Moawhitu Wetland hydrological restoration: inundation scenarios, control structure and peak flow assessment

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Collaborations

Executive Summary

Department of Conservation (DOC) are considering the installation of an outlet control structure at Moawhitu Wetland on D'Urville Island. The structure would have the ability to adaptively control water levels over a large area, allowing for successive water level raises to promote hydrological and ecological restoration of the wetland, while also ensuring levels can be managed to maintain infrastructure.

Following a site visit in February 2020, new survey points collected from the wetland and outlet channel were integrated with an existing digital elevation model (DEM) of the site. The updated DEM provides a detailed overview of ground elevations, which has been used to create water level inundation and depth contour maps for 3.0, 3.3, 3.6, 3.8 and 4.0 mRL.

The survey data was also utilised by an engineering subcontractor whom developed a concept outlet control structure for the wetland inclusive of drawings, construction methodology and a cost estimate. DOC and Collaborations had ongoing involvement with the concept design through workshops, provision of sketches and indications of preferences over precast elements versus use of in-situ material.

The final design consists of an open top box culvert with concrete wing walls keyed into the channel bank. Two slat weirs will be cast into the culvert channel ~3 m apart. This will allow successive water level raises to occur while also providing a resting pool between the weirs for fish passage. The fish passage design design needed to account for water levels to be raised at least 1.2 m, and the presence of Inanga (which require ramps <15°). Two temporary rubber fish ramps will be utilised by being bolted onto the top of weir control boards.

A rainfall and peak flow assessment was undertaken for a 100 year ARI (1% AEP) 24 hour storm. The rainfall assessment utilized both long term daily records from Greville Harbour (~30 years) and Stephens Island (~120 years), and short term event based rainfall from a tipping bucket rain gauge installed within the wetland. This ~2 year record was used to assess storm rainfall patterns (hyetographs) for six of the largest and most intense storms to occur through this period.

The site hyetographs were then used to create a design storm for the wetland and lake catchment based on the 24 hour rainfall depth determined from NIWA's High Intensity Rainfall Database (HIRD's). Hydrological modelling was undertaken in HEC-HMS and this initial assessment was highly conservative, not accounting for wetland or lake storage that would significantly lower and delay the peak event.

The 100 year ARI peak flow for the critical storm at the proposed outlet control structure was ~58 m^3/s. An empirical assessment of the potential peak flow from the outlet structure during a sunny day failure (i.e. no storm event occurring simultaneously) was also undertaken for a water level height of 3.6 mRL. Three empirical methods were applied with a safety factor of 1.3. The average peak flow if the outlet structure failed was estimated at 46.8 m^3/s. This is considered conservative given flow would likely be confined within the outlet structure and channel, while the formulas were developed for failure assessments on dams over a wider crest width (and greater water level height), meaning they often predict a larger peak flow due to the volume being released over a shorter timeframe.

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1 Introduction

1.1 Background

In February 2020, Collaborations (James Blyth) and Department of Conservation (DOC) staff undertook a site visit to Moawhitu Wetland on D'Urville Island. During this visit, possible locations were considered for a permanent water level control structure on the outlet channel in addition to collection of further ground survey data. A summary of this site visit and a number of recommendations for future projects at Moawhitu can be found in Blyth (2020), which is considered to be Phase 1 of the work to date.

1.2 Purpose of report

Phase 2 (this report) expanded on some of the recommendations in Blyth (2020). The main objectives of this study were to:

- Update the existing site digital elevation model (DEM) with new survey data
- Develop five inundation scenario maps at 3.0, 3.3, 3.6, 3.8 and 4.0 mRL. Complement these maps with water level depth contours to help advise restoration goals.
- Engage an engineering firm (Orogen Limited) to develop concept drawings for an outlet water level control structure (utilising outlet channel cross sectional survey data). The outputs should:
    - Design a structure that provides for adaptive water level management
    - The structure should preferable be pre-cast and simple to transport and install on site
    - Provide some options for fish passage and a brief construction methodology
- Collate local and historical rainfall data to understand storm patterns (rainfall hyetographs) and rainfall statistics for the wetland
- Undertake a simplified peak flow assessment of the wetland and lake catchment using storm patterns from the wetland, to improve understanding of peak flows at the outlet channel..

2 Methodology

2.1 DEM update and inundation maps

During Phase 1 site visit, Davis Oglive and Partners surveyed a number of locations around the wetland to infill holes, based on Collaborations recommendations from earlier discussions. The survey points are summarised in the document 'Moawhitu Wetland Survey Feb 2020' and presented in Figure 1.

Phase 2 GIS analysis required the existing DEM (created by Jacobs in September 2018) to be updated with the new survey data (Figure 1). The process to update the existing DEM involved:

- Filtering both the previous survey data (Davis Oglive and Partners 2018) and new survey data to include only relevant ground level points.
- The previous break-line layer was edited to include additional lines by connecting matching survey points (i.e. top of bank). Further break-lines were added to help guide the DEM generation, particularly around the channel and wetland boundary.

- The above layers were used to create a TIN (Triangular irregular network)
- A wetland boundary polygon was used to clip the TIN
- The TIN was then converted to a raster DEM with a cell size of 1 m

Once the DEM was checked and finalised, inundation extent and depth layers were created by subtracting the DEM from five water level scenarios, 3 m, 3.3 m, 3.6 m, 3.8 and 4.0 mRL (Appendix 1)

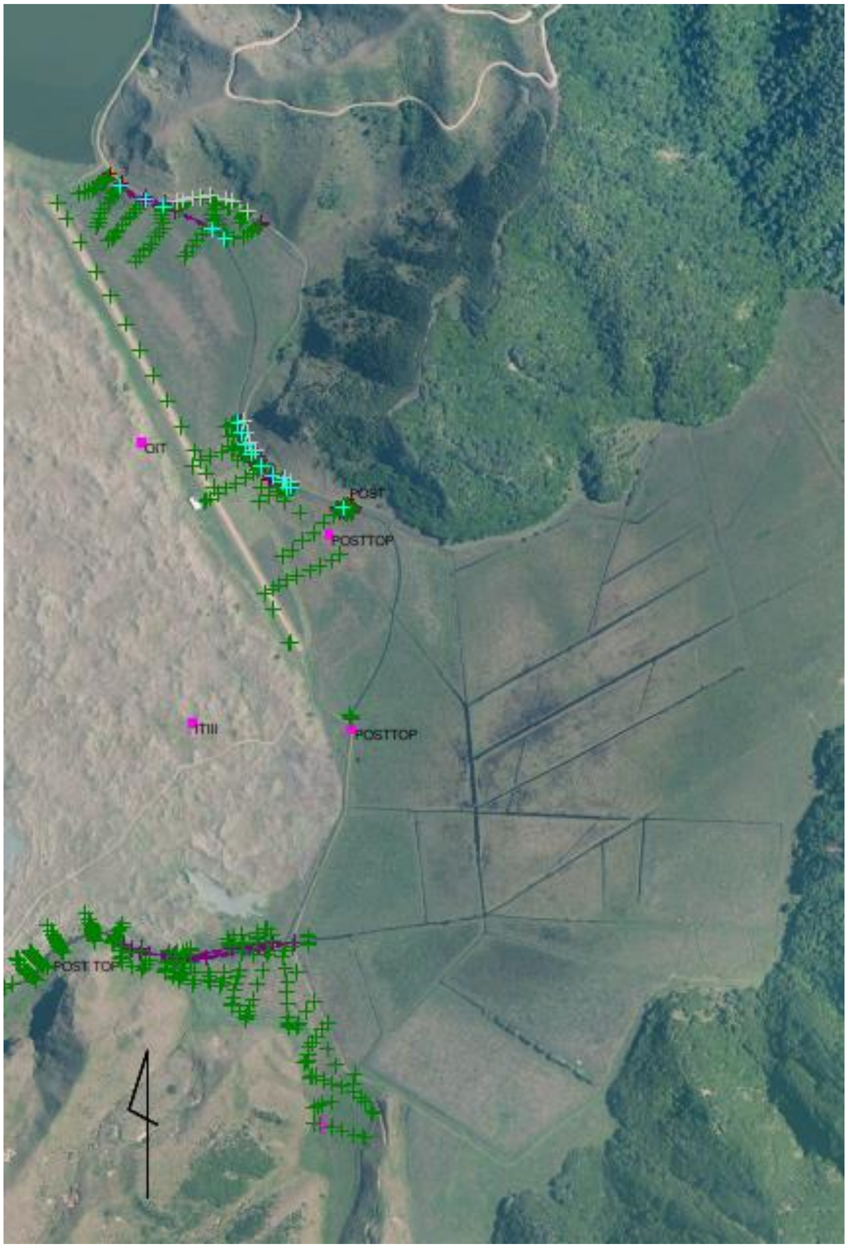

Figure 1. Feb 2020 survey points (Davis Oglive and Partners).

2.2 Concept Structure Design

Collaborations and DOC staff provided concept water level control structure sketches to Orogen Limited, based on observations and discussions from the Phase 1 site visit. The proposed location for the outlet control structure has been identified in Figure 2, at location 2.

Location 2 is slightly upstream from some large log jams (due to storm surges), but at a natural pinch point (with the sand dunes on the true right). This location also provides easier access for an excavator than Location 1. See Blyth 2020 for more discussion on these locations.

Figure 2. Proposed location of the outlet control structure (Blyth 2020).

2.2.1 Workshop and fish passage discussion

A 2-hour workshop was undertaken between Collaborations, DOC and Orogen Limited. The aim of this workshop was to consider the initial structure design (for example, pre-cast versus constructed from material on site) and fish passage with an adaptive water level control.

Considerations were given to a range of structures, including rock-ramp weirs, earth bunds and slat weirs. The resulting outcome of the workshop was a recommendation to proceed a concept design on:

- An open top box culvert (which can be pre-cast and transported to site, and dropped in easily)
- Concrete wing walls that can be connected to the culvert and keyed into the stream banks
- Water level control through treated timber slat boards acting as a weir
- Structure design for >50 years into the future, allowing a large range of adaptive water level increases (up to 4.0 mRL if necessary).

In situations where an increase in water level is required to restore wetland hydrology, maintaining effective fish passage is important. Where practical, a permanent rock-ramp weir would be the preferred structure to raise water levels, as described in the NZ Fish Passage Guidelines. However, it is critical at Moawhitu wetland to have a structure that allows for modification of water levels. Therefore, an alternative fish ramp/structure is required which DOC and Collaborations consider appropriate for provision of passage of target species: inanga and eel (informed by on site surveys).

DOC fish passage experts identified the use of a prefabricated rubber fish ramp (ATS Environmental 2020) that can be bolted onto the top of a weir board and could provide a suitable temporary solution, and with monitoring, has good potential to be suitable as a permanent solution. The design is shown in Appendix B. Floating ramps have been used throughout New Zealand for similar applications, often as a permanent solution to provide fish passage.

The artificial ramp for Moawhitu wetland has been designed to ensure the angle is no more 15° and a large resting pool has been incorporated between the two ramps – criteria that are consistent with the recommendations of the NZ Fish Passage Guidelines. Inanga are known to be relatively weak swimmers so require low grade ramps with opportunity for resting. Studies by Baker 2014 have shown that inanga have limited success in passing steeper grade ramps so slope has been minimised as much as possible to be no more than 15°. We recommend monitoring to ensure the fish pass is functioning effectively after installation.

Orogen Limited's concept drawings, proposed construction methodology, structure cost estimate and fish passage considerations have been presented in Appendix B.

2.3 Rainfall assessment

2.3.1 Long term rainfall data

The purpose of the rainfall assessment was to provide an overview of the meteorological conditions that are present at Moawhitu Wetland to help inform understanding of rainfall intensity in the catchment and peak flows at the outlet channel.

A number of suitable long-term rainfall records were identified from stations near the wetland. These are presented in Table 1. Greville Harbour (station G03882) had >30 years of complete daily rainfall records, however, became increasingly 'patchy' from mid-1980.

Table 1. Historical daily rainfall data available for Moawhitu Wetland

Station No	Start	End	% Complete	Location	Lat	Long
G03882	30-Jun-56	31-May-94	~90	Greville Harbour (~1 km from wetland)	-40.809	173.818
G04601	01-Feb-1894	30-Sep-19	90	Stephens Island (~23 km NE of wetland)	-40.666	174.001

The daily rainfall data series were downloaded and analysed to determine monthly and annual statistics.

A ~120 year record exists for Stephens Island, which was also correlated with Greville Harbour data for the same period. Daily rainfall data from both stations were cumulated and compared to understand how rainfall changes with distance from D'Urville Island.

A ~28 year dataset was used for the analysis, which began on 1 Jan 1958 and ended on 31 December 1985. This has been plotted along a 1:1 line. Monthly rainfall totals were also compared as a scatterplot, with a linear correlation created to determine a simplified formula for correcting monthly rainfall data from Stephens Island to what could be predicted at Durville Island (should rainfall monitoring at Greville Harbour cease in the future).

2.3.2 Local site data and storm hyetographs

A tipping bucket rain gauge was installed at Moawhitu wetland in winter 2017. This collects event based rainfall (i.e. is only triggered when rainfall occurs). DOC provided a number of data files downloaded from the rain gauge which were analysed in the software HOBOware. This was converted to CSV files and then assessed in Excel to convert the event based information into hourly and daily timeseries. Table 2 and Figure 3 provides a summary of the data outputs from the rain gauge.

Table 2. Moawhitu tipping bucket rainfall data

Parameter	Description
Start Date	4 July 2017
End Date	2 February 2020
Event depth interval	0.2 mm
Data Gap period	9 August 2018 to 21 January 2019 (memory full)
Data Gap length	165 days
Total Record Length	797 days (excluding data gap), ~2.2 years

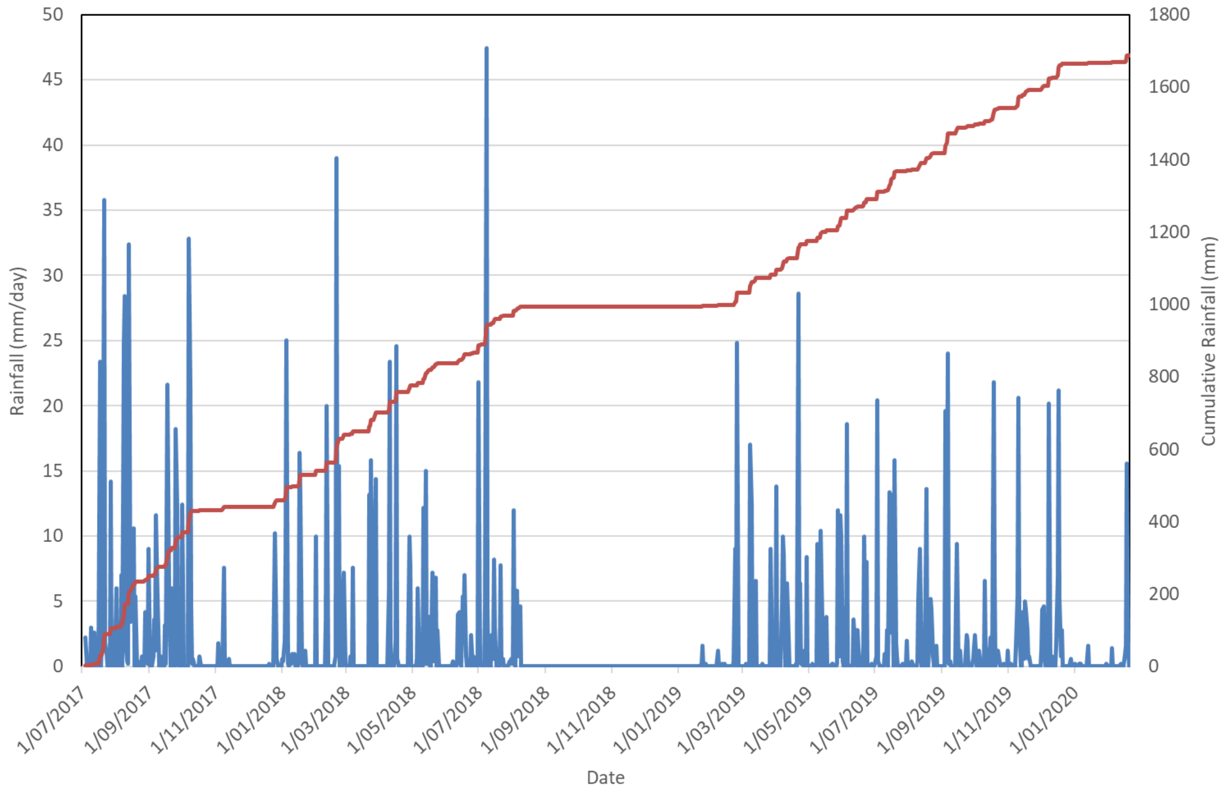

Figure 3. Moawhitu tipping bucket rain gauge daily and cumulative totals (mm) from 1 July 2017 to 20 February 2020 (note the data gap through winter 2018)

The local rainfall gauge was used to assess storm patterns (a rainfall hyetograph) to help inform modelling of peak flow at the outlet control structure for a 100 year ARI storm (1% annual exceedance probability or AEP) (see Section 2.4).

The hourly and daily rainfall data from the local gauge was ranked from highest to lowest, to determine the largest storms that were captured throughout the data series. Six of the largest storms were selected based on their hourly rainfall intensity and their daily (24 hour) total depth. These were subjectively ranked from 1 to 6, based on a combination of the hourly and daily rainfall depths.

The rainfall intensity and depth (presented graphically as a hyetograph in Figure 4) for these storms were used to develop design storm hyetographs for modelling (see Section 2.3.3).

Table 3. Storms selected for creation of rainfall hyetographs

Storm Date	Storm Rank	Hourly peak (mm/hr)	Daily total (mm/day)
20/02/2018	1	9.6	39
18/09/2017	2	8.6	36
21/07/2017	3	8.4	54
16/04/2018	4	10	25
11/02/2018	5	9.6	20
21/04/2019	6	8.4	29

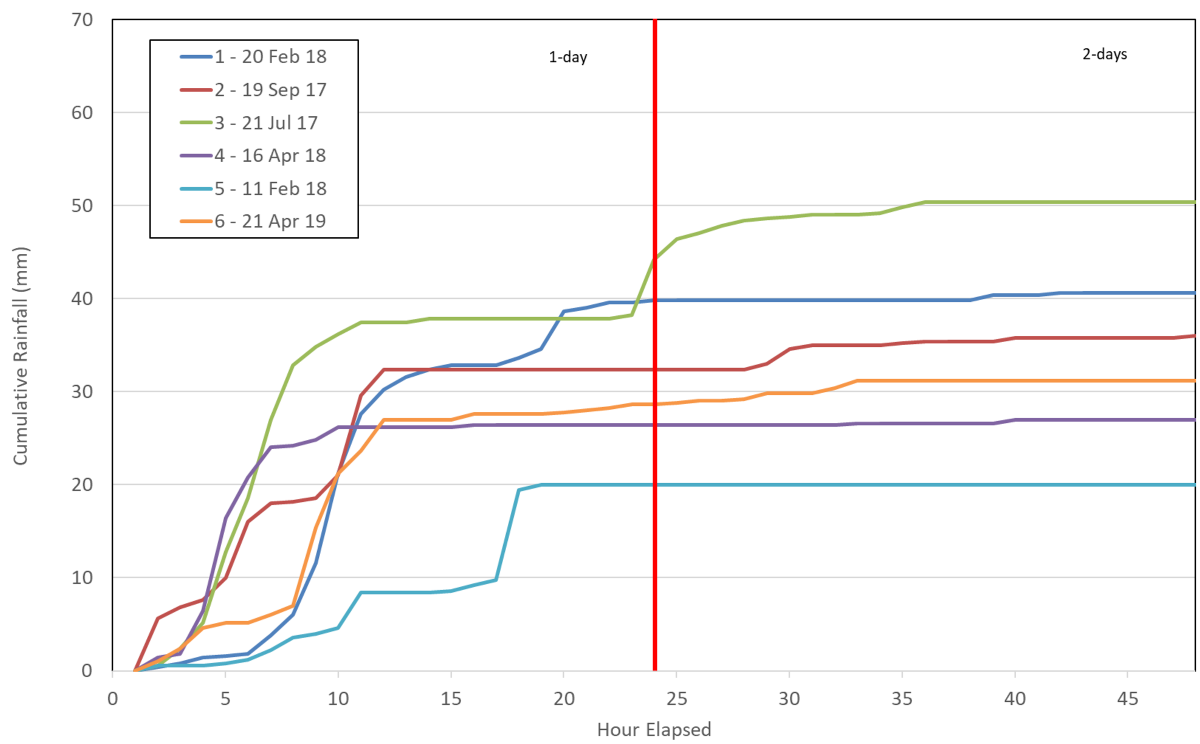

Figure 4. Observed storm hyetographs from the Moawhitu rain gauge.

2.3.3 Design storm hyetographs (for catchment modelling)

Figure 4 shows the greatest intensity and volume in the observed storms occurs over periods shorter than 24 hours, and for this reason a 12 and 24 hour design storm depth was considered appropriate to model the catchment peak flow.

The high intensity rainfall design system (HIRD’s) is a service provided by NIWA. This tool can estimate the magnitude and frequency of high intensity rainfall at any point in New Zealand. Observed rainfall data (historical timeseries) are used with various algorithms to interpolate and predict rainfall intensity at different locations. Given the long term records available at Greville Harbour and Stephens

Island, it was considered appropriate to use the HIRD's outputs for design storm depths rather than correcting from sites a greater distance away but with potentially more data.

The 12 and 24 hour 100 year ARI (1% AEP) rainfall depths from HIRDS (133 and 166 mm, respectively) were used to develop design storm hyetographs for peak flow modelling. This was undertaken for the storms ranked 1 to 3 only (see Table 3) given their greater daily volumes and hourly intensities over storms ranked 4 to 6, and to minimise the number of simulations.

To develop the design hyetographs, the start date and time of each of the storms was identified. The total observed rainfall depth over 12 and 24 hours was then calculated, and used to determine proportions (%) of the hourly rainfall (mm/hr) from the 12 or 24 hour total. This hourly proportion was then multiplied with the 100 year ARI 12 or 24 hour storm depth to create the design storm hyetographs, presented in Figure 5 and Figure 6.

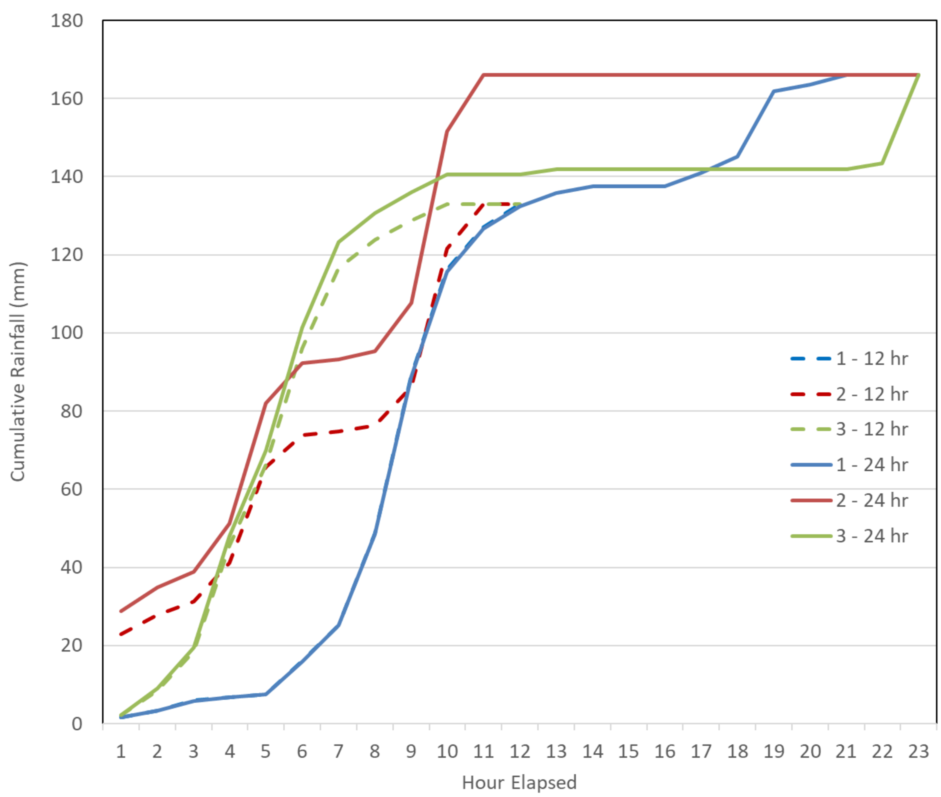

Figure 5. 12 and 24 hour cumulative rainfall depth (mm) for design storm hyetographs adapted from Figure 4.

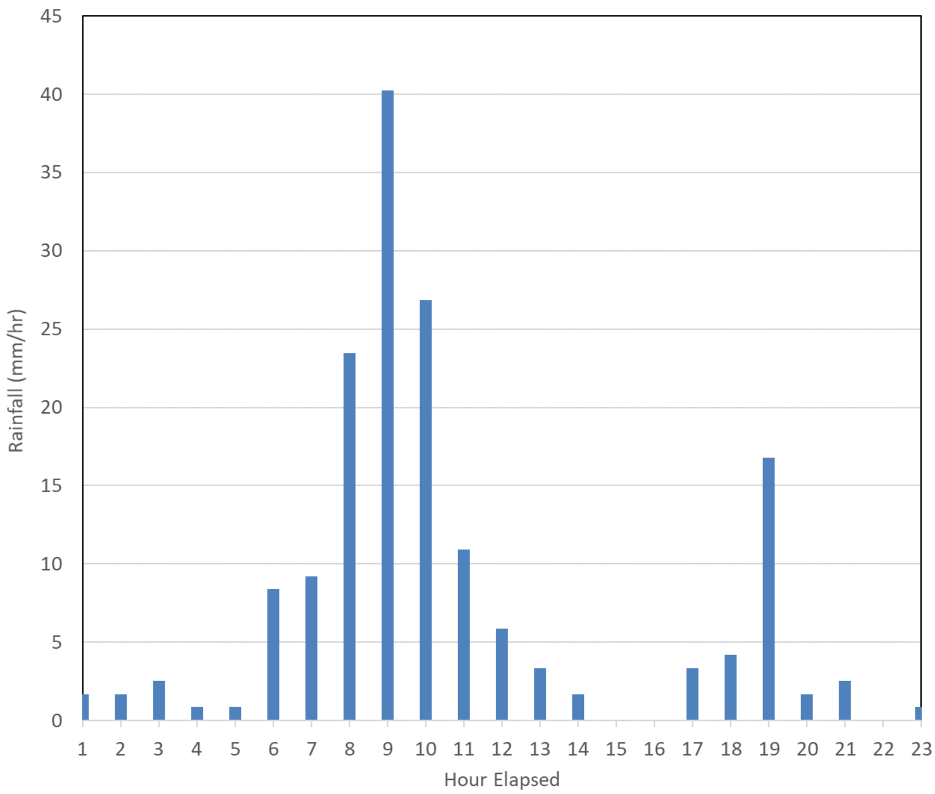

Figure 6. Storm 1 rainfall intensity (mm/hr) for 24 hour design storm hyetograph

2.4 Catchment modelling for peak flow

Hydrological modelling of peak flow at the proposed outlet control structure (Location 2) was undertaken in the software HEC-HMS version 4.3 (US Army Corps of Engineers 2018). No hydraulic modelling was undertaken (for example, incorporating channel routing or dimensions, wetland and lake storage).

2.4.1 Time of Concentration

The time of concentration (Tc) is the estimated time taken for a droplet of water to travel to the outlet of a catchment during a storm event, based on catchment specific criteria. Catchments were delineated from the national REC2 database (and are presented in Figure 7).

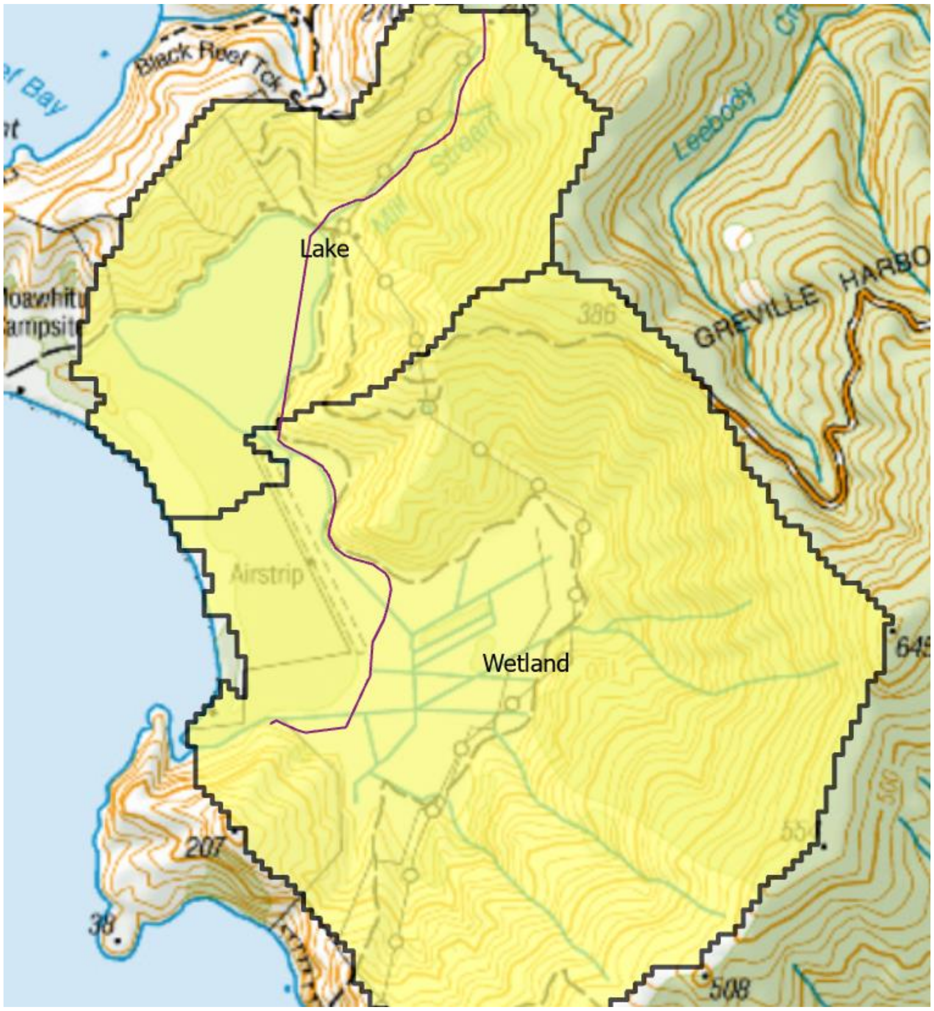

Figure 7. Wetland and Lake catchment boundaries and longest flow path to the outlet channel

Time of concentration (in minutes) was calculated using three methods, with the average incorporated into modelling inputs (see Table 4). Tc is an important input parameter applied in empirical equations used to model a hydrograph from a design storm hyetograph.

Table 4. Time of concentration (minutes) for wetland and lake catchments in Figure 7.

Description		Units	Lake	Wetland	Comment
Input parameters	Area	km2	1.98	4.89	
	Drainage Length	m	1844	2088	Longest drainage path
	Elevation Change	m	376	642	Estimated from Topo Maps
	Horton Infiltration	-	0.1	0.1	Parameter for Rank Grass
Method	Bansby Williams	Tc (mins)	35	33	
	Kirpach	Tc (mins)	29	28	
	Horton Rational	Tc (mins)	72	69	
Output	Average	Tc (mins)	42	41	Used in modelling

2.4.2 HEC-HMS inputs and modelling

The wetland and lake were modelled as two distinct catchments, with their outflows 'combining' at the and outlet node used to estimate the peak flow rate at the control structure. In reality, the lake flows would be routed through part of the wetland prior to reaching the outlet channel, which would further reduce the peak flow.

Inputs to the HEC-HMS software are presented in Table 5.

Table 5. Input parameters to HEC-HMS hydrological simulation

Method	Description	Wetland	Lake	Comment
Canopy Interception	% full at start	20%	20%	Some preceding rainfall
	canopy storage (mm)	4 mm	3 mm	Larger forest catchment in wetland
Soil and Infiltration	Infiltration Loss Rate	No loss	No Loss	Assume saturated soil, middle of winter, no infiltration – conservative
	Surface Soil Storage	No storage	No Storage	Assumes saturated soil, middle of winter, no storage – conservative
Hydrograph Transformation	Hydrograph Method	Clarke		Synthetic Unit Hydrograph, suitable for rural/forested watersheds
	Tc (hours)	0.68	0.71	See Table 4
	Storage Coefficient (hours)	1	1.5	Modified to represent lake and wetland storage buffer without simulating actual storage (with volume elevation curves)
Stream Routing	Roughness, storage and Delay	None	None	Assumes no attenuation or storage in the stream channel, transfer of all rainfall through system - conservative
Baseflow	Initial Discharge	0.3	0.2	Assume 500 L/s at outlet

Six storms (see Figure 5) were modelled to determine the critical storm (the highest peak flow). A sensitivity analysis was undertaken by changing key parameters, such as baseflow, canopy interception and storage coefficient (applied in the Clark Unit Hydrograph).

The storage coefficient (representing catchment storage effects resulting in delays to the hydrograph) has the most significant impact on peak flow. Calculation of this value using empirical equations resulted in very short storage times of <20 minutes, due to the small and steep nature of the catchments. As described by Straub *et al.* (2000), calculation of the storage coefficient for a catchment can become increasingly in-accurate when catchment detention is present, such as through lakes and wetlands, that isn't accounted for in the equations.

This storage coefficient was therefore modified to 1 to 1.5 hours, as a rough means for encompassing some of the detention that would occur in both the lake and wetland, and subsequently to lower the peak flow from what would be considered a 'flashy' catchment if no storage was considered.

2.4.3 Limitations

The catchment modelling undertaken to determine the 100 year ARI (1% AEP) peak flow is highly conservative. The scenario assumes the soil is completely saturated with no infiltration, only a minor interception loss and storage from the forest canopy. There is also likely to be some channel attenuation that hasn't been accounted for, although this would be minimal (due to the steep and short drainage pathways) when compared to the effects of wetland and lake storage.

The modification of the storage coefficient to delay the hydrograph and lower the peak flow is a simplistic approach in an otherwise complicated hydrological setting. This is not considered as a replacement for modelling detention in the lake and wetland (see Section 2.4.3), which could be expected to lower the peak flow even further.

Peak flow estimates are considered to be appropriate for this initial phase of work, where the outputs will be used for concept structure design and to inform an understanding of the peak flows expected at the outlet channel for a 100 year ARI storm.

2.5 Outlet structure failure peak flow

DOC have requested a high level assessment of the peak flow that could arise from the failure of weir boards on the outlet structure. An empirical assessment was undertaken using three dam breach formulas. This utilised:

- inundation storage volume calculated in GIS for a water level height of 3.6 mRL
- water level stage height of 1.2 m (3.6 mRL – 2.4 mRL, the latter representing stream bed level).
- sunny day failure (occurs independently of a storm event)

The results are conservative and have a 1.3 factor of safety incorporated in the formulas due to their simplistic nature. These formulas have primarily been applied to large dams, with high dam crests (often exceeding 10 m), so may be increasingly unreliable when considering storage behind a shallow weir over a large surface area. Hydraulic modelling would help refine these assumptions.

3 Results

3.1 Inundation and depth maps

Presentation of the inundation extent and depth contour maps can be found in Appendix A.

3.2 Long term rainfall

The monthly and annual average rainfall statistics for Greville Harbour has been presented in Table 6. There may have been changes to the rainfall intensity and frequency over the last 40 years due to climate change that hasn't been accounted for in these results, however the ~28 year dataset analysed provides a useful baseline understanding of the climate at Moawhitu Wetland.

The wettest and driest months are July and February, respectively. The wetland is still however susceptible to storms during the summer periods, including cyclones that may make their way down towards the Nelson Region. The peak design storm used in modelling occurred in February 2018.

Table 6. Monthly and annual average rainfall (mm) for Greville Harbour from 1958 to 1985.

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
75	60	78	87	102	98	123	96	91	86	76	80	1052

Table 7 shows the maximum annual rainfall recorded for the wetland was ~1,679 mm in 1962, while during the driest year only 653 mm fell (1969), which also followed with a month of no rainfall in summer of 1970.

Table 7. Maximum and minimum monthly and annual rainfall (mm) at Greville Harbour from 1958 to 1985.

Parameter	Depth (mm)	Month/Year
Monthly Max	285.8	July 1978
Monthly Minimum	0	February 1970
Annual Maximum	1678.6	1962
Annual Minimum	653.5	1969

3.3 Stephens Island and Greville Harbour correlation

Figure 8 shows that the cumulative rainfall at Greville Harbour was greater than Stephens Island over a ~28 year record (1958 to 1985).

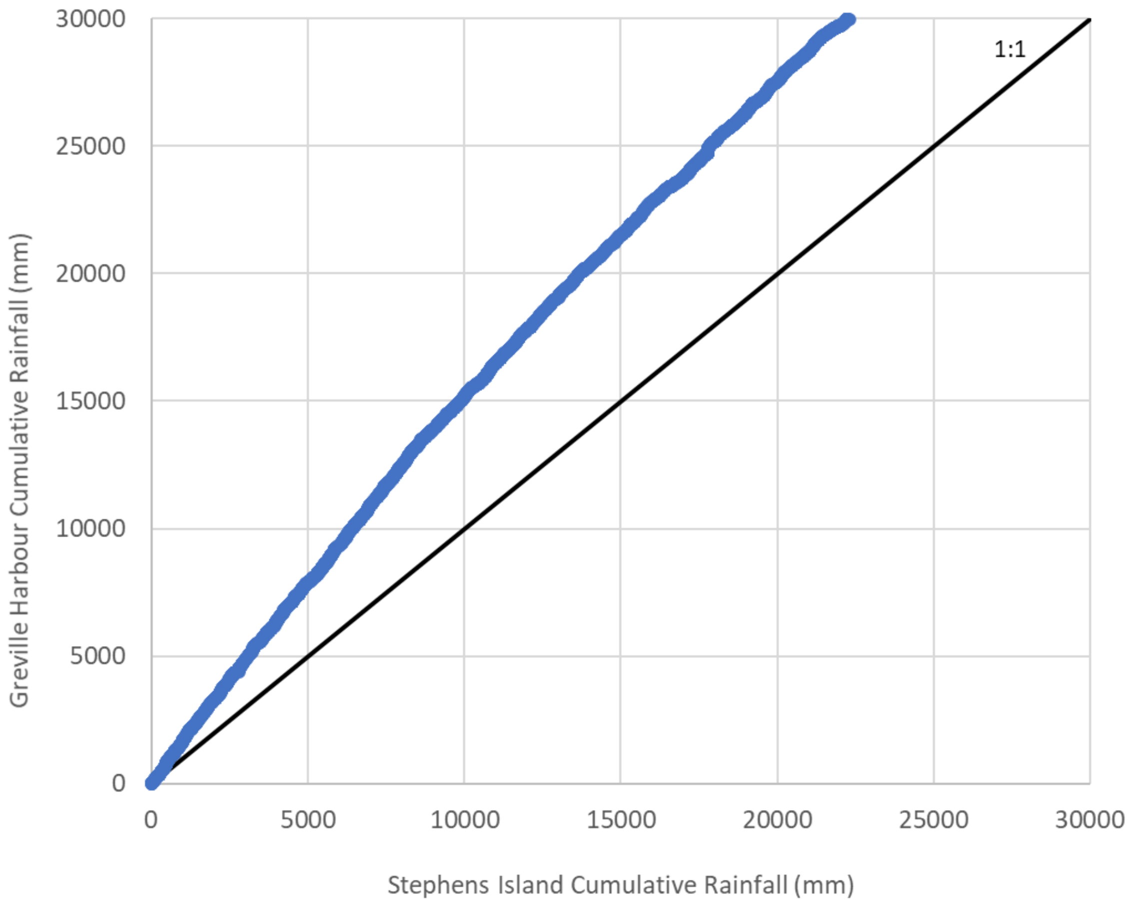

Figure 8. Greville Harbour and Stephens Island cumulative rainfall correlation from 1958 to 1986

Figure 9 plots the monthly rainfall totals for Stephens Island and Greville Harbour. A linear regression line has been included in the correlation which can be utilised to estimate monthly rainfall totals for the wetland from Stephens Island long term rainfall gauge, should the local rain gauge at Moawhitu be discontinued.

It should be noted, while the monthly rainfall relationship between the two islands is reasonable (~67%), there is a wide variance in the scatter which increases as monthly rainfall depths exceed 150 mm.

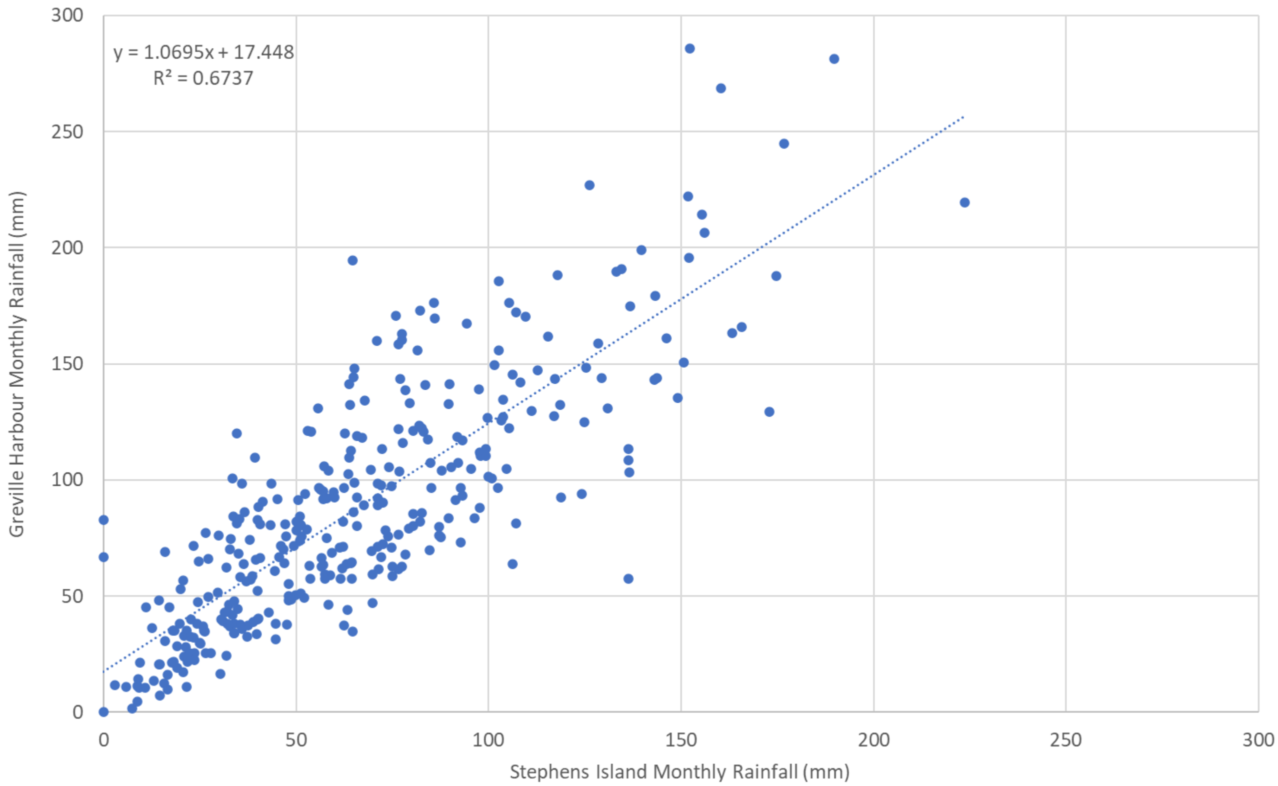

Figure 9. Monthly rainfall total (mm) correlation between Greville Harbour and Stephens Island from 1958 to 1985.

3.4 Catchment Peak Flow

The greatest peak flow was simulated for storm 1, which occurred on 20 February 2018 (Table 3, Figure 6).

When considering the hydrographs of storms 1–3 (Figure 10), the 24 hour storm consistently has a greater or equal peak flow than the 12 hour storm. For this reason, the critical storm for the catchment is considered to occur over 24 hours. Further simulations of 3, 6, and 48 hour events could be useful to help validate this assessment.

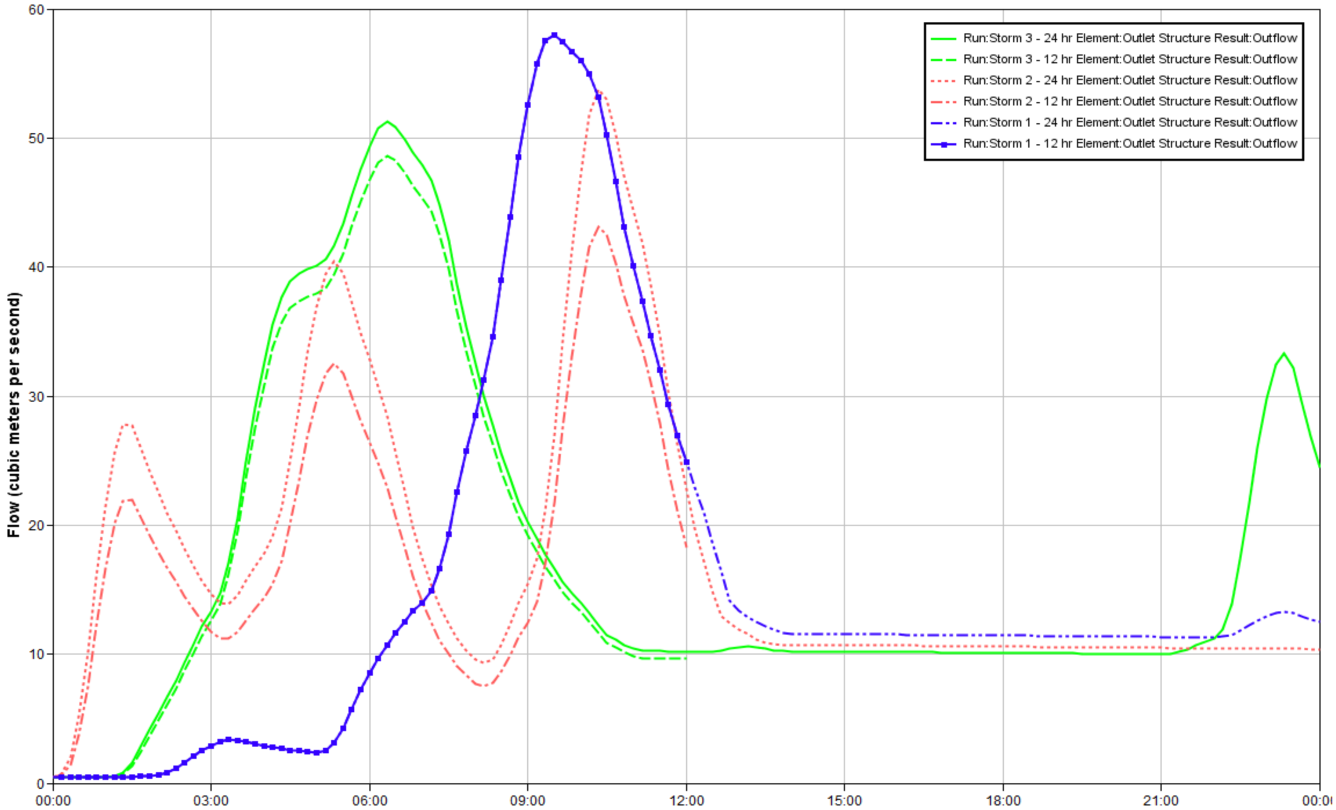

Figure 10. Outlet control structure hydrographs for the 100 year ARI (1% AEP) design storms presented in Figure 5.

Table 8 shows the peak flow at the outlet was predicted to be ~58 m^3/s and occurred ~9 hours and 20 minutes after the start of the storm event. This 100 year ARI 24 hour peak flow is highly conservative (as no storage has been considered) and therefore could be considered to be greater than what would occur in reality, with the peak occurring in a shorter timeframe than would likely happen when storage is accounted for.

Table 8. 100 year ARI (1% AEP) design storm peak flows and time to peak for Moawhitu Wetland at the proposed outlet control structure.

Storm event and duration	Peak flow (m^3/s)	Time to Peak
1 - 12 hr	58.0	09:20
1 - 24 hr	58.0	09:20
2 - 12 hr	43.1	10:10
2 - 24 hr	53.8	10:10
3 - 12 hr	48.6	06:10
3 - 24 hr	51.2	06:10

3.5 Outlet structure failure peak flow

The estimated peak flow from a failure of the outlet structure has been presented in Table 9. The average peak is ~46.8 m^3/s and incorporates a safety factor of 1.3 to account for uncertainty in this empirical approach. There are a wide range of formulas available to assess dam breaks, however the Froehlich equation has been widely applied globally.

Table 9. Estimated outlet structure peak flow during a failure of the weir boards at 3.6 mRL (empirical estimates only).

Method	Storage Volume (m^3) at 3.6 mRL	Peak Flow (m^3/s)	Reference
1	~200,000	74.2	Queensland Government (2018)
2		36.2	Froehlich (1995) in Wahl (2004)
3		29.9	SCS (1981) in Wahl (2004)
Average peak (m^3/s)		46.8	

A failure of the weir boards at 3.6 mRL would likely confine flow within the outlet channel, with water routed through the planned concrete structure. Due to the natural topographic constraints, water would be unlikely to flow around or over the structure at this water level. The formulas used in this assessment are applied to dams which commonly have a wider and higher crest, that during a failure would erode and release a greater volume over a shorter timeframe.

For comparison, a mannings flow calculation was undertaken for a rectangular concrete channel with the planned structure dimensions and water level (1.5 m (w) x 1.2 m (h)). A flow rate of ~14 m^3/s (under steady flow) conditions was estimated. This supports the idea that Table 9 is a conservative assumption for peak flow.

4 Recommendations

A number of recommendations for assessments at Moawhitu have been documented in Blyth 2020. The assessments below relate specifically to the hydrological modelling undertaken in Phase 2.

4.1 Outlet channel hydraulics and water level height

In the absence of a hydraulic model, the channel cross section data collected by the surveyor could be incorporated into a mannings open channel flow assessment. The 100 year ARI peak flow can then be used with various roughness and slope parameters to predict the wetted perimeter and water level height at different cross sections.

Further modelling may be required with other software packages to assess potential scour and inundation (as this empirical approach would only provide an average velocity), although this may not be required given that scour may be a natural feature during peak discharge and during coastal storm surges.

4.2 Rainy day peak flow at outlet structure

A rainy day failure would consider the 100 year ARI storm occurring at the same time as the outlet control structure fails at the design invert level. Currently, the hydrological model incorporates a baseflow of 0.5 m^3/s at the outlet (see Section 2.4.2). In the most simplistic approach, this baseflow could be replaced with the peak flow estimated from the outlet structure ‘breach’. However, it would be more accurate to model an actual dam breach occurring within HEC-RAS.

The corresponding rainy day peak flow could then be considered with the mannings open channel method described in Section 4.1.

The downstream property is considered to be at low risk from any effects of flood inundation from an outlet control structure failure, due to its distance and elevation from the main outlet channel. Simulations from Phase 2 and possibly the incorporation of a mannings open channel flow assessment to determine water levels at various cross sections may be sufficient for risk assessments to validate this assumption.

4.3 Climate change

NIWA HIRD’s provides predictions of rainfall intensity under various climate changes scenarios (for example, RCP 8.5). A climate change scenario could be considered to estimate peak flow for the catchment, which would require updating of the design storm hyetographs and re-running of the model.

5 References

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Appendix A – Dry Weather Inundation Maps

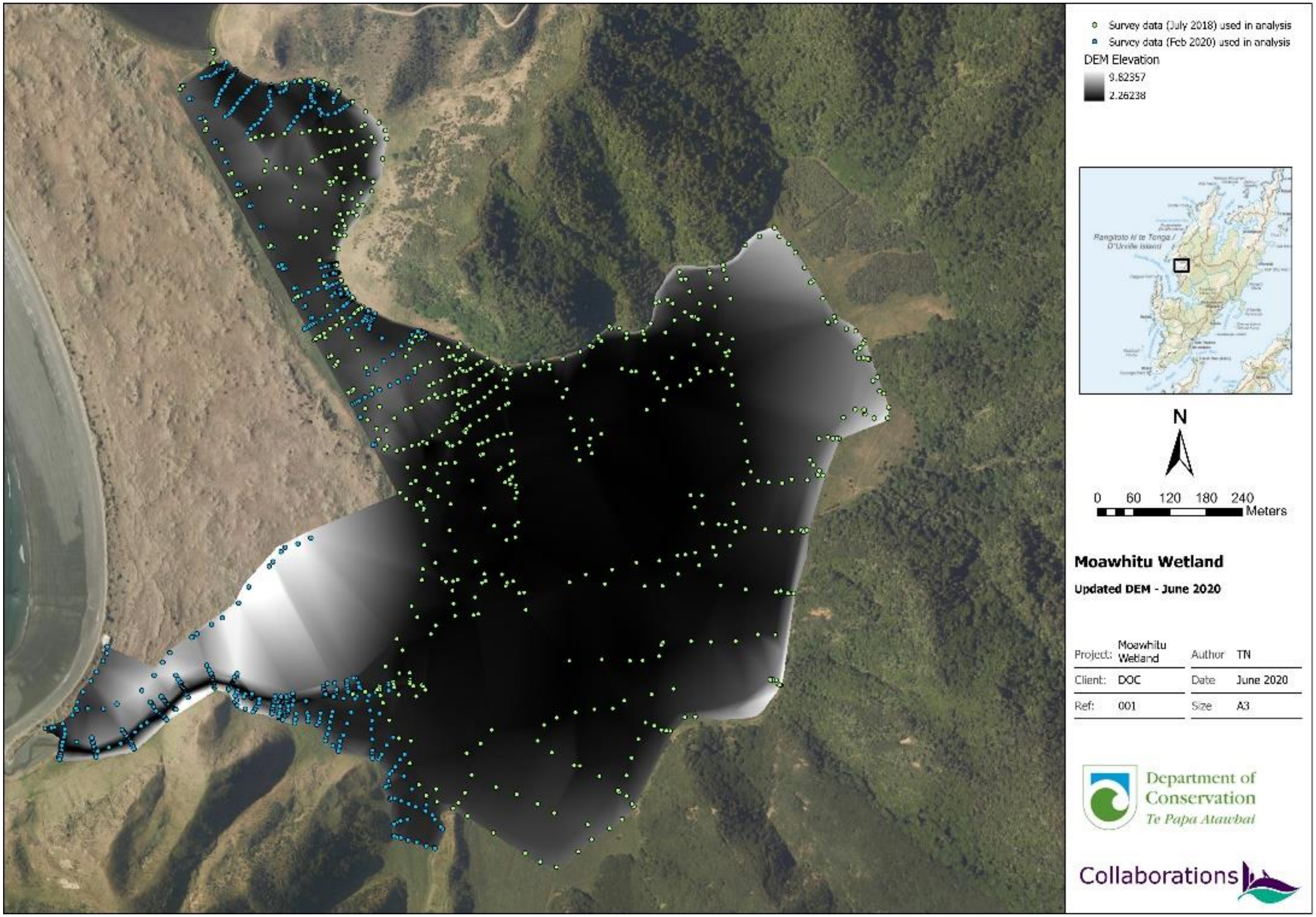

Figure A 1. Davis Oglive and Partners survey points used to develop Moawhitu DEM

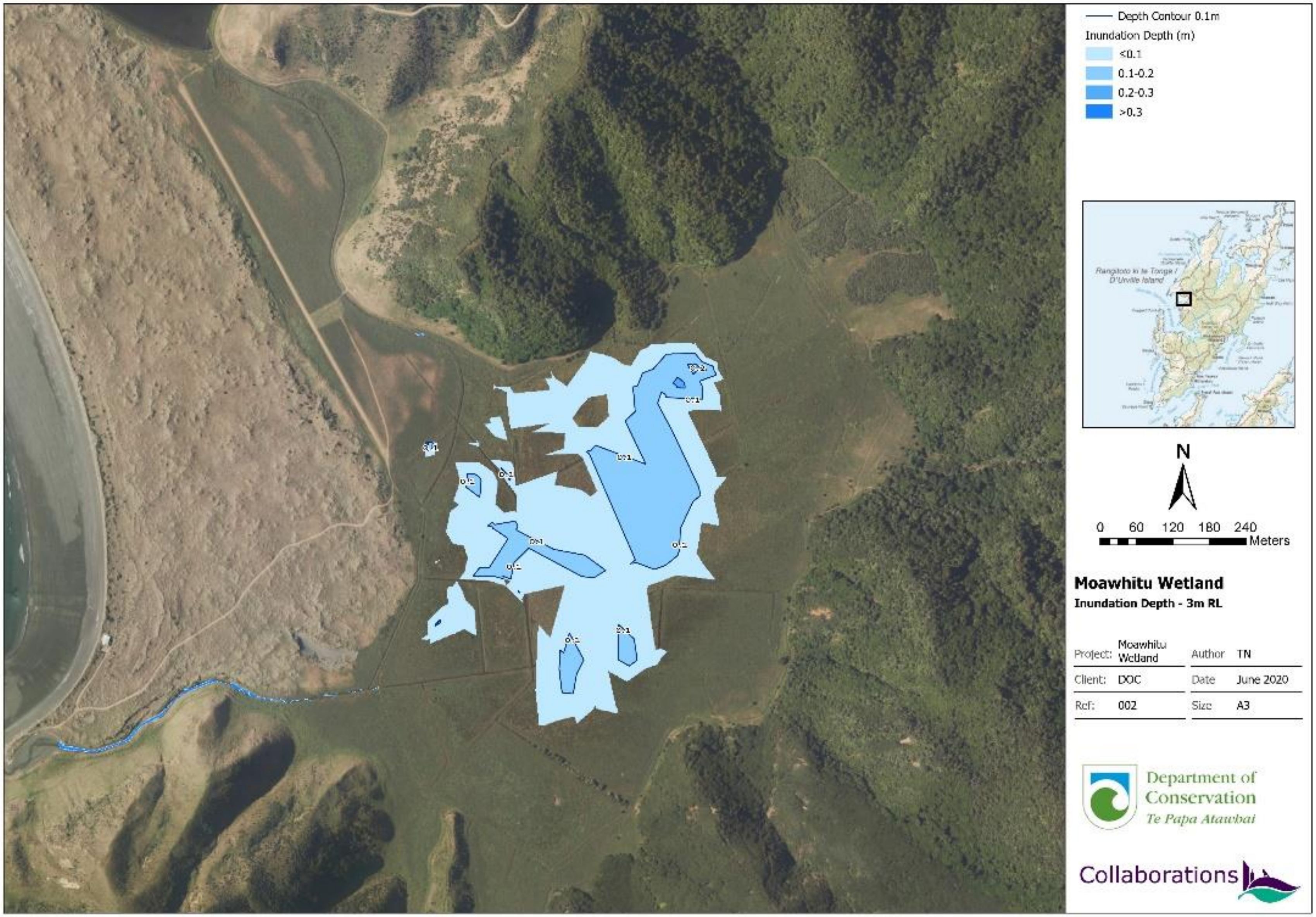

Figure A 2. Water level depth contours at 3 mRL

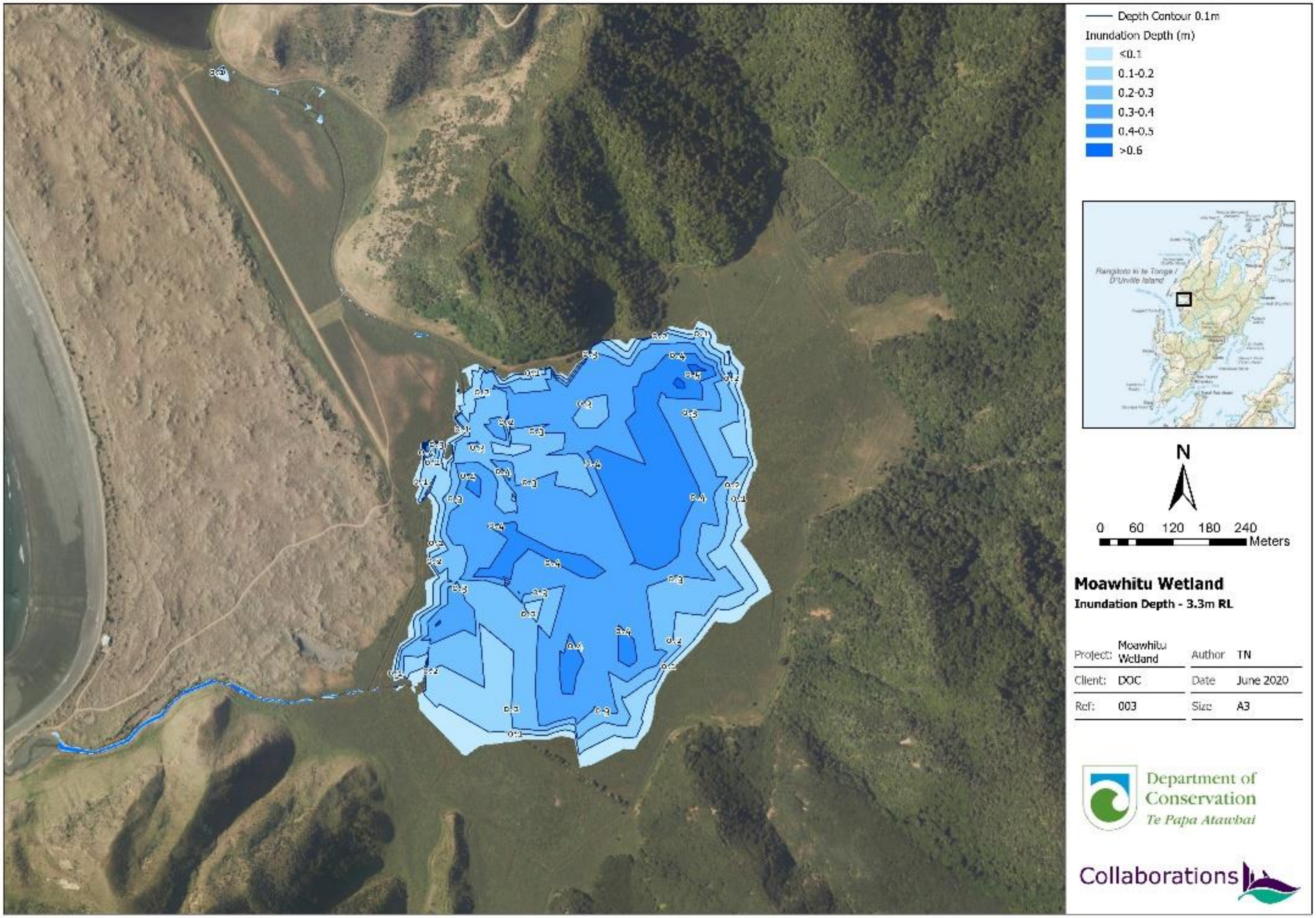

Figure A 3. Water level depth contours at 3.3 mRL

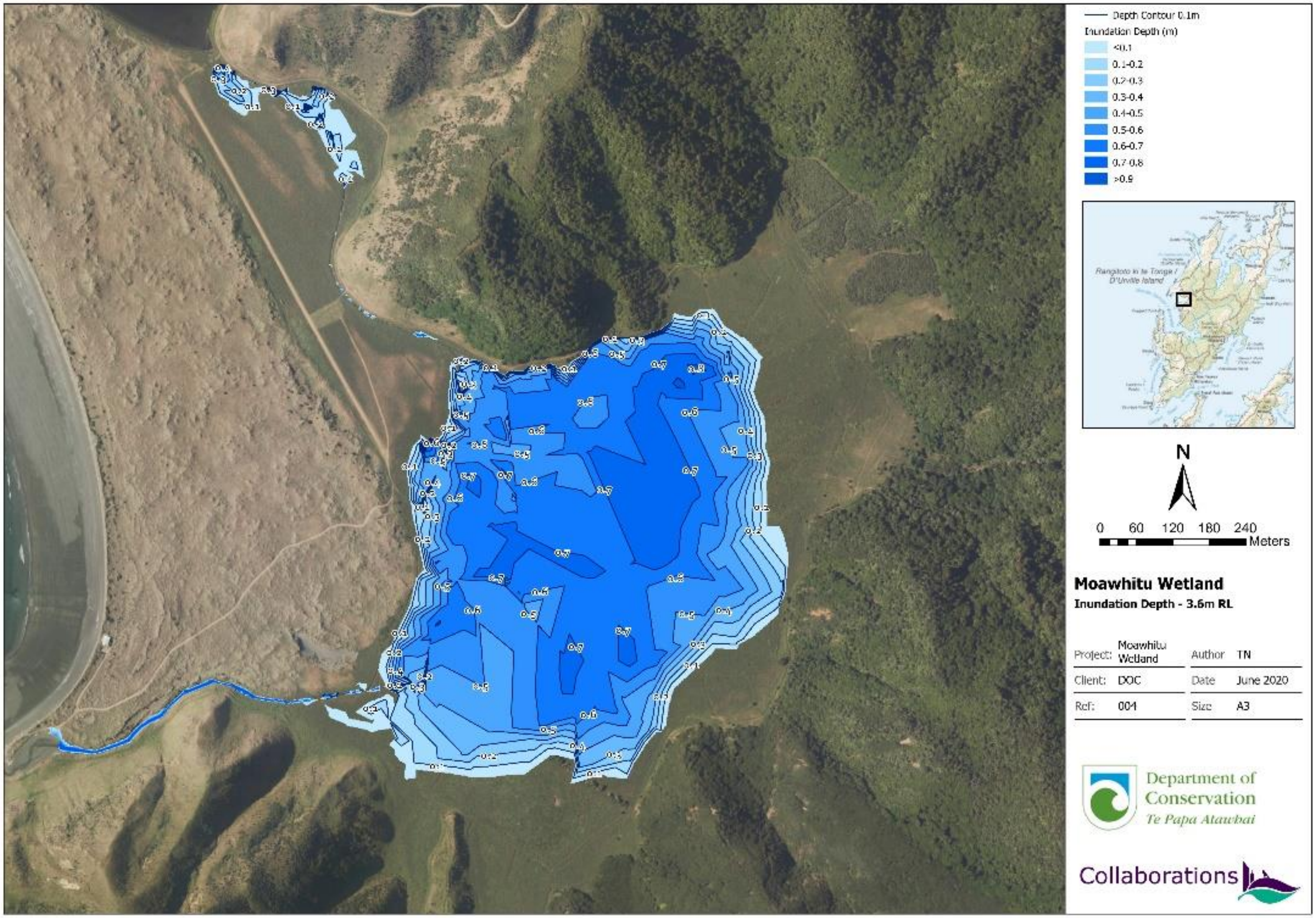

Figure A 4. Water level depth contours at 3.6 mRL

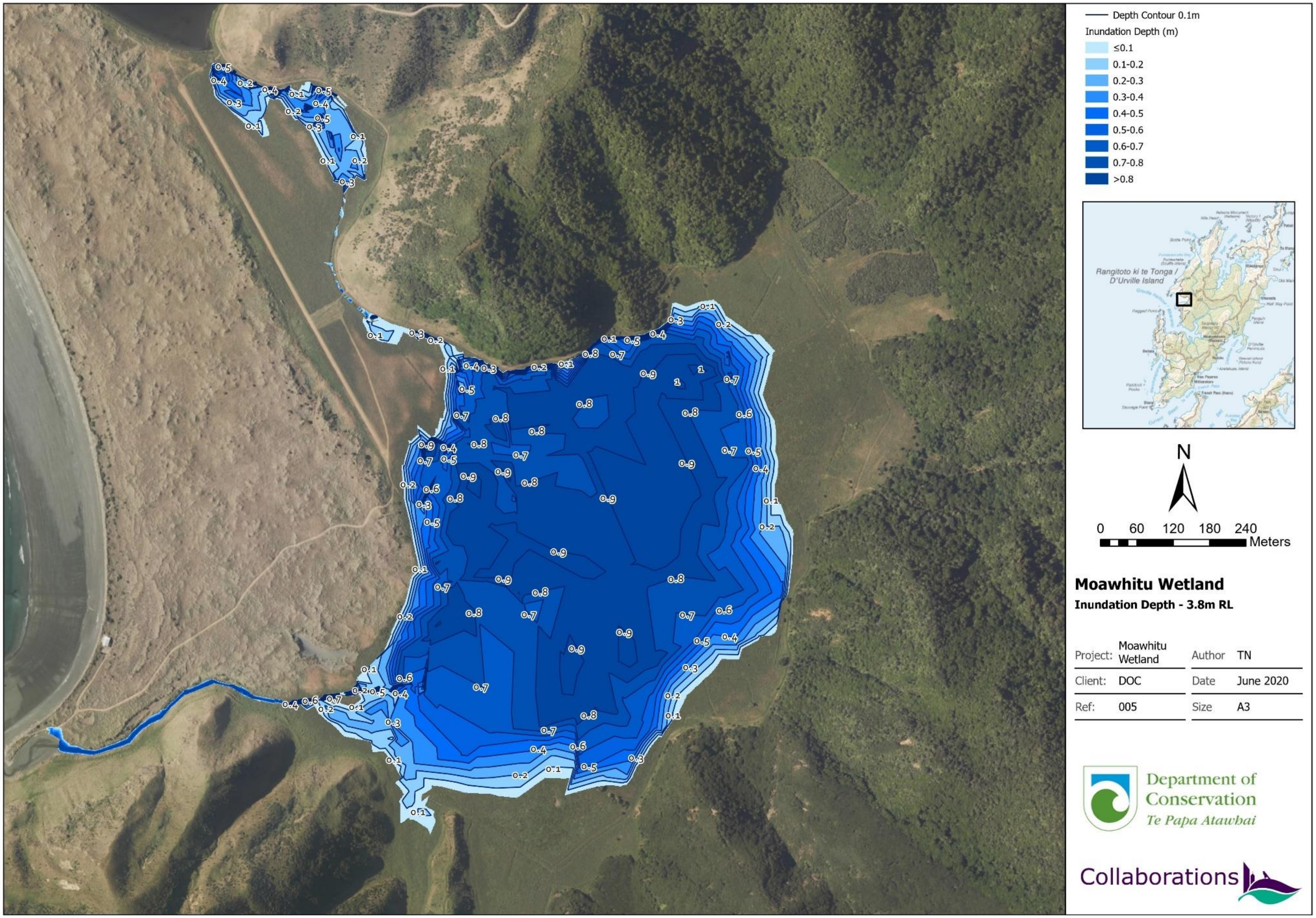

Figure A 5. Water level depth contours at 3.8 mRL

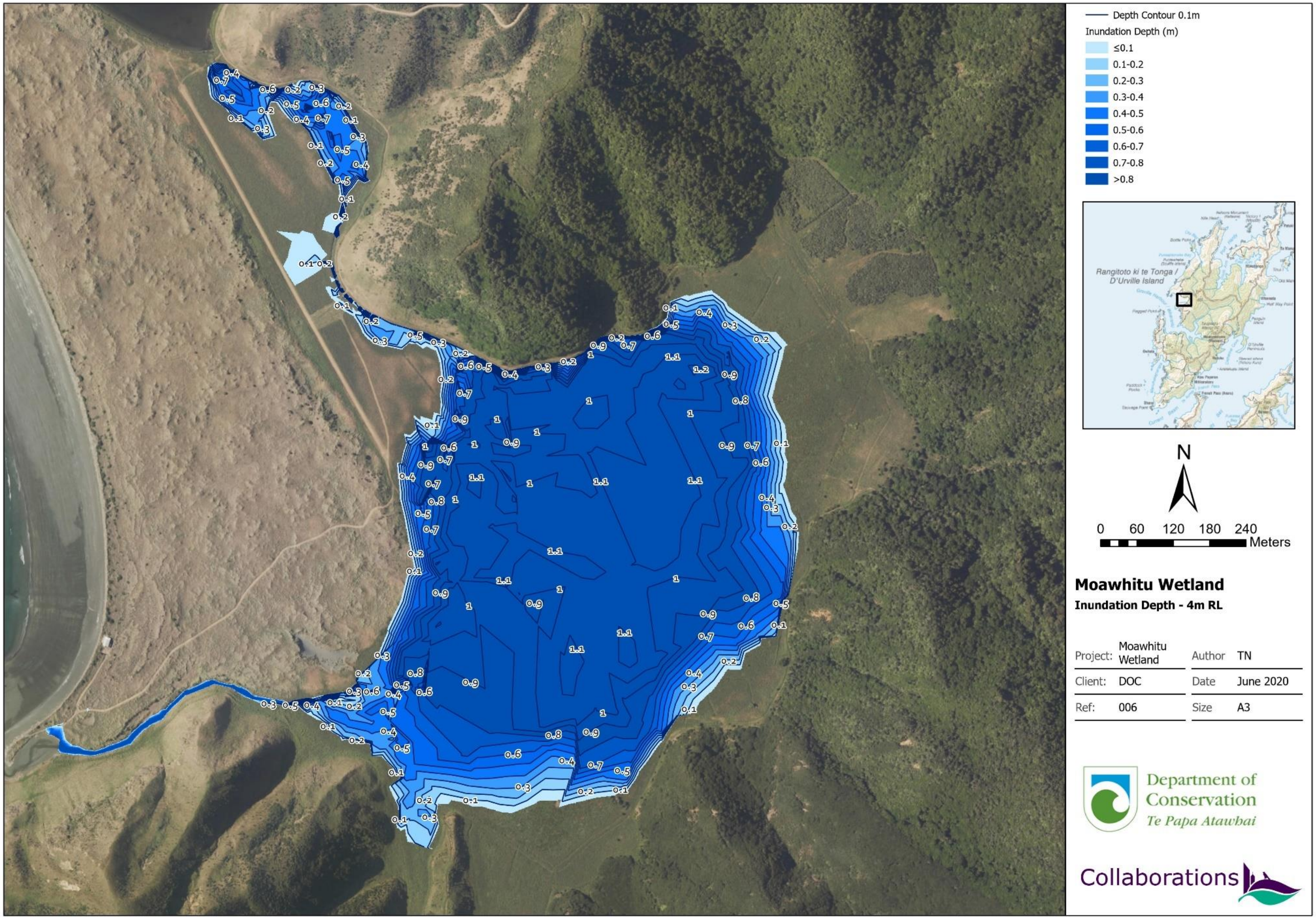

Figure A 6. Water level depth contours at 4.0 mRL

Appendix B – Outlet Control Structure Concept Design

Orogen Limited
Registered Company 5908349

Our Ref: W20043

19 June 2020

Collaborations Ltd

Attention: James Blyth
Via email james@collaborations.co.nz

Dear James

CONSTRUCTION METHODOLOGY – D'URVILLE ISLAND WEIR STRUCTURE FOR MOAWHITU WETLAND

Presented below is an outline for an indicative construction methodology to be used for the purposes of outlining the works required and the potential temporary works required during construction. The contractor employed to undertake the works may utilise an alternative method.

Principals for construction

- Work in the dry – ensure the existing stream channel is diverted around the worksite.
- Keep excavations supported – excavations over 1m in height shall be supported by either utilising a stable cut angle of 1v:1.5h or employing trench shields.
- Manage sediment discharges from the construction works.
- Mitigate flood risk by checking the weather forecast prior to and during construction and amend the works as required.
- Utilise pre-cast elements to accelerate the construction period.

Methdology

- Monitor the weather forecast and only begin works when a period of clear weather is forecast.
- Excavate a diversion channel around the work site, starting at the downstream end. The diversion channel shall be at a gradual angle to the main stream flow and be at least 2- 3m wide to allow rainfall induced flows to pass around the site. The channel shall be stabilised with onsite materials or geotextile fabric to reduce the risk of silt runoff and scour. Once stabilised the upstream connection to the existing channel can be made. This connection shall be at least 5m upstream of the worksite. The diversion channel shall preferably be on a constant grade to ensure fish passage is available throughout the construction period.
- Utilise sandbags across the stream channel both above and below the work site, to keep the work area dry. A pump maybe required to dewater the work site excavation.
- Ensure all fish are removed from the work area and placed in the running stream.
- Excavate soft stream bed material to reach solid ground sub-grade material – minimum excavation depth shall be 700mm. All machinery shall be operated from a stable platform on the stream banks. No mobile plant shall enter the stream channel.
- Excavate areas around headwalls to specified width.
- Place geotextile fabric to prevent soil migration through the sub-base metal.
- Place sub-base metal to a depth of at least 450mm to create a solid foundation for the pre-cast concrete elements to be placed on. Ensure protrusions within the pre-cast elements are catered for, so the pre-cast structure is fully supported along its length by the sub-base metal.

- Place channel section central to the stream channel.
- Place headwall to inlet and outlet of channel – final level of these concrete headwalls shall be 180mm below the existing stream bed level.
- Place second layer of fabric to prevent backfill from migrating into the sub-base metal.
- Backfill behind headwalls to re-create the stream banks. Place rocks around inlet headwall where areas will be below water level and therefore subject to scour.
- Face mount the weir supports to the concrete channel and seal.
- Secure desired weir height using Stop Logs and place fish ramps.
- Weir heights shall have a maximum height difference of 500mm to ensure fish ramps maintain a gradient of 15 degrees.
- Place 200 – 300mm rocksto 600mm deep around the headwall in areas within 300mm of the set weir level (both above and below the set water level)
- Re-spread topsoil and carry out planting to stabilise the backfill area.
- Remove sandbags within the stream channel, and allow the water level to stabilise.
- Remove fish and backfill the diversion channel.

Any excess material can be placed or stockpiled on the true left bank at the direction of the Engineer, to begin formation of a bund, to enable weir heights to reach up to RL4m.

Heavy Rainfall Contingency Measures
If heavy rainfall is forecast once work onsite has begun, we would suggest:

- Ensuring the diversion channel is at least the width of the existing stream channel and placing additional sandbags upstream of the work site.
- Once the sandbags get over 0.5m in height they may require reinforcing to ensure they remain in place during peak flows. This can be done using geofabrics over top of the bags, secured in place with waratahs. Earth can also be placed behind the sandbags to reinforce them, although this should be covered in geofabric material to stabilise.
- Move all machinery and uninstalled materials to higher ground.

Please find attached a product brochure of the stop logs which are manufactured to designer provided specifications, which means the weir heights can be specified to various heights to create a range of wetland depths. We would recommend constructing these out of marine grade Aluminium, to reduce the corrosion risk. Various coatings are also available to enhance the design life.

Please do not hesitate to contact the undersigned with any queries.

Yours faithfully

Karla Beamsley
Principal Engineer
Orogen Limited

Encl Drawings PL501, PL531
Product Brochure for Stop Logs

Project: W20043 - Moawhitu Wetland

Application: Concept Design

BUDGET COST ESTIMATE

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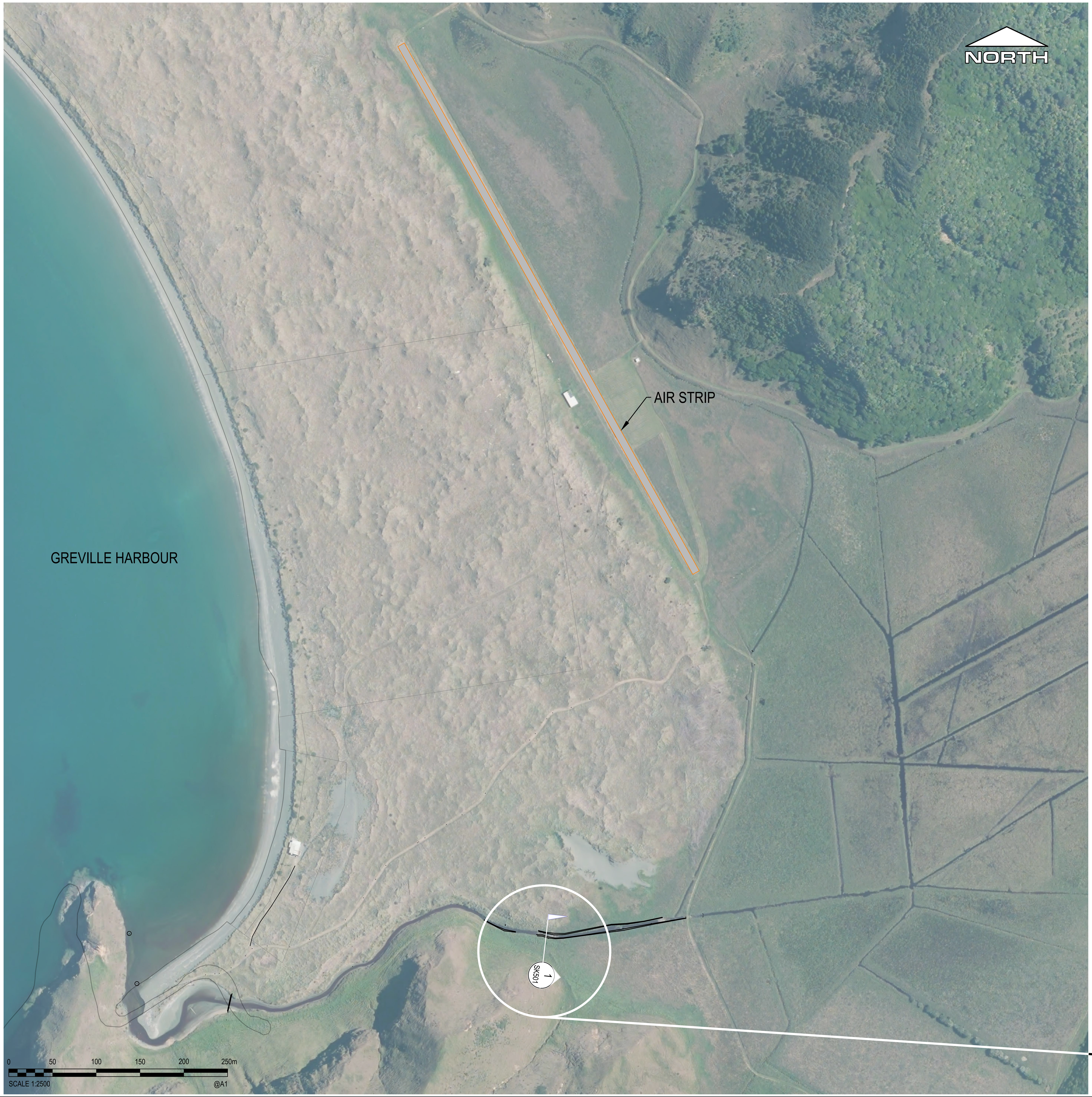
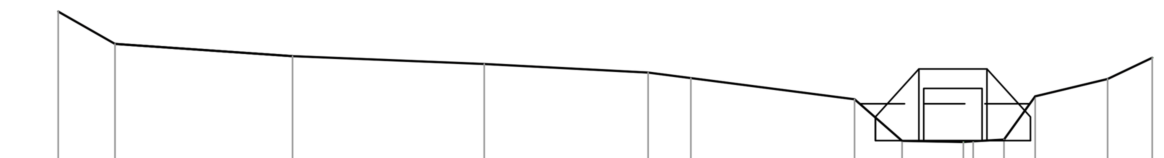
Date of Estimate 29/06/2020

Cost Estimate Assumptions

1 Supply of pre-cast structures and materials to Nelson
2 Does not include transport to D'Urville Island
3 Structure costs do not include tranportation to site or installation costs, unless noted
4 Assumes quarry material for backfill is available - we have provided an estimated quantity of quarry rock as bedding and backfill material
5 All prices are in NZD. Stop logs were quoted in AUD so an exchange rate of $1 AUD = $1.08 NZD was applied
6 Prices are valid for 30 days from the date of estimate
7 No allowance for labour accomodation onsite during construction.

Item	Description	Unit	Quantity	Rate	Total	Notes
1	**STRUCTURE**					
1.1	Pre-cast concrete wingwall	Ea	2	$ 1,500.00	$ 3,000.00	Headwalls are supplied to Nelson without a base, base to be poured onsite (2.8m³ concrete). Option for base to be poured offsite and transported to site.
1.2	Pre-cast concrete channels	Ea	2	$ 3,800.00	$ 7,600.00	Supplied to Nelson
1.3	Upstream Weir Stop Logs - 100mm high x 1500mm wide (max height 1.3m)	Ea	13	$ 383.40	$ 4,984.20	Frame 316 SST, stop boards Marine Grade Aluminium
1.4	Downstream Weir Stop Logs - 100mm high x 1500mm wide (max height 1.1m)	Ea	11	$ 383.40	$ 4,217.40	Frame 316 SST, stop boards Marine Grade Aluminium
1.5	Lifting Ladder 1500mm Long to safely insert/install stop boards	Ea	1	$ 1,300.00	$ 1,300.00	Includes delivery with Stop Log order
				Structure Total	$ 21,101.60	** doesn't include install costs of these structures*
2	**MATERIALS**					
2.1	Geotextile Fabric	Roll	1	$ 420.00	$ 420.00	Roll
2.2	Fish Ramp	Ea	2	$ 769.57	$ 1,539.13	2.4m long x 0.54m wide (option for 1.2m long also) - doesn't include install costs
2.3	GAP20 Metal subbase for pre-cast concrete elementss	m³	25	$ 120.00	$ 3,000.00	rate based on supply from nearby quarry on D'Urville Island with a 4hr round trip with 5m³
2.4	Excavate unsuitable material for stockpile	m³	45	$ 10.00	$ 450.00	estimated quantity, based on topo survey
2.5	Backfill from stockpile	m³	40	$ 10.00	$ 400.00	estimated quantity, based on topo survey
2.6	Stabilisation of backfill - grass seed or planting	PS	1	$ 500.00	$ 500.00	nominal allowance for planting
2.7	Onsite concrete as required	PS	1	$ 1,000.00	$ 1,000.00	nominal allowance
				Materials Total	$ 7,309.13	
3	**TEMPORARY WORKS**					
3.1	Excavate bypass channel	PS	1	$ 1,200.00	$ 1,200.00	allows for 1 day of work by excavator
3.2	Sandbag stream channel	PS	1	$ 550.00	$ 550.00	nominal allowance to supply sandbags
3.3	Dewater work site	PS	1	$ 800.00	$ 800.00	nominal allowance
3.4	Allowance for Electric Fishing	PS	1	$ 1,000.00	$ 1,000.00	nominal allowance
				Temporary Works	$ 3,550.00	
				Budget Estimate	**$ 31,960.73**	**+GST**
				+Allowance for Install costs	$ 47,941.10	+50%
				+Allowance for Transportation costs	$ -	To be confirmed
				+ Allowance for Establishment onsite	$ -	To be confirmed
				+ Allowance for accomodation costs	$ -	To be confirmed

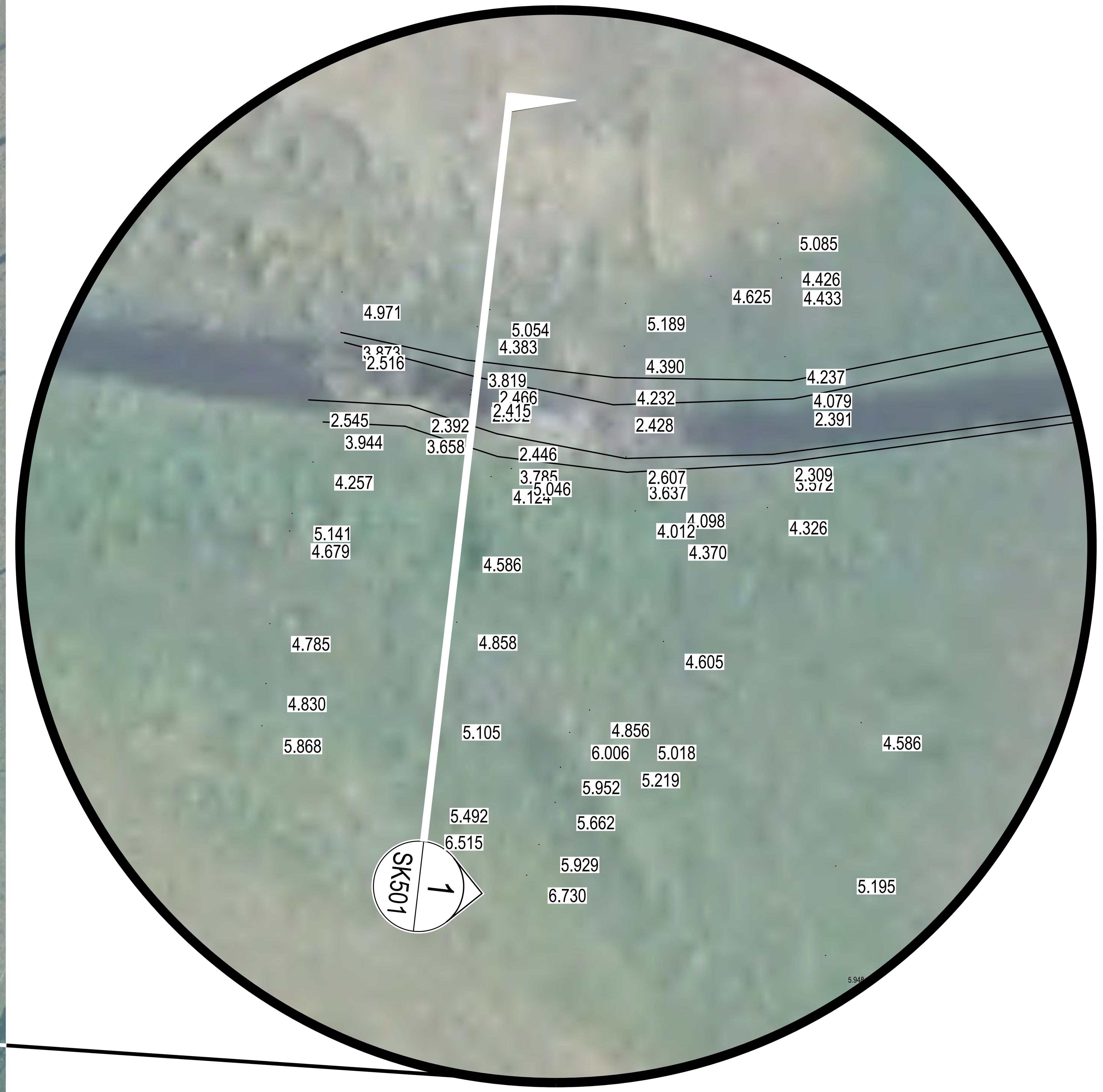




DATUM RL: 0.00

GROUND LEVEL	6.515	5.492	5.105	4.858	4.586	4.411	3.741	2.426	2.392	2.415	2.469	3.835	4.383	5.054
CHAINAGE	0	1.794	7.408	13.467	18.649	20	25.174	26.678	28.614	28.924	29.9	30.884	33.175	34.584

SECTION 1 SK501
SCALE 1:200m



Issue	Date	Description	Des.	Veri.	App.
1	DD/MM/YY	CONCEPT ISSUED FOR CONSENT	?	?	?

Drawn JBC	Date 18/06/2020
Checked	Date 18/06/20
Design/Surveyed KJB	Date 18/06/20
Verified KJB	Date 18/06/20
Approved	Date DD/MM/YY

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Unit 1a - 10 Surrey Street, Tawa
PO Box 56051 - Tawa - Wellington 5249

Client COLLABORATIONS LIMITED
Project D'URVILLE ISLAND MOAWHITU WETLAND
Title SITE PLAN
Status FOR CONSENT
Datum WGTN2000 | Ref | Scale 1:2500 | Original Size A1
Drawing Number W20043-01-PL501 | Issue 1

DATE: 19 June 2020 5:07 PM
Reference Files: ####
Drawing File: W20043-SK000-TOPOGRAPHICAL-01.DWG

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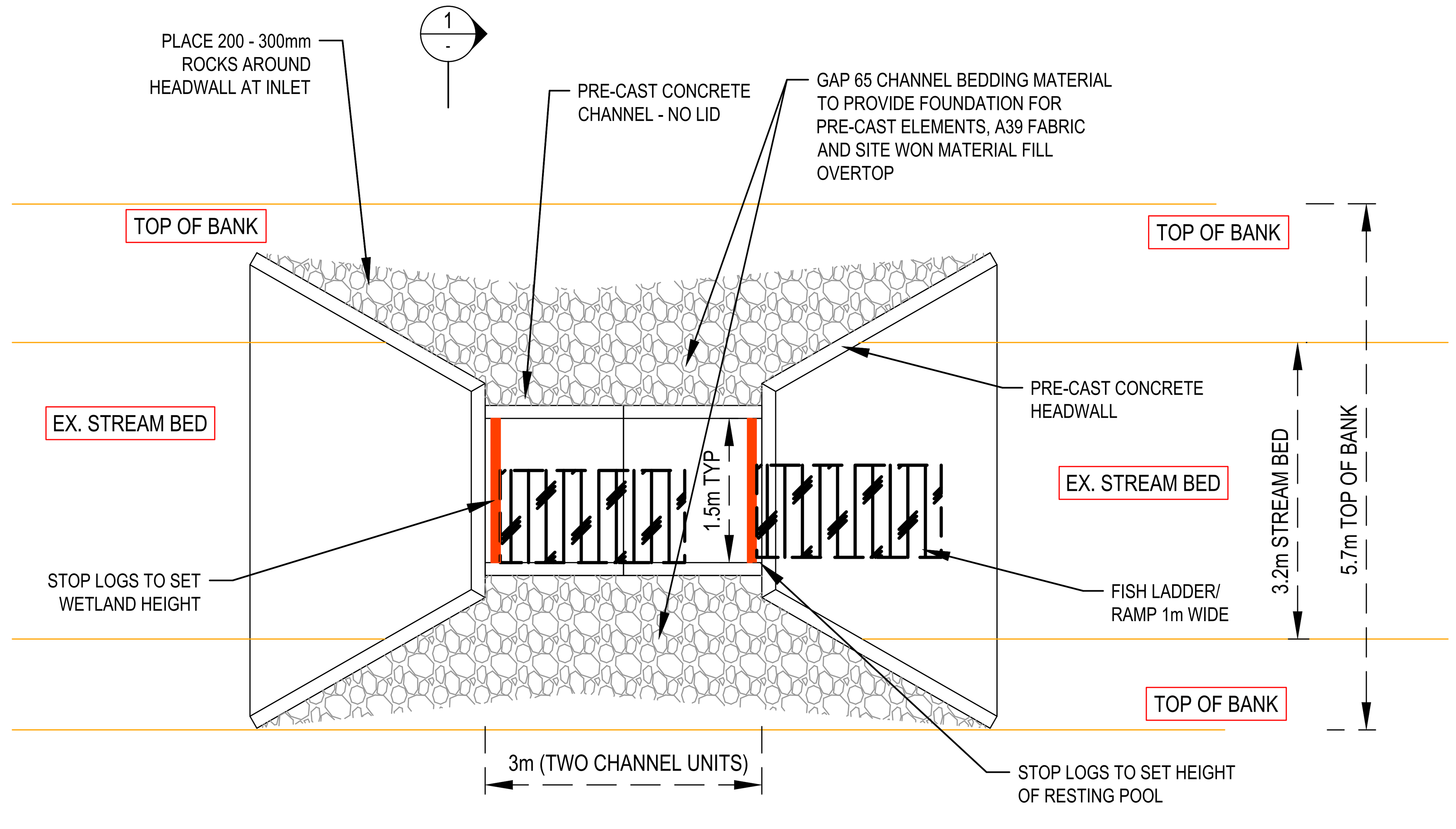
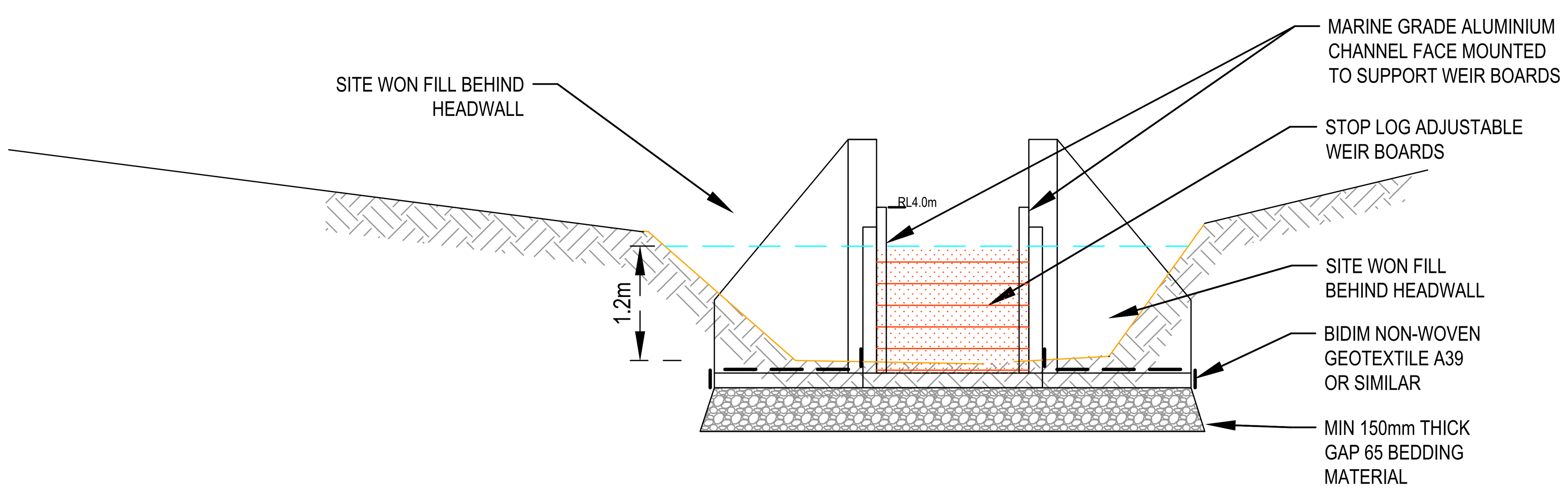


PLAN DETAIL 501
1 : 50



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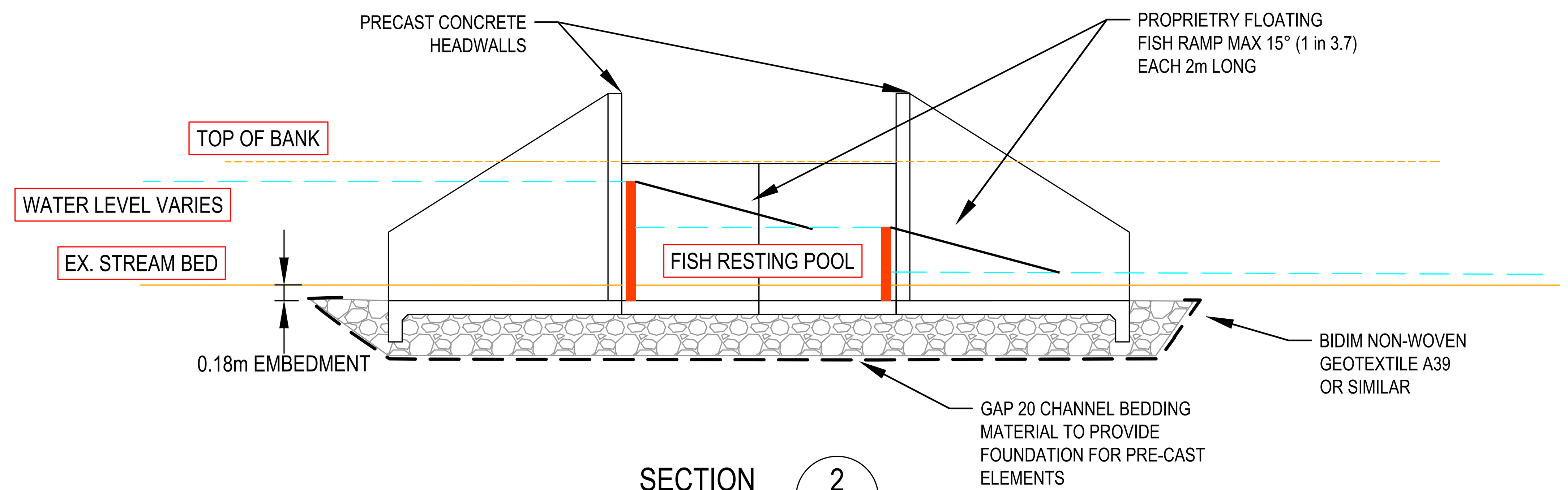

SECTION 2
SCALE 1:50

NOTES:

1. BASE DATA FROM DAVIS OGILVIE SURVEY OF MOAWHITU WETLAND, FEB 2020. LEVELS GIVEN ARE BASED ON THIS DATA.
2. ALL DIMENIONS ARE IN METERS
3. PLACEMENT OF EMBEDMENT, TRENCHFILL AND COMPACTION TO MEET REQUIREMENTS OF PRECAST MANUFACTURERS.
4. EXCAVATE STREAM BED TO PROVIDE FLAT FIRM BASE TO SUPPORT BEDDING MATERIAL TO MINIMISE SETTLEMENT.
5. ENSURE BEDDING IS FORMED AROUND PROJECTIONS SO THAT PRECAST PROJECTIONS DO NOT REST ON TOP OF BEDDING.
6. LOCATION AND LEVELS TO BE CONFIRMED PRIOR TO CONSTRUCTION
7. WEIR HEIGHTS SHALL HAVE A MAXIMUM HEIGHT DIFFERENCE OF 0.5m TO ENSURE FISH RAMPS ARE AT OPTIMAL GRADE

Issue	Date	Description	Des.	Veri.	App.
2	19/06/20	AMENDMENTS FOLLOWING CLIENT FEEDBACK			
1	15/06/2020	PRELIMINARY	KB	KB	KB

	Date
Drawn KB	15/06/2020
Checked KB	15/06/2020
Design/Surveyed ----	DD/MM/YY
Verified	DD/MM/YY
Approved	DD/MM/YY

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Unit 1a - 10 Surrey Street, Tawa
PO Box 56051 - Tawa - Wellington 5249

Client COLLOBORATIONS LIMITED
Project D URVILLE ISLAND
MOAWHITU WETLAND
WEIR CONSTRUCTION
Title WEIR STRUCTURE
Weir Details

Status FOR CONSENT
WGTN2000 | Ref --- | Scale AS SHOWN | Original Size A1
Drawing Number W20043-01-PL531 | Issue 2

DO NOT SCALE DRAWING 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

DATE: 19 June 2020 5:20 PM
Reference Files: ####
Drawing File: W20043-01-PL530-DRAINAGE-DETAILS-01.DWG

STOPLOGS

AWMA's Stoplog range consists of fabricated modular segments of any size, joined to effectively isolate flows for maintenance, re-direction or containment.

FEATURES

- Stoplogs are typically designed for installation and removal under equalised head conditions (no flow). AWMA design options include equalisation valves and roller guides to allow the Stoplogs to be operated under flow conditions.
- Custom designed and fabricated to suit any size or shaped orifice.
- Uni-direction sealing as standard with bi-directional models available on request.
- Insertion and removal of boards via AWMA's self engaging Lifting Frame.
- Storage solutions available.

APPLICATIONS

- The Stoplog range is utilised for applications across all industry sectors.
- Isolation of open channel flow for maintenance purposes.

STOPLOGS

DESIGN
DESIGN SUPPORT
- AWMA's design team will provide full support to ensure the most appropriate solution is developed and specified during the preliminary design.

SIZES
- All AWMA water control gates are custom sized to ensure they meet specific site and operational requirements.
- Customisation reduces installation costs.

MATERIALS
- AWMA select materials to meet a minimum design life of 25 years. Where required, AWMA can offer higher grade materials, coatings and protection systems to extend the design life to 100+ years.
- AWMA use ultra high molecular weight polyethylene (UHMWPE) for penstock door guides and/or wedges to provide maintenance free bearing surfaces.
- Plasticised PVC or EPDM are used for the manufacture of seals. These materials offer superior endurance in wastewater and freshwater applications.
- Materials used for penstock door and frames include marine grade aluminium and grades 304, 316, 2205 and 2507 stainless steel.
- Alternative material options are available to suit the application and/ or environment specific requirements.
- Materials used in the construction of the Stoplog range have a high corrosion resistance and can be operated for many years with minimal maintenance.

SEALING
- The sealing ability of this gate exceeds that required by the 'Australian Technical Specification for Fabricated Water Control Infrastructure'.

MAINTENANCE
- The Stoplog range has a minimum 25 year design life.
- Minimal maintenance is required offering low 'whole of life costs'.
- If required, all the wearing components can be changed, with ease, on site.

MANUFACTURE
QUALITY
- All fabrication is in accordance with the 'Australian Technical Specification for Fabricated Water Control Infrastructure'.
- All stainless steel welding is continuous to avoid crevice corrosion.
- All procedures are in accordance with AWMA's accredited ISO 9001 Quality Management System to ensure each gate is manufactured to a high standard, tested and ready for trouble free operation post approved installation.

INSTALLATION
MOUNTING OPTIONS
- The Stoplog range is typically wall mounted.
- The side frames can be face mounted or embedded.
- The sill is available in a raised or flat sill configuration.

ACTUATION SYSTEMS
- Mechanical lifting devices available.

OPERATION SYSTEMS
- Insertion and removal of boards via AWMA's self engaging Lifting Frame.

COMMISSIONING
DOCUMENTATION AND TRAINING
- Detailed documentation on operation, testing procedures and maintenance will be provided with all AWMA water control solutions.
- Comprehensive on and/or off site training available.

awma Water Control Solutions

Certified System Quality ISO 9001 SAI GLOBAL

HEAD OFFICE
Phone +61 3 5456 3331 Email info@awmawatercontrol.com.au
118 Roviras Road, PO Box 433, Cohuna Victoria 3568, Australia.

www.awmawatercontrol.com.au

Appendix C – NIWA HIRDS

Table A 1. NIWA HIRDS rainfall depths (mm) for Greville Harbour

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h
1.58	0.633	8.46	11.7	14.1	19.3	26.2	41.1	53	66.4	80.3	88.2
2	0.5	9.25	12.8	15.4	21.1	28.7	44.9	57.9	72.5	87.7	96.2
5	0.2	12	16.5	19.9	27.3	37	58	74.7	93.4	113	124
10	0.1	14.1	19.4	23.3	32	43.3	67.8	87.2	109	132	144
20	0.05	16.2	22.3	26.9	36.8	49.9	78	100	125	151	166
30	0.033	17.5	24.1	29.1	39.8	53.9	84.2	108	135	163	179
40	0.025	18.5	25.4	30.6	41.9	56.8	88.6	114	142	172	188
50	0.02	19.2	26.5	31.9	43.6	59	92.2	118	148	178	196
60	0.017	19.9	27.3	32.9	45	60.9	95.1	122	153	184	202
80	0.012	20.9	28.7	34.5	47.2	63.9	99.7	128	160	193	211
100	0.01	21.6	29.8	35.8	49	66.3	103	133	166	200	219
250	0.004	24.9	34.3	41.2	56.3	76.2	119	152	190	229	251

Appendix D – Additional modelling options

A 1-D hydraulic model has the ability to incorporate structures (such as the various culverts and spillways between the lake and wetland) and channel dimensions and roughness, which may have significant influences on water levels during large floods.

The current model lacks any lake and wetland storage. Bathymetry data exists for the lake while the wetland has a detailed DEM. Both data sets can be used to create stage storage (elevation-volume-area) tables which can be integrated into the models. Catchment areas would be refined to the stream inflow points only, with the rest of the catchment (the lake and wetland) considered to be an open ponded area.

A range of structures could be incorporated in the model, such as lake spillway and culvert parameters (diameter, slope and invert heights), bridge crossings and in addition, the proposed outlet control structure.

Survey data of the stream channels from the lake to the outlet can be integrated into a 1-D hydraulics model, incorporating cross sections and gradients. Furthermore, tide can also be considered as a king or high tide water level occurring at the same time as a design storm.

A 1-D model will provide water level, flow and velocity information for each of the nodes and cross sections. A simplified inundation map can be created by interpolating water level depth between cross sections. Given the comprehensive DEM available for the wetland, a 2-D model could also be constructed to simulate the design storm, which would then capture inundation extent with greater accuracy. This would also allow velocity profiles to be developed.