok | 37 | SRTM |
| Sowano | 46 | SRTM |
| Yabatauwe | 47 | SRTM |
| Wakiawei | 50 | SRTM |
| Mapupoli | 50 | SRTM |
| Maposi | 50 | SRTM |
| Moropote | 52 | SRTM |
| Walio | 51 | SRTM |
| Sanapien | 68 | SRTM |
| Biaga | 72 | SRTM |
| Nein | 45 | SRTM |
| Pei | 47 | SRTM |
| Sio | 58 | SRTM |
| Paru | 62 | SRTM |
| Sinen | 41 | SRTM |
| Wasuware | 45 | SRTM |
| Swagup | 95 | SRTM |
| Oum 1 | 52 | SRTM |
| Oum 2 | 55 | SRTM |
| Yapram | 51 | SRTM |
| Painum | 49 | SRTM |
| Wanium | 41 | SRTM |
| Iyie | 42 | SRTM |
| Aumi | 49 | SRTM |
| Ibu 1 | 42 | SRTM |
| Pekwei | 49 | SRTM |
| Wanamoi | 51 | SRTM |
| Sumwari | 52 | SRTM |
| Yei | 29 | SRTM |

### 1.2.3 Downstream surface roughness

SRK has classified the downstream area into three representative zones of surface roughness represented by Manning's n-values – rivers, highlands and lowlands. The zones were identified using the latest aerial photography and assigned Manning's n-values based on recommendations from US Army Corp of Engineers HEC-1 Manual[2] and Ven Te Chow[3]. Note that these Manning's values are for overland flow and not for open-channel flow, and Manning's n-values are typically higher for overland flow than flow in open-channel structures. It is conservative practice to overestimate the Manning's n-values as this will provide a conservative estimate of inundation. However, sensitivity analysis shows that variation of the Manning's n-values has a negligible impact on the modelling results.

The downstream area zoning is shown in Figure 1-3 and the Manning's n-values assigned are listed in Table 1-3.

**Table 1-3: Manning's n-values used in the model**

| Symbol | Type | Manning's n-value |
|---|---|---|
| | River/ ponded water | 0.02 |
| | Lowlands – vegetated, flood plain | 0.4 |
| | Highlands – heavily vegetated | 0.6 |

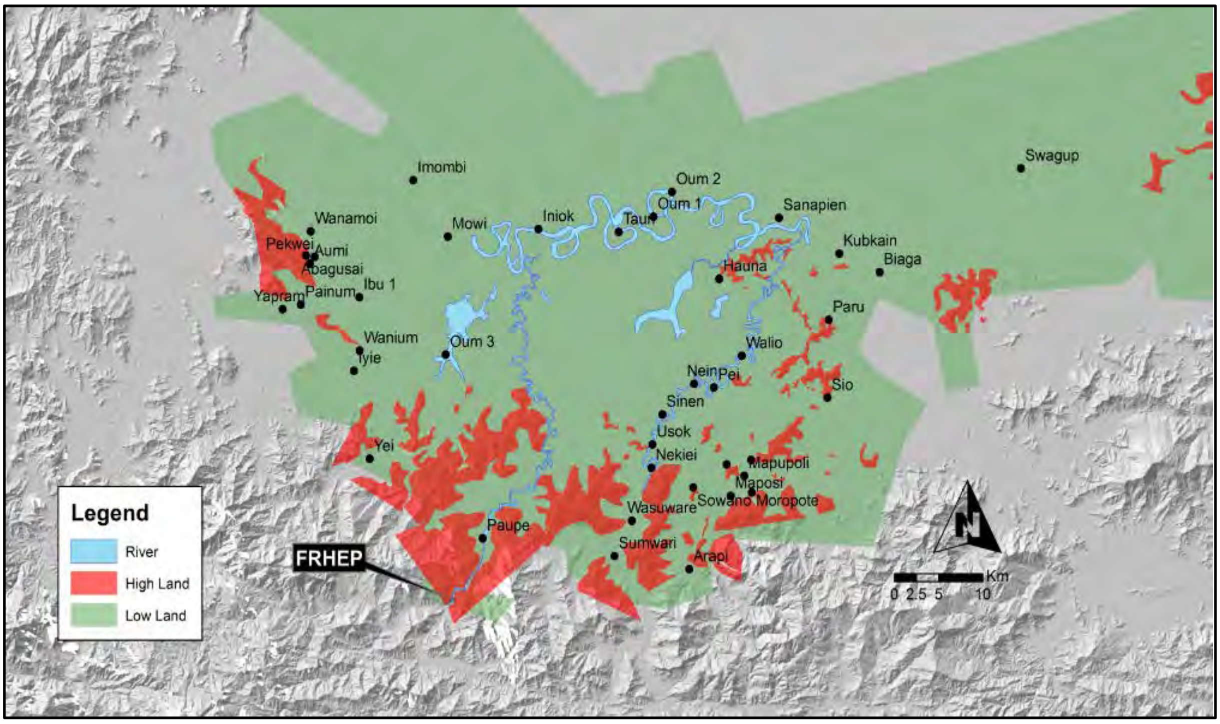



**Figure 1-3: Modelled surface roughness**

## 1.3 River flows

The runoff model does not consider any river flows in addition to the dam breach flood. As the assumed dam breach flows are multiple orders of magnitude larger than natural flows in the Frieda River (Section 1.5), it is not expected that the natural flows will have a significant impact on the results of this study. The impact of the Sepik River flows is discussed in Section 2.3.

[2] US Army Corp of Engineers 1990, HEC-1, Flood Hydrograph Package User's Manual, Davis, CA
[3] Van Te Chow 1959, Open-channel hydraulics, New York, McGraw- Hill Book Co

More detailed analysis should include a regional runoff model to estimate the flood peaks and extents in the lower catchments. This is particularly true in the case of a PMF event where the villages are likely to be flooded in the natural flood scenario, i.e. without a dam breach.

## 1.4 Modelling scenarios and hydrograph estimation

### 1.4.1 Model scenarios

Two dam break scenarios have been modelled to represent 'Sunny Day' and 'Rainy Day' cases as recommended by ICOLD 1998. A dam break induced by the PMF event is the case which will result in the highest mobilisation of storage volume. The model disregards solids flow such as tailings, waste and natural sediments as the majority of the contents of the reservoir is water. Modelling the outflow as water (rather than mudflow) generally results in conservatively short estimates of flood arrival times.

A third outflow scenario was additionally modelled to assess the impacts of a Sunny Day spillway gate malfunction.

The three scenarios are described below and summarised in Table 1-4.

**Case 1 (Rainy Day Dam Break):** The embankment has been constructed to its final height and is operated at crest level. Inflow from a PMF results in overtopping and subsequent failure of the embankment.

Although pre-existing river flows have not been considered in this analysis (as discussed in Section 6.3), it is likely that the downstream catchment is also being subjected to significant flooding. Some villages are likely to be flooded without a dam breach. The results of Case 1 are compared to the inundation extents of a PMF prior to construction of the FRHEP to assess the incremental consequences of a dam break.

**Case 2 (Sunny Day Dam Break):** The embankment has been constructed to its final height and is operated at the maximum operating level. The embankment fails due to a reason other than overtopping (i.e. slope failure, piping and/or a seismic event).

**Case 3 (Sunny Day Spillway Gate Malfunction):** The embankment has been constructed to its final height and is operated at the maximum operating level. The spillway gates inadvertently open, releasing the 13.7 m of water stored above the spillway crest level (RL 212.4 m). The embankment does not fail in this scenario.

**Table 1-4: Summary of modelling scenarios**

| Model | Description | Stored volume ($Mm^3$) | Flood volume ($Mm^3$) | Release volume ($Mm^3$) |
|---|---|---|---|---|
| Case 1 (Rainy Day Dam Break) | Maximum storage (RL 235 m) + PMF | 10,761 | 703 | 11,464 |
| Case 2 (Sunny Day Dam Break) | Storage at maximum operating level (226.1 m RL) | 9,640 | 0 | 9,640 |
| Case 3 (Sunny Day Spillway Gate Malfunction) | Incremental storage between spillway crest (RL 212.4 m) and maximum operating level (RL 226.1 m) | 9,640 | 0 | 1,607 |

### 1.4.2 Release volumes

The volume of material released or outflow volume of tailings/ water in the event of a dam break has a large influence on the extents of the downstream inundation. Literature reviews of historical tailings storage facility (TSF) failures indicate that between 1% and 100% of the stored volume can be released, with average release volumes being in the order of 20% - 40%. In water dams with a breach spanning the full height of the embankment, 100% of stored volume is released. SRK has applied a conservative approach to modelling the FRHEP as a water dam that results in a release volume of 100% of the stored volume in the case of a hypothetical embankment failure.

## 1.5 Outflow hydrographs

### 1.5.1 Dam breach

A dam breach was simulated using the US National Weather Service's BREACH code to predict the breach parameters (breach dimensions, shape and time of failure) and outflow hydrograph for Cases 1 and 2.

The breach outflow hydrographs developed are shown in Figure 1-4 and maximum peak flows are summarised in Table 1-5. It is noted that these flows exceed the range of previous experience elsewhere and should be considered indicative only. Although water is released from the time when breaching occurs, and then only increases significantly once the large erosion of the embankment occurs, it can not be seen from the graph due to the scale.

**Table 1-5: Breach hydrograph maximum peak flows by scenario**

| Scenario | Maximum peak flow ($m^3/s$) |
|---|---|
| Case 1 (Rainy Day Dam Break) | 790,000 |
| Case 2 (Sunny Day Dam Break) | 660,000 |

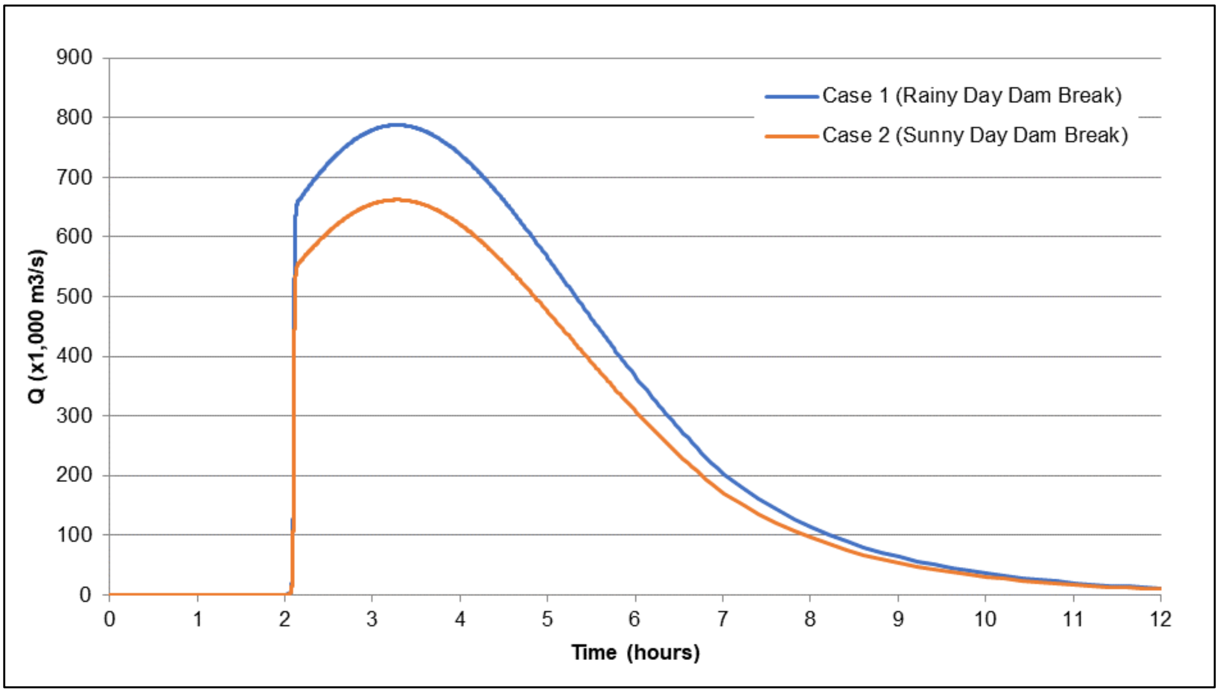



**Figure 1-4: Dam break breach hydrographs**

The calculated peak flows are in the order of 20 - 30 times greater than the PMF, and multiple orders of magnitude greater than any previous flows through the river channel. Therefore, in the event of a dam break, the flood wave will cut away the slopes on both sides of the valley, which will likely reduce flow energy and inundation depths, and increase the volume of sediment transport downstream. This phenomenon has not been accounted for in the modelling as a sediment transport analysis is beyond the scope of this work. Reducing flow energies to account for valley cutting would result in less conservative estimates of breach flows, arrival times, and extents of maximum inundation.

### 1.5.2 Spillway gate malfunction

The outflow proceeding a spillway gate malfunction will be controlled by the flow capacity of the spillway. As such, the outflow hydrograph has been estimated assuming the reservoir is operated at the maximum operating level (RL 226.1 m), the gates instantaneously open and the outflow controlled by an ungated spillway rating curve with an ogee crest at RL 212.4 m. A constant inflow of an average flow of 225 $m^3/s$ has been assumed to flow into the reservoir. The resulting outflow hydrograph is shown in Figure 1-5, indicating a peak discharge of 2,900 $m^3/s$ and a period of approximately 30 days until the reservoir is emptied to the spillway crest level.

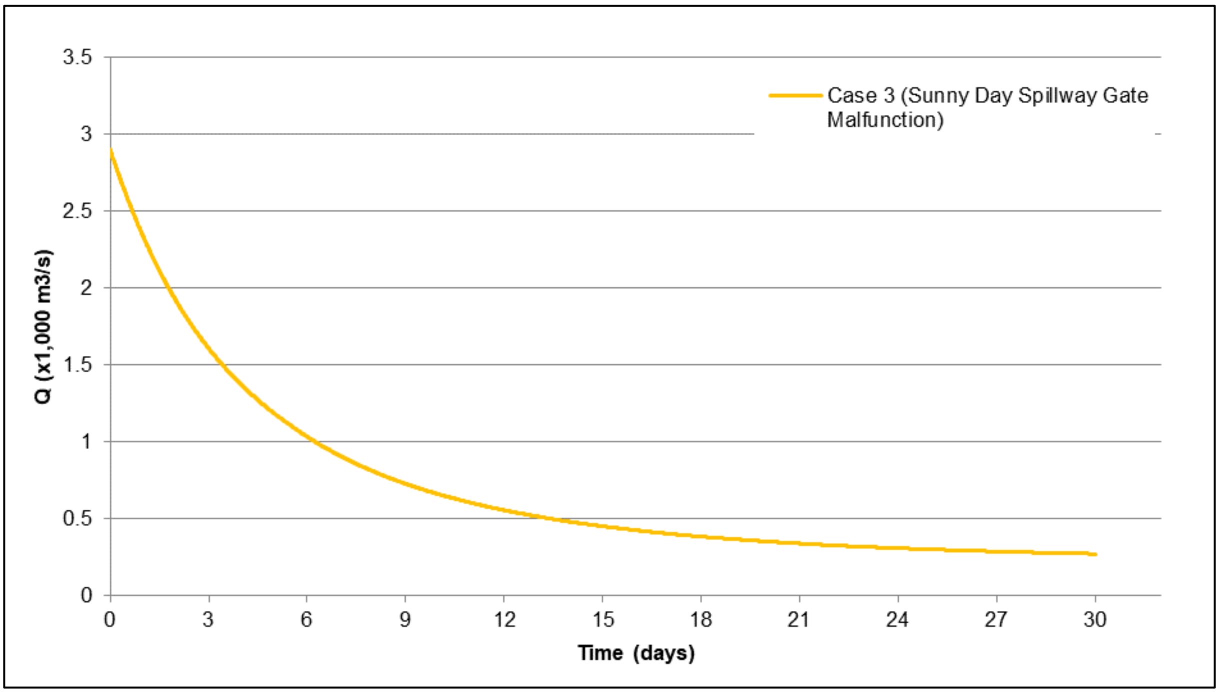



**Figure 1-5: Case 3 (Sunny Day Spillway Gate Malfunction) outflow hydrograph**

# 2 Results

The results of the dam breach modelling and analysis are summarised in the sections that follow.

## 2.1 Inundation mapping

Inundation maps were developed for each of the modelling scenarios, as shown in Figure 2-1 to Figure 2-3. Detailed and higher resolution results illustrating maximum flood depths, peak velocities and time to arrival (0.3 m flood depth) for each of the three scenarios modelled are provided in Appendix A.

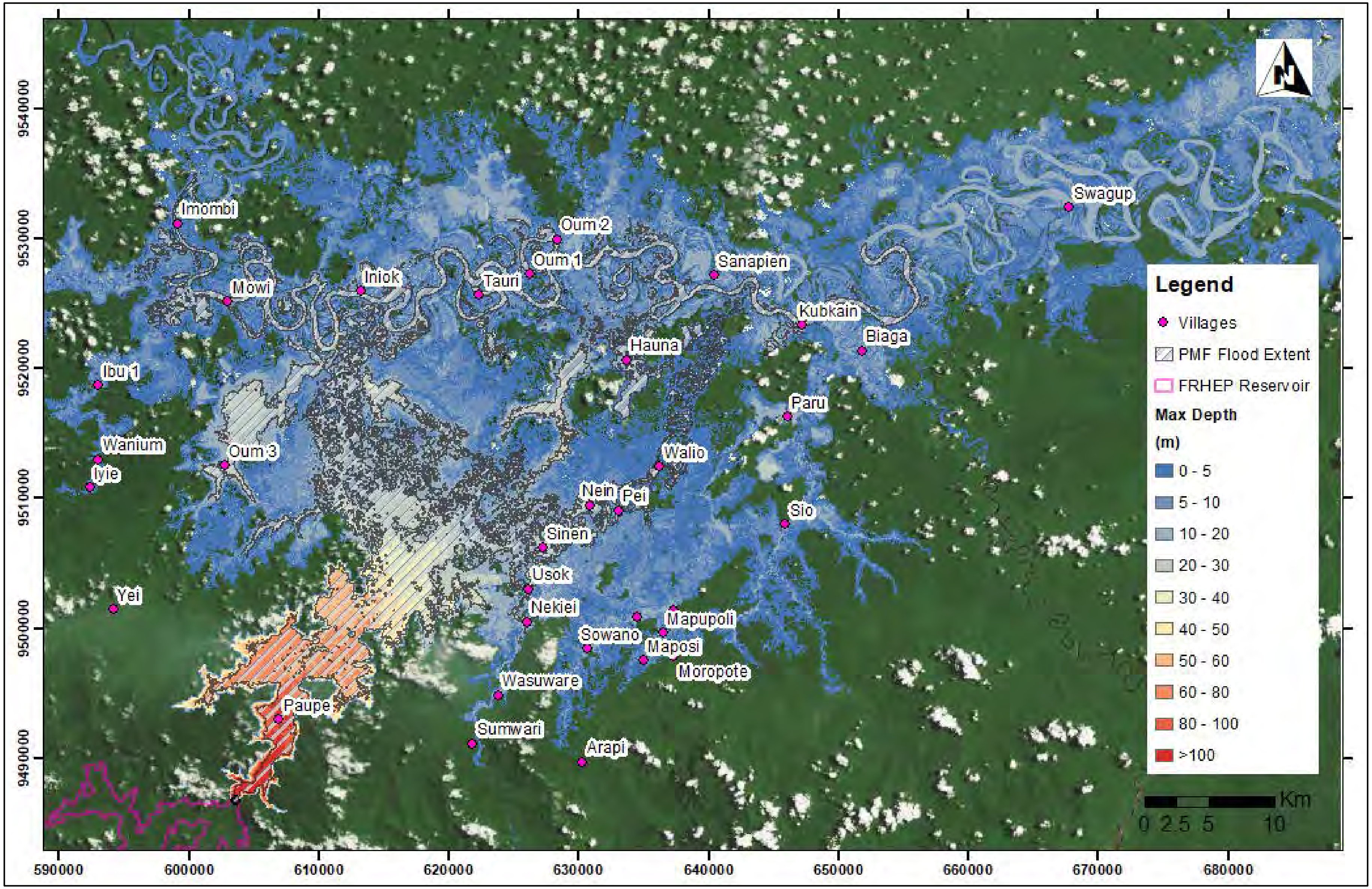



**Figure 2-1: Inundation extents – Case 1**

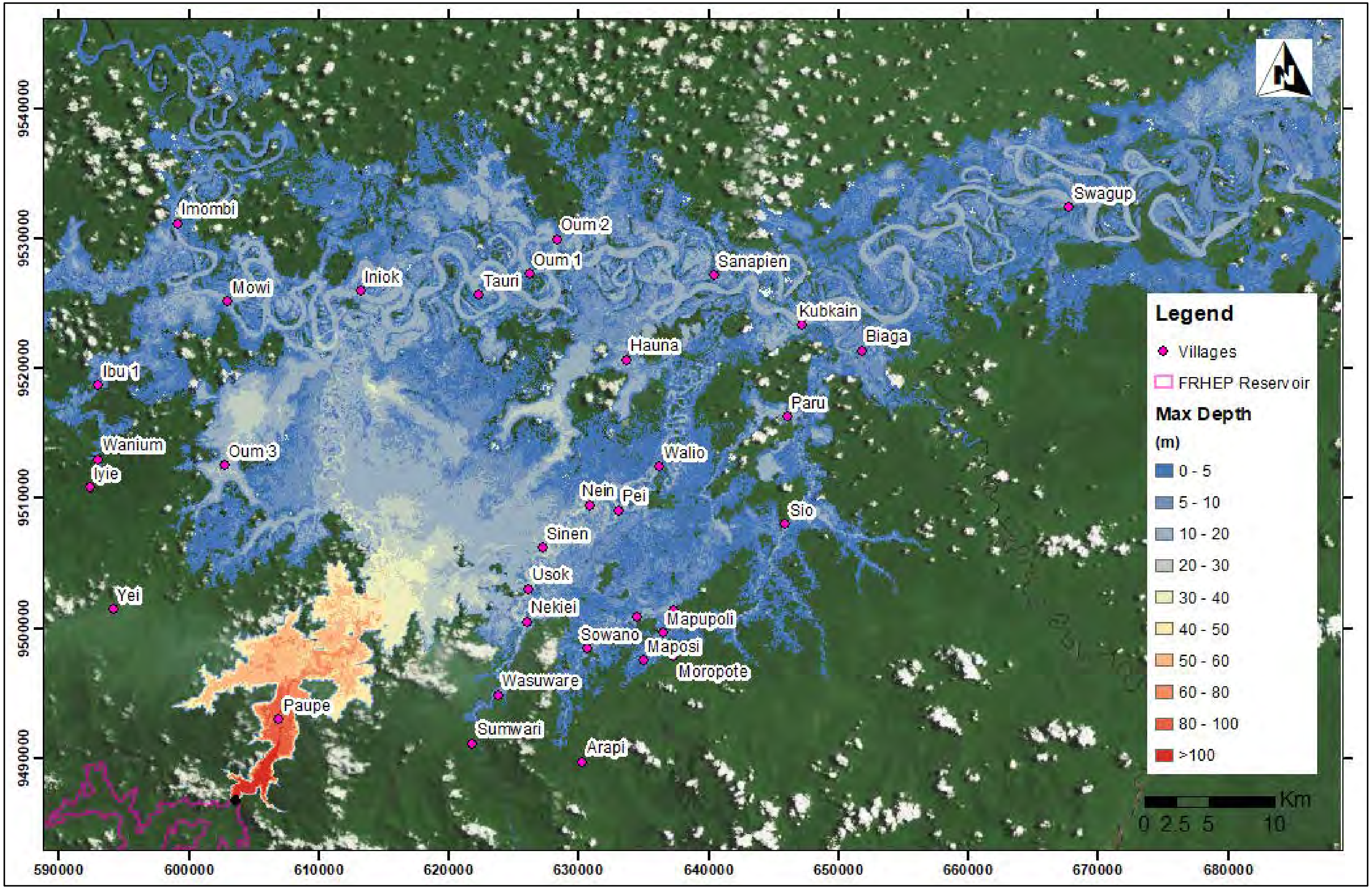



**Figure 2-2: Inundation extents – Case 2**

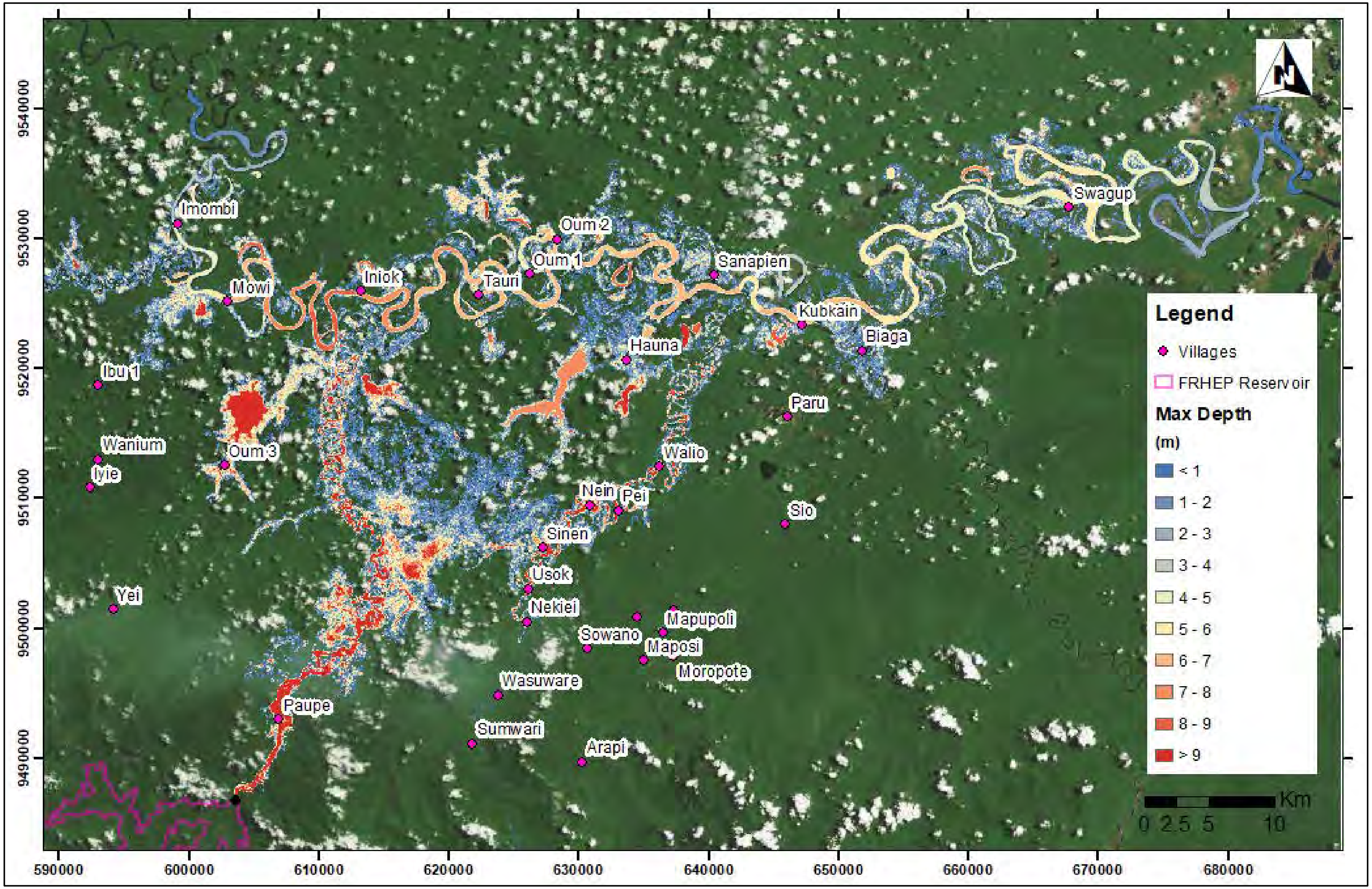



**Figure 2-3: Inundation extents – Case 3**

## 2.2 Flood arrival times and depths

Case 1 (Rainy day dam break) has shortest arrival times and highest maximum depths in comparison to the other scenarios. The Case 1 results for each village are summarised in Table 2-1(in order of arrival time). It is estimated that 10 of the villages flooded by a potential Rainy Day dam break would also be affected during a PMF event without a dam break. As such, the incremental consequence of a Rainy Day dam break is the flooding of 23 villages.

While the impacts of Case 2 (Sunny Day Dam Break) are less severe than Case 1, the incremental impacts of the dam break are more severe (32 villages are flooded).

The Case 3 (Sunny Day Spillway Gate Malfunction) scenario indicates that 12 villages are likely to be affected by an inadvertent opening of the spillway gates.

**Table 2-1: Results (Case 1)**

| Village | Time to arrival >0.3 m depth[4] (hrs) | Maximum flood depth (m) | Peak velocity (m/s) | Depth during PMF (without dam break) (m) |
|---|---|---|---|---|
| Paupe | 2 | 110 | 12 | 28 |
| Usok | 6 | 22 | 0.7 | 6 |
| Nekiei | 7 | 10 | 0.6 | 0 |
| Sinen | 7 | 20 | 1.2 | 5 |
| Nein | 9 | 20 | 0.8 | 10 |
| Oum 3 | 10 | 18 | 0.3 | 1 |
| Pei | 10 | 19 | 0.8 | 9 |
| Yabatauwe | 11 | 6.2 | 1.0 | 0 |
| Iniok | 12 | 11 | 0.3 | 0 |
| Wakiawei | 12 | 14 | 0.3 | 0 |
| Walio | 12 | 14 | 0.8 | 6 |
| Hauna | 13 | 9 | 0.7 | 0.1 |
| Mapupoli | 13 | 13 | 0.2 | 0 |
| Tauri | 16 | 15 | 0.3 | 4 |
| Maposi | 16 | 8.1 | 0.3 | 0 |
| Mowi | 17 | 7.3 | 0.7 | 0 |
| Moropote | 17 | 8.5 | 0.1 | 0 |
| Oum 2 | 18 | 14 | 0.4 | 3 |
| Sio | 20 | 6.5 | 0.5 | 0 |
| Paru | 22 | 9.9 | 0.3 | 0 |
| Sanapien | 23 | 7.7 | 0.3 | 0 |
| Oum 1 | 24 | 4.4 | 0.3 | 0 |
| Imombi | 25 | 5.6 | 0.2 | 0 |
| Ibu 1 | 30 | 8.2 | 0.3 | 0 |
| Biaga | 36 | 11 | 0.3 | 0 |
| Wanamoi | 39 | 3.3 | 0.3 | 0 |
| Pekwei | 41 | 7.9 | 0.3 | 0 |
| Aumi | 43 | 5.7 | 0.4 | 0 |
| Kubkain | 44 | 1.4 | 0.4 | 0 |
| Wanium | 57 | 2.3 | 0.2 | 0 |
| Iyie | 73 | 3.1 | 0.1 | 0 |
| Painum | 74 | 1.7 | 0.1 | 0 |
| Swagup | 82 | 2.6 | 0.3 | 0 |

[4] The flood arrival times shown commence from the initiation of overtopping. Embankment failure is estimated to occur approximately 2 hours after the initiation of overtopping (as shown in Figure 1-4). As such, the time to arrival from embankment failure is approximately 2 hours less than the times shown.

## 2.3 Sediment transport and deposition

Sediment transport analysis and deposition modelling of tailings and waste rock is beyond the scope of this report; however, it should be noted that reactive tailings and waste will likely be transported and deposited in locations further downstream, posing a long-term risk. In the case of the recent incident at the Samarco operation in Brazil, tailings were transported a distance of 650 km to the ocean. The FRHEP is substantially larger than the Samarco TSF and has significantly higher potential release volumes and peak flows; it therefore can be expected that some of the tailings would similarly flow to the ocean.

Although the breach flood will be diluted once it reaches the Sepik River, suspended solids will still be transported in the river and deposited along the river banks. The reactive tailings and waste, both in the river and deposited across the inundation extents, will represent a very significant long-term source of contamination throughout the region.

# 3 Summary and conclusions

A HEC-RAS 5.0.3 model was developed to assess the possible consequences of a hypothetical breach of the FRHEP embankment. The results of the modelling were used to create inundation maps and predict flood wave arrival times, depths and maximum flow velocities. Based on the results of the analysis, SRK concludes as follows:

- Potential breach volumes have been estimated based on historic facility failures and result in volumes of material (tailings, waste, and water) being released in the order of 10,000 $Mm^3$ or 100% of stored volume.
- Within the extents of the modelling, dam breach flows are expected to impact 33 villages downstream. The most critical village affected is Paupe, which has a warning time of 2 hours from initiation of the breach or almost instantaneously after catastrophic failure. The remaining villages will have a minimum of 6 hours before the breach flood arrives. All villages are expected to be impacted within 4 days of the initial breach.
- The flood wave is predicted to reach the Sepik River approximately 9 hours after the breach is initiated, and the wave height is expected to affect river traffic for some distance downstream on the Sepik River.
- A sudden opening of the spillway gates will affect at least 12 of the villages. The flood wave of at least 0.3 m is predicted to reach Paupe approximately 3 - 4 hours after the gates open, and the expected maximum water depth is expected to reach 14 m.

While a sediment transport analysis has not been undertaken, it should be noted that reactive tailings/ waste will be transported and deposited far downstream of the breached dam, posing a very significant long-term source of contamination throughout the region.

## Compiled by


**Claude Prinsloo**

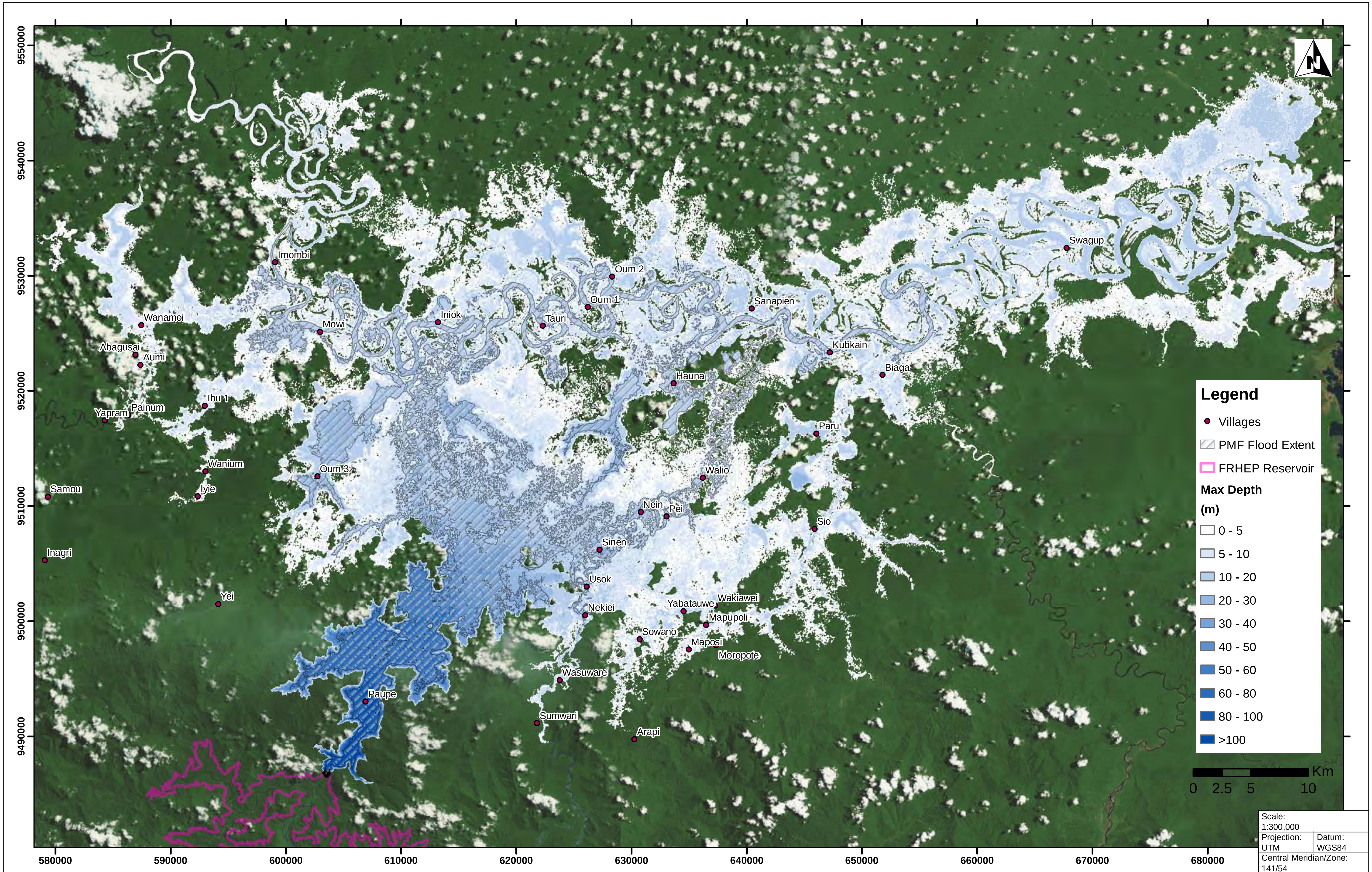
Senior Consultant

## Peer Reviewed by


**Pepe Moreno**

Principal Consultant

# Appendix A: Modelling Scenarios

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**Frieda River Hydroelectric Project - Dam Break Analysis**

**Case 1 (Rainy Day Dam Break) - Maximum Depth**

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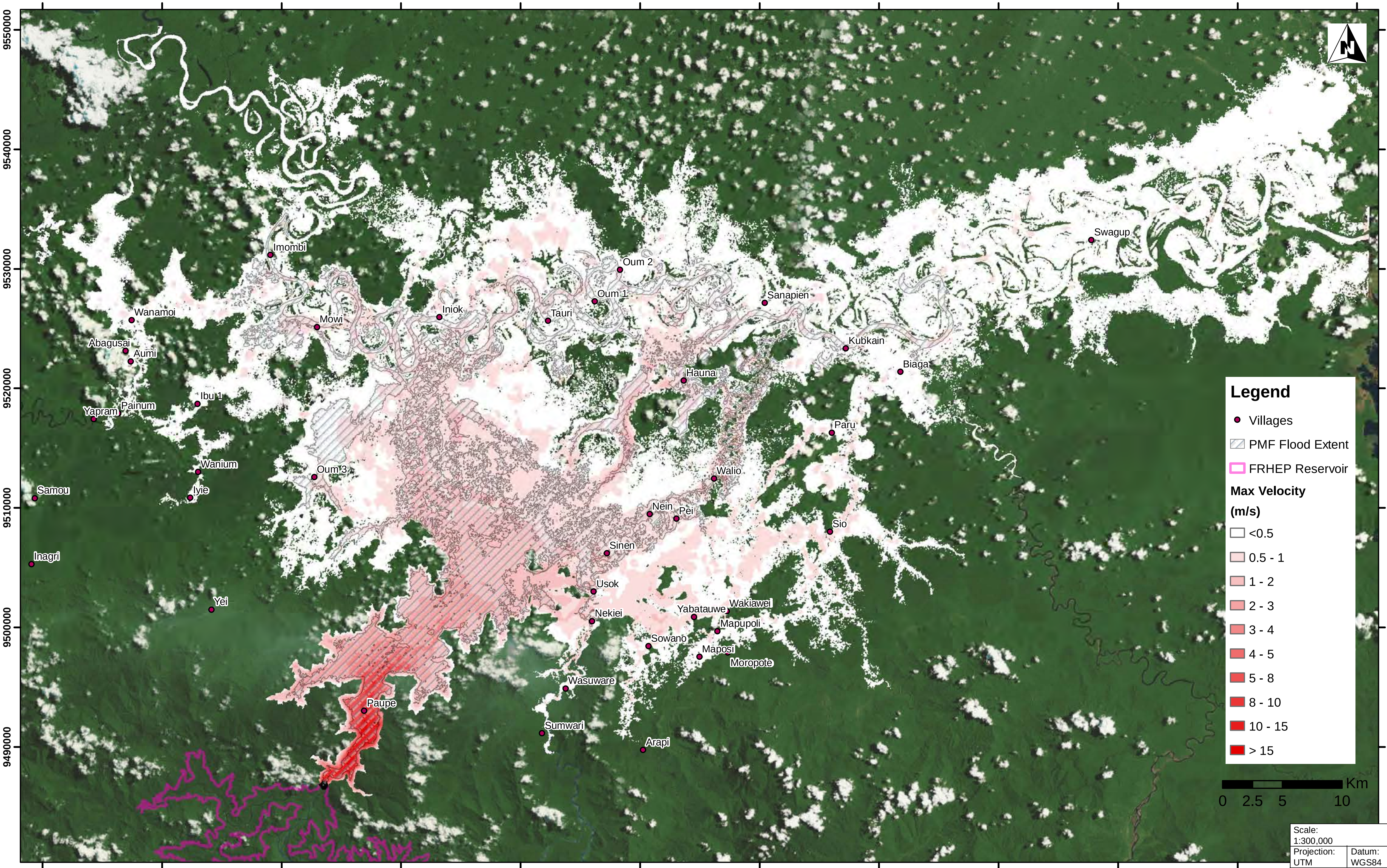


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**Frieda River Hydroelectric Project - Dam Break Analysis**

**Case 1 (Rainy Day Dam Break) - Maximum Velocity**

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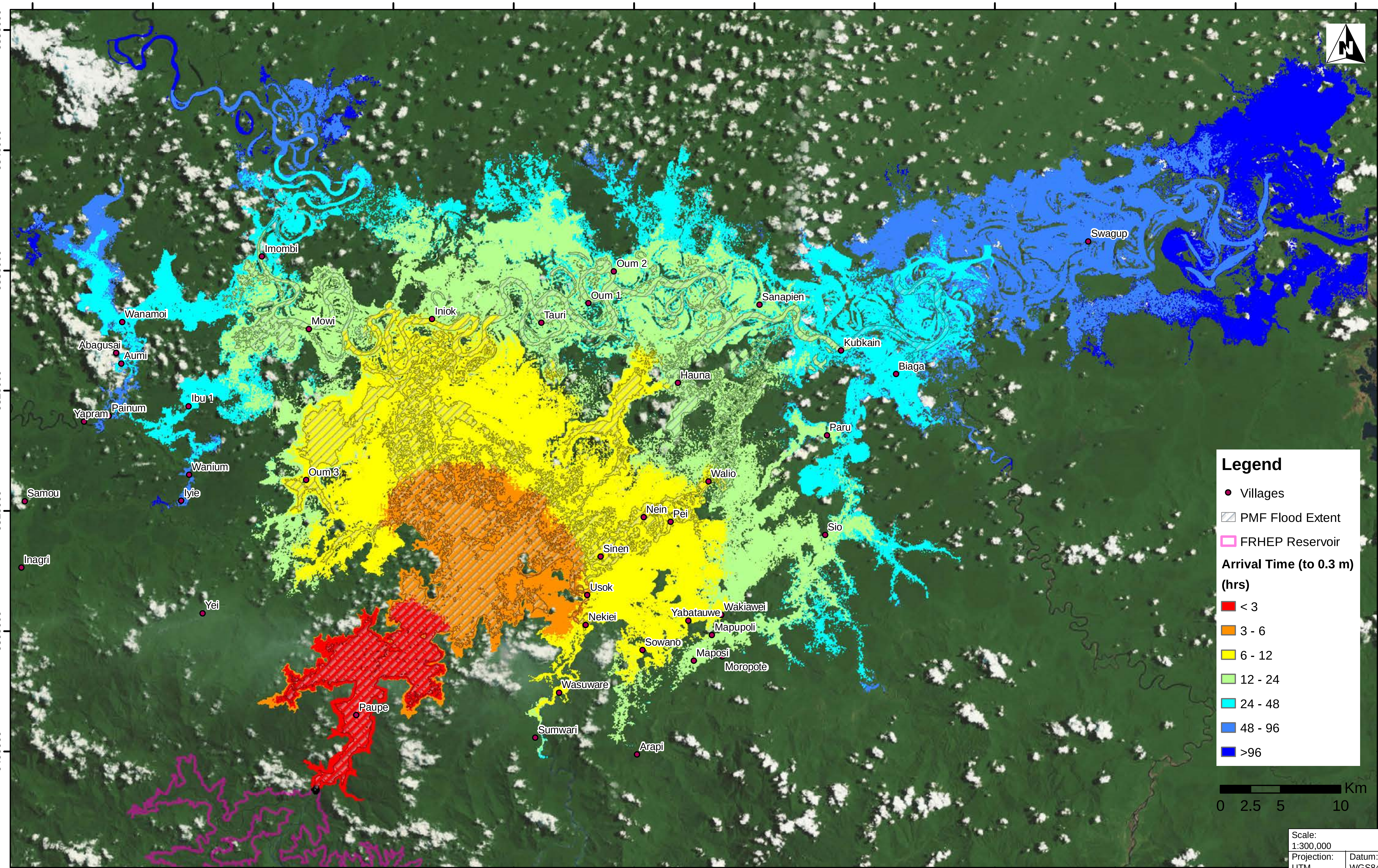


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# Frieda River Hydroelectric Project - Dam Break Analysis
## Case 1 (Rainy Day Dam Break) - Arrival Time

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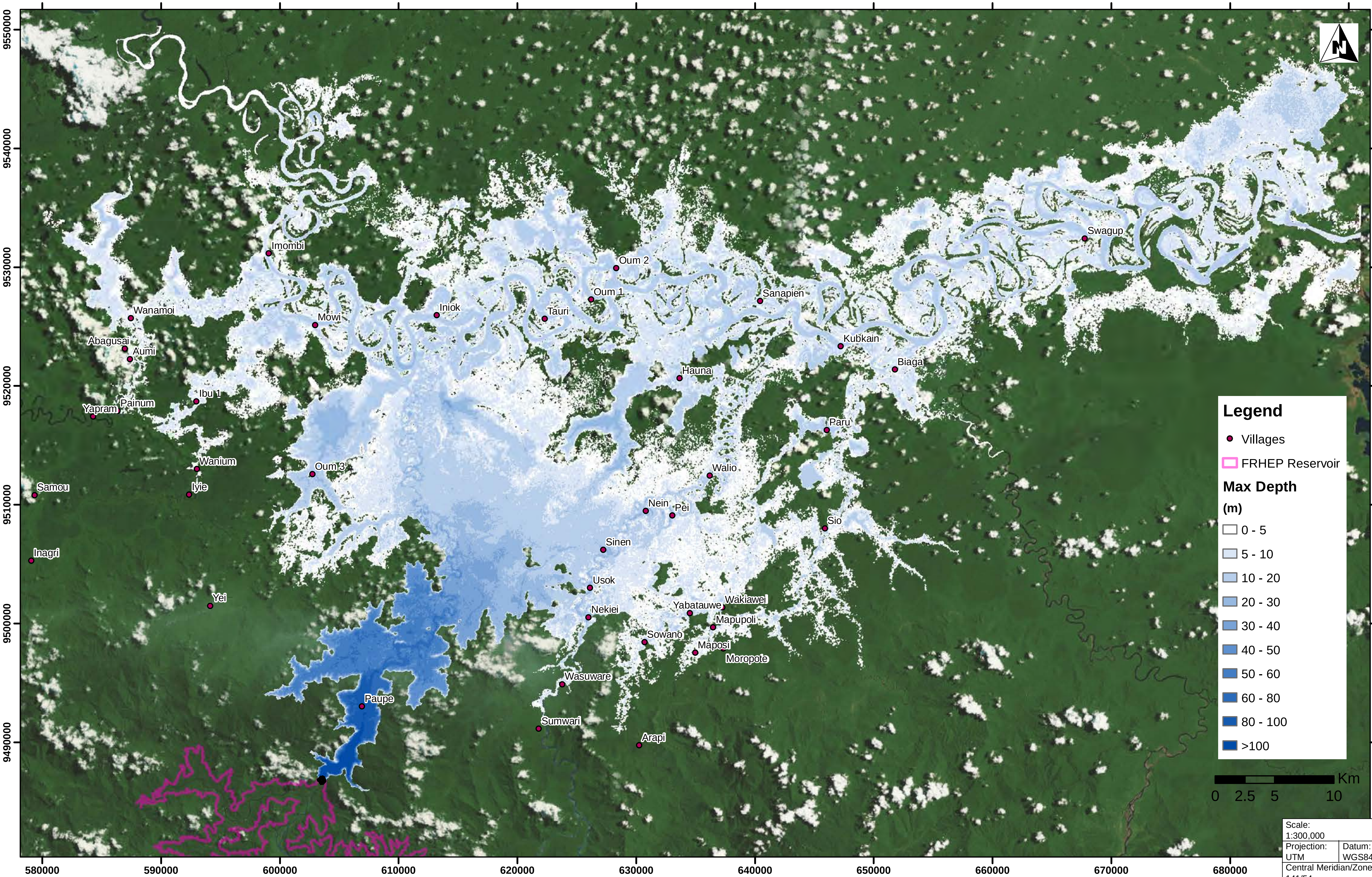


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**Frieda River Hydroelectric Project - Dam Break Analysis**
**Case 2 (Sunny Day Dam Break) - Maximum Depth**

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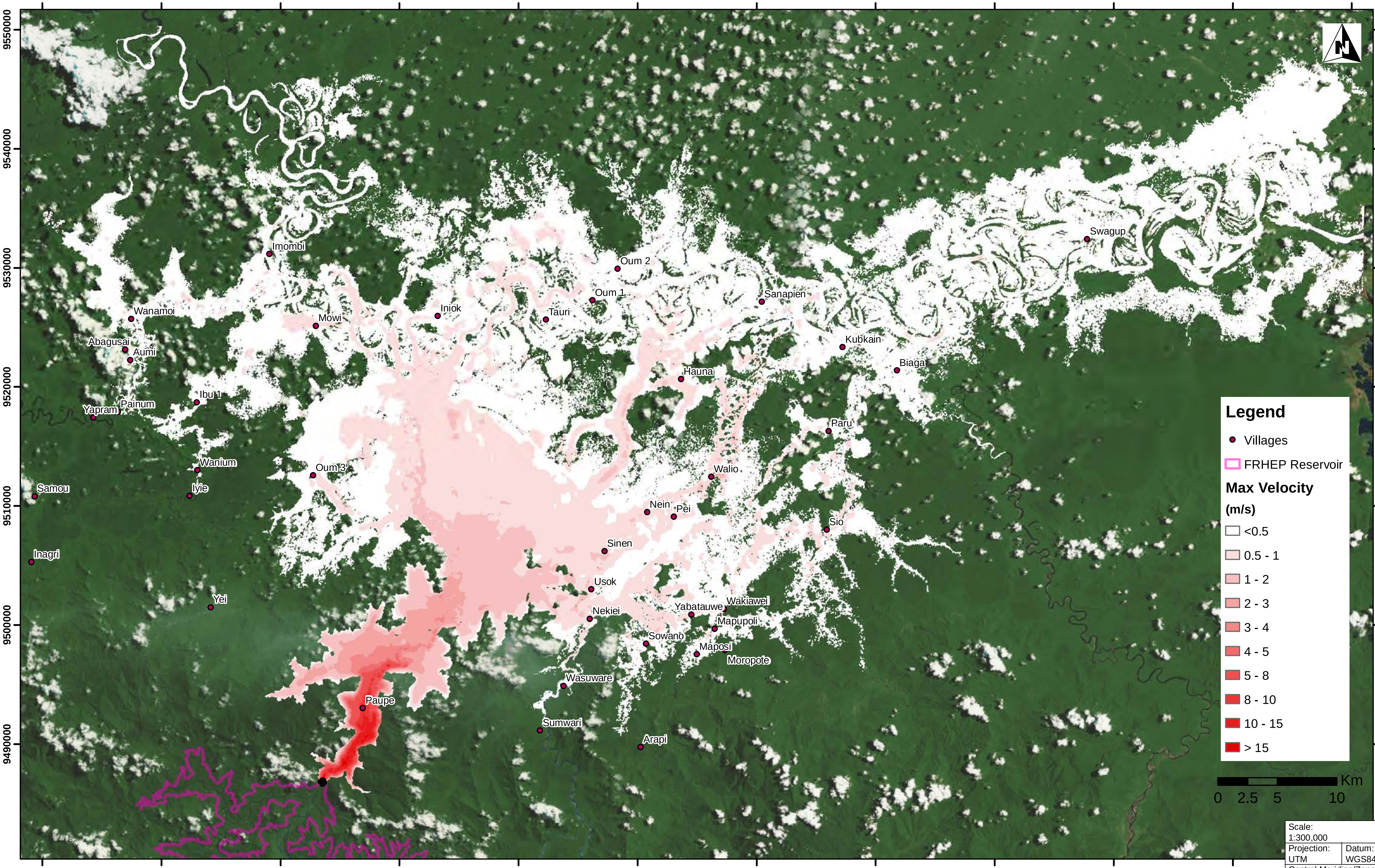


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# Frieda River Hydroelectric Project - Dam Break Analysis
## Case 2 (Sunny Day Dam Break) - Maximum Velocity

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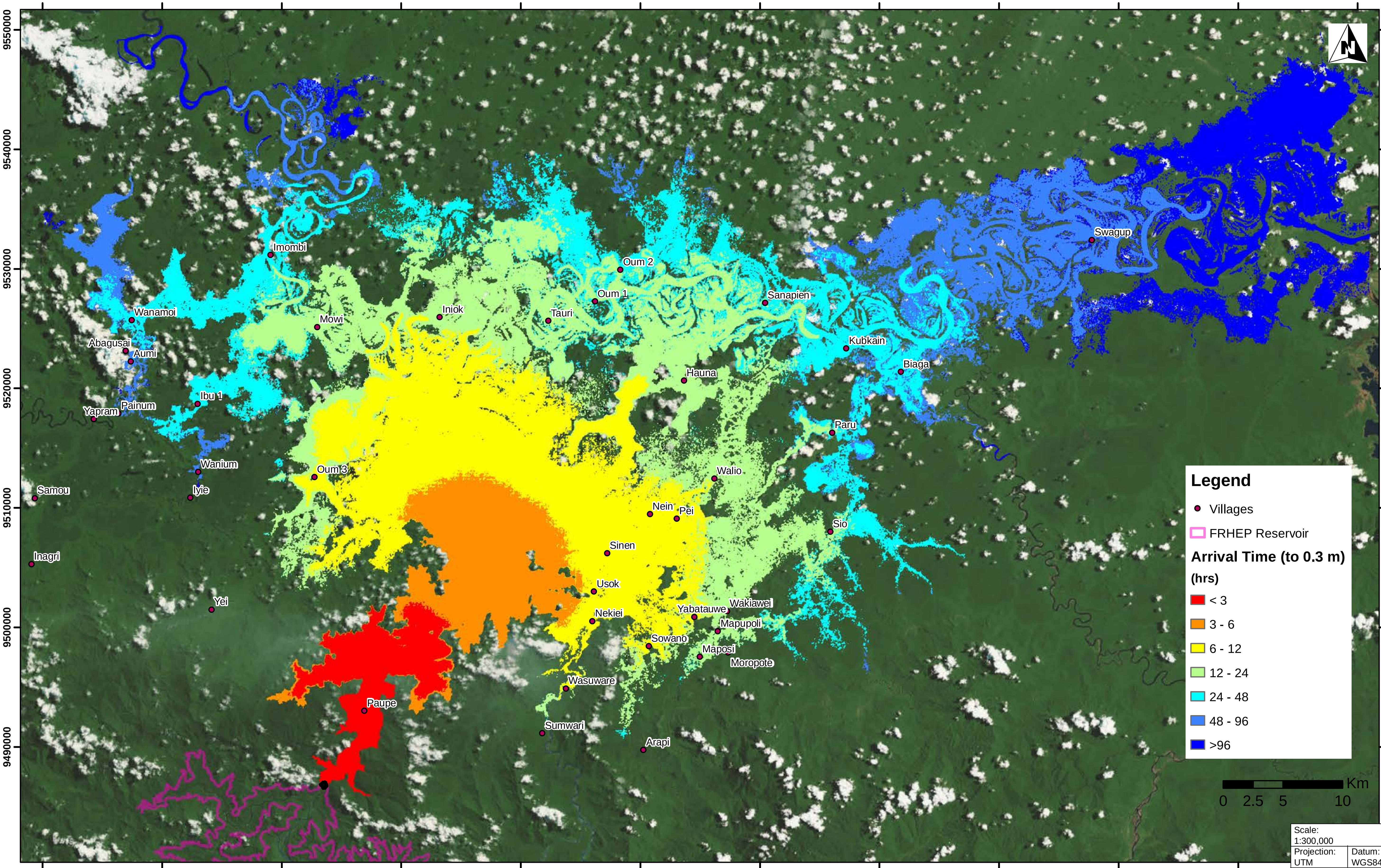


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| Date: 13/03/18 | Compiled by: HTHOM |
| Project No: PNA009 | Fig No: |

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**Frieda River Hydroelectric Project - Dam Break Analysis**
**Case 2 (Sunny Day Dam Break) - Arrival Time**

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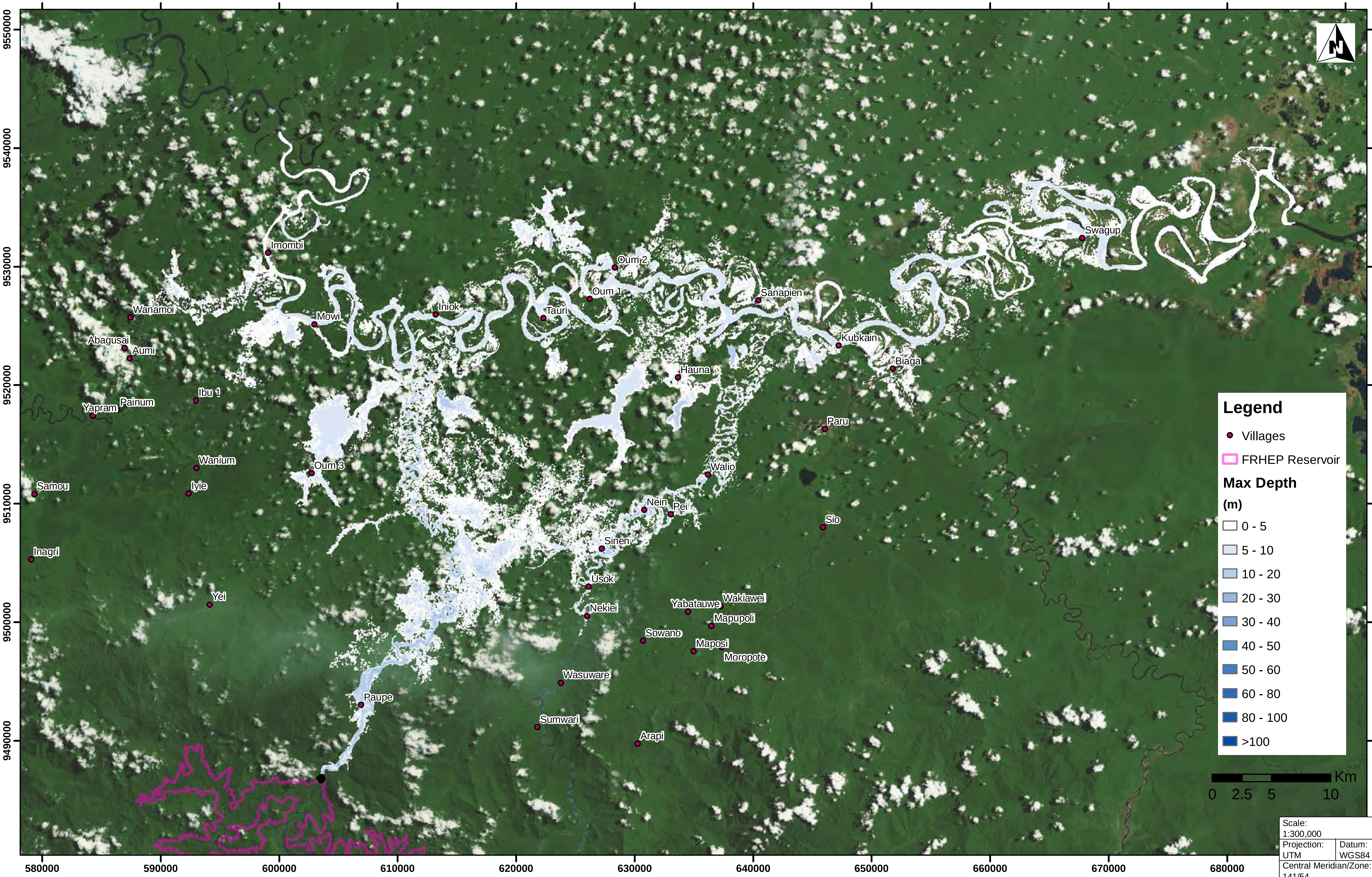
Revision: B

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IF THE ABOVE BAR DOES NOT SCALE 25mm, THE DRAWING SCALE IS ALTERED

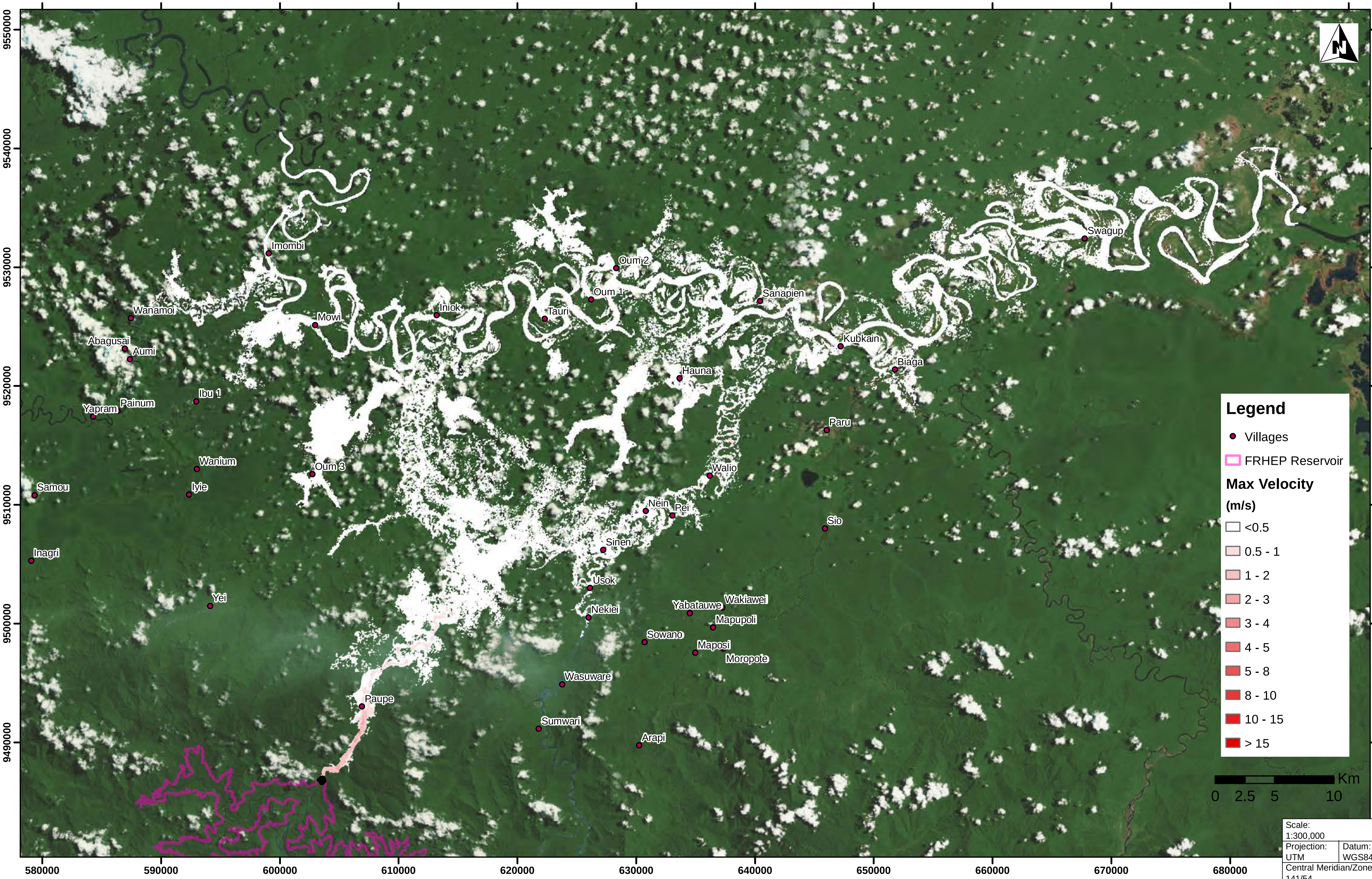
**Frieda River Hydroelectric Project - Dam Break Analysis**

**Case 3 (Sunny Day Spillway Gate Malfunction) - Maximum Depth**

| Scale: 1:300,000 | |
|---|---|
| Projection: UTM | Datum: WGS84 |
| Central Meridian/Zone: 141/54 | |
| Date: 13/03/18 | Compiled by: HTHOM |
| Project No: PNA009 | Fig No: |

Path: P:\PNA009 - Frieda River HEP PFS\04_Working_Files_\10_Dam break assessment\Results\PNA009_Result Map_Rev B.mxd

Revision: B




| Scale: 1:300,000 | |
|---|---|
| Projection: UTM | Datum: WGS84 |
| Central Meridian/Zone: 141/54 | |
| Date: 13/03/18 | Compiled by: HTHOM |
| Project No: PNA009 | Fig No: |

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IF THE ABOVE BAR DOES NOT SCALE 25mm, THE DRAWING SCALE IS ALTERED

## Frieda River Hydroelectric Project - Dam Break Analysis
## Case 3 (Sunny Day Spillway Gate Malfunction) - Maximum Velocity

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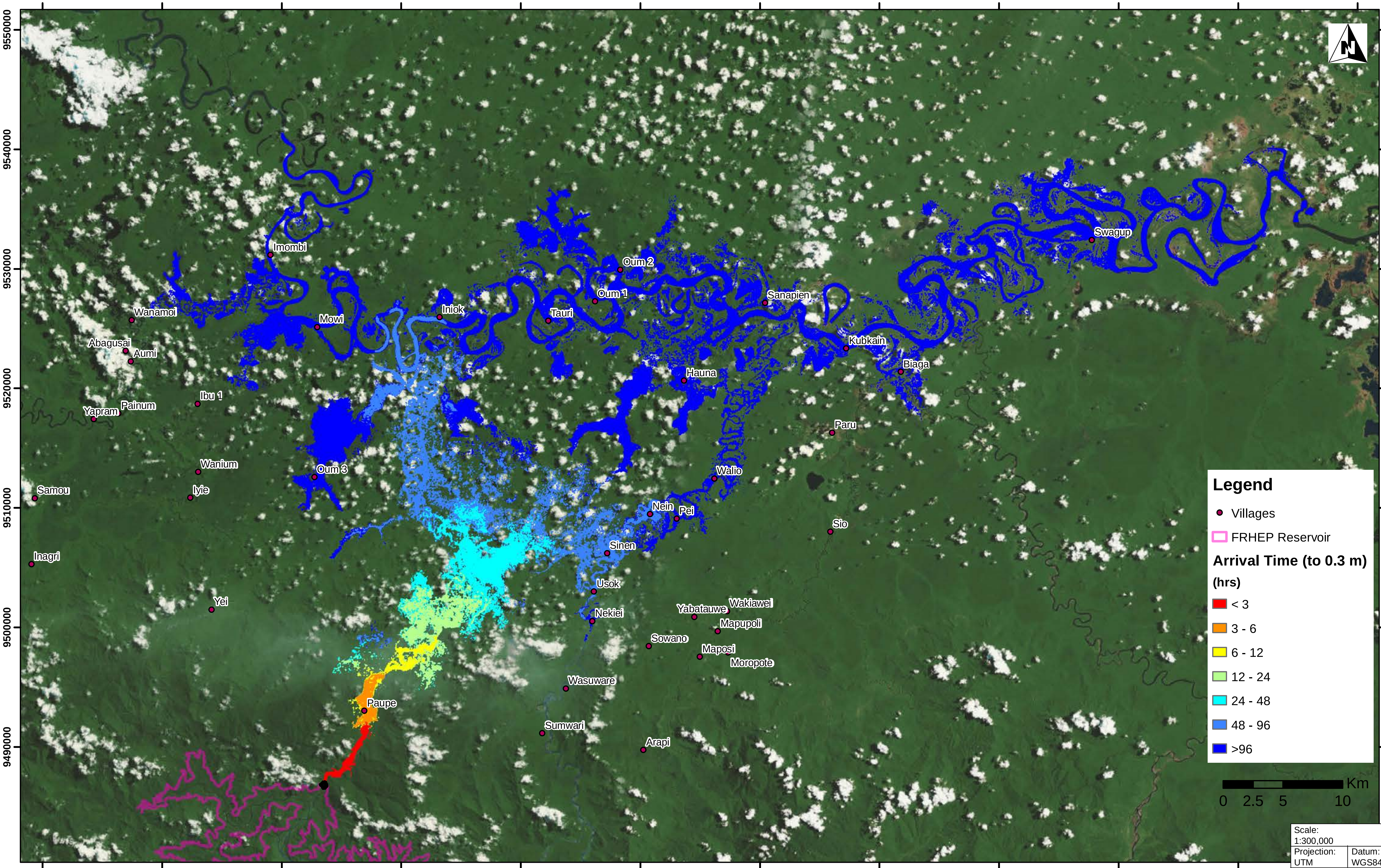
Revision: B




| | |
|---|---|
| Scale: 1:300,000 | |
| Projection: UTM | Datum: WGS84 |
| Central Meridian/Zone: 141/54 | |
| Date: 13/03/18 | Compiled by: HTHOM |
| Project No: PNA009 | Fig No: |

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IF THE ABOVE BAR DOES NOT SCALE 25mm, THE DRAWING SCALE IS ALTERED

# Frieda River Hydroelectric Project - Dam Break Analysis
## Case 3 (Sunny Day Spillway Gate Malfunction) - Arrival Time

Path: P:\PNA009 - Frieda River HEP PFS\04_Working_Files_\10_Dam break assessment\Results\PNA009_Result Map_Rev B.mxd

Revision: B