# Kyleakin Fish Feed Factory

Marine Harvest

## Kyleakin Flood Risk Assessment

B2261401_KYL|04

May 2017

# Flood Risk Assessment (FRA) Checklist

(SS-NFR-F-001 - Version 13 - Last updated 15/04/2015

***This document should be attached within the front cover of any flood risk assessments issued to Local Planning Authorities (LPA) in support of a development proposal which may be at risk of flooding. The document will take only a few minutes to complete and will assist SEPA in reviewing FRAs, when consulted by LPAs. This document should not be a substitute for a FRA.***

## Development Proposal

| | | | | | |
|---|---|---|---|---|---|
| Site Name | | Kyleakin | | | |
| Grid Reference | Easting: | 173790 | Northing: | 826430 | |
| Local Authority | | Highland Council | | | |
| Planning Reference number (if known) | | | | | |
| Nature of the development | | Commercial | If residential, state type: | | |
| Size of the development site | | 11.5 | Ha | | |
| Identified Flood Risk | Source: | Fluvial | Source name: | Alt Anavig | |

## Supporting Information

| | | | | |
|---|---|---|---|---|
| Have clear maps / plans been provided within the FRA (including topographic and flood inundation plans) | | Yes | | |
| Has a historic flood search been undertaken? | | Yes | | |
| Is a formal flood prevention scheme present? | | No | If known, state the standard of protection offered | |
| Current / historical site use | | Salt and Gravel Quarry | | |

## Hydrology

| | | | | | |
|---|---|---|---|---|---|
| Area of catchment | | 5.5 | $km^2$ | | |
| Qmed estimate | | 8.1 | $m^3/s$ | Method: | Catchment Descriptors |
| Estimate of 200 year design flood flow | | 30.1 | $m^3/s$ | | |
| Estimation method(s) used * | | Rainfall-runoff | | If other (please specify methodology used): | REFH2 |
| | | | | If Pooled analysis have group details been included | Select from List |

## Hydraulics

| | | | | | | |
|---|---|---|---|---|---|---|
| Hydraulic modelling method | | Linked 1D 2D | | Software used: | TuFlow | |
| If other please specify | | | | | | |
| Modelled reach length | | 235.8 | m | | | |
| Any structures within the modelled length? | | Culvert | | Specify, if combination | | |
| Brief summary of sensitivity tests, and range: | | | | | | |
| variation on flow (%) | | 20 | % | | | |
| variation on channel roughness | | 20 | | | | |
| blockage of structure (range of % blocked) | | 50% | % | Reference CIRIA culvert design guide R168, section 8.4 | | |
| boundary conditions: | | **Upstream** | | | **Downstream** | |
| (1) type | | Flow | | | Tidal | |
| | Specify if other | | | Specify if other | | |
| (2) does it influence water levels at the site? | | No | | | Select from List | |
| Has model been calibrated (gauge data / flood records)? | | No | | | | |
| Is the hydraulic model available to SEPA? | | No | | | | |
| Design flood levels | 200 year | See notes | m AOD | 200 year plus climate change | See notes | m AOD |

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## Coastal

| | | | | | | |
|---|---|---|---|---|---|---|
| Estimate of 200 year design flood level | | 3.85 | m AOD | | | |
| Estimation method(s) used | | Tide gauge analysis | If other (please specify methodology used): | Coastal Flood Boundary (CFB) method | | |
| Allowance for climate change (m) | | 2.5 | m | | | |
| Allowance for wave action etc (m) | | 0 | m | | | |
| Overall design flood level | | 6.53 | m AOD | | | |

## Development

| | | | | | | |
|---|---|---|---|---|---|---|
| Is any of the site within the functional floodplain? (refer to SPP para 255) | | No | | If yes, what is the net loss of storage | | $m^3$ |
| Is the site brownfield or greenfield | | Brownfield | | | | |
| Freeboard on design water level (m) | | 1.56 & 0.325 | m | | | |
| Is the development for essential civil infrastructure or vulnerable groups? | | Select from List | | If yes, has consideration been given to 1000 year design flood? | Yes | |
| Is safe / dry access and egress available? | | Vehicular and Pedestrian | | Min access/egress level | 7.1 | m AOD |
| If there is no dry access, what return period is dry access available? | | 200 | years | | | |
| If there is no dry access, what is the impact on the access routes? | Max Flood Depth @ 200 year event: | | m | Max Flood Velocity: | | m/s |
| Design levels | Ground level | 8 | m AOD | Min FFL: | 8.25 | mAOD |

## Mitigation

| | | |
|---|---|---|
| Can development be designed to avoid all areas at risk of flooding? | | Yes |
| Is mitigation proposed? | | Yes |
| If yes, is compenstory storage necessary? | | No |
| Demonstration of compensatory storage on a "like for like" basis? | | Yes |
| Should water resistant materials and forms of construction be used? | | No |

## Comments

| | | |
|---|---|---|
| Any additional comments: | | Design flood levels at reservoir: pre-development 13.49mAOD; post-development 12.95mAOD (-0.54m); at lagoon: post-development 7.44mAOD; and existing culvert inlet: pre-development 8.321mAOD; post-development 8.175mAOD (-0.146m) |

**Approved by:**
**Organisation:**
**Date:**

Note: Further details and guidance is provided in 'Technical Flood Risk Guidance for Stakeholders' which can be accesssed here:- **CLICK HERE**

**** ReFH2 is now accepted by SEPA for flow estimates in Scotland. Any use of this method should be compared with other accepted methods.***

## Project Name

| | |
|---|---|
| Project No: | B2261401 |
| Document Title: | Kyleakin Flood Risk Assessment |
| Document No.: | |
| Revision: | |
| Date: | May 2017 |
| Client Name: | Marine Harvest |
| Project Manager: | Joanne Moran |
| Author: | Luke Bland |
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Jacobs U.K. Limited

1 City Walk
Leeds, West Yorkshire LS11 9DX
United Kingdom
T +44 (0)113 242 6771
F +44 (0)113 389 1389
www.jacobs.com

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### Document history and status

| Revision | Date | Description | By | Review | Approved |
|---|---|---|---|---|---|
| P01 | October 2016 | Kyleakin Flood Risk Assessment | Luke Bland | Phil Raynor | Phil Raynor |
| P02 | March 2017 | Kyleakin Flood Risk Assessment addressing SEPA conditions | Luke Bland | Amanda Kitchen | Alan Brown |
| P03 | May 2017 | Minor amendments to address SEPA comment | Phil Raynor | Amanda Kitchen | Alan Brown |
| P04 | May 2017 | Bund removed from mitigation | Phil Raynor | Amanda Kitchen | Alan Brown |
| | | | | | |
| | | | | | |

## Contents

# 1. Introduction

## 1.1 Background

Marine Harvest is developing a new fish feed factory within the Allt Anavig Quarry, just west of Kyleakin, a small village situated on the east coast of the Isle of Skye, Inner Hebrides (National Grid Reference (NGR) 173790, 826430). A site plan is presented overleaf in Figure 1.1.

The Allt Anavig flows through the site, passing first through an on-line waterbody to the south west of the site before running in open channel and culvert to discharge to the sea. The existing flood risk from this watercourse is a constraint on development. This flood risk assessment assesses this risk, identifying how it is to be managed and the nature of any residual risk.

The site's coastal location also requires consideration of coastal flood risk, and the risk from surface water and groundwater sources is also considered.

## 1.2 Aims and Objectives

Jacobs has been commissioned by Harvest Marine to undertake a Flood Risk Assessment (FRA) of the proposed site for the new factory. The objectives of this study are to:

- Assess the existing sources of flood risk to the site;
- Assess the impact on flood risk as a result of the development of the scheme;
- Identify how these flood risks are to be managed; and
- Identify any residual risks and any potential further mitigation measures that could be incorporated.

This flood risk assessment has been carried out in accordance with SEPA's *Technical flood risk guidance for stakeholders* (Ref. SS-NFR-P-002, SEPA, 2015) and the SEPA flood risk assessment checklist is attached as a cover sheet.


**Figure 1.1: Site location plan**

# 2. Background site data

## 2.1 Topography and Land Use

The site is located on the northern shore of southern Skye immediately adjacent to the Kyle Akin narrows and the Skye Bridge crossing from the mainland from which the site can be viewed. It is part of a wider active quarry location although this part of the quarry has been heavily worked into a flat-bottomed, open-fronted ‘bowl’ with access to the sea via a substantial jetty/pier. The higher land between these workings and the A87 trunk road, running to the south of the site, is heavily wooded, as is the rising land to the south of the road. There is an existing quarry access from the A87.

The site lies within the Allt Anavig Quarry, the ground levels within which vary from approximately 7.75 to 6.00m AOD (see Appendix A for pre-development ground levels). Land surrounding the quarry rises up to approximately 30m AOD and continues to rise in a south easterly direction towards Beinne na Greine (611m AOD) and Sgùrr na Coinnich (739m AOD). The upper catchment is mountainous which is likely to be associated with shallow soils and peat, whilst the lower flanks are forested with significant stands of conifers. Aerial photographs indicate that these undergo occasional felling. There are likely therefore to be surface water drainage features associated with the forestry that influence the hydrology of the catchment.

## 2.2 Photographs

Photographs of the Allt Anavig reservoir, existing overflow and the route of the proposed overflow are given in Appendix B.

## 2.3 Existing flood alleviation measures

There are no existing flood alleviation measures in place.

## 2.4 Existing culverts

The Allt Anavig reservoir overflow runs in culvert downstream of the reservoir to the sea outfall. The culvert is 210 metres long with a fall of approximately 4.9 metres (from 5.3mAOD to 0.4mAOD) giving a bed slope of 0.023 or 1 in 43. At the inlet, the culvert construction comprises concrete side walls and steel soffit, whilst at the outlet it comprises a large diameter HDPE structural walled culvert pipe. The location of the transition from one construction to the other unknown. The condition of the culvert is unknown, although the outfall is in good condition.

## 2.5 Existing/historic data

There is no readily available historic flood incident data for the site.

# 3. Methodologies

## 3.1 River flows and hydrographs

### 3.1.1 Catchment description

The quarry lies and near the outlet of the Allt Anavig catchment, which is approximately 5.5km$^2$ in size at the inlet to the culvert within the Kyleakin site. It receives runoff from the western flank of Beinne na Greine and Sgùrr na Coinnich via the Allt na Pairce-fraoich and Allt Lochain na Saile, which itself drains Lochan na Saile and two other slightly larger lochans.

The catchment receives a high amount of rainfall, averaging over 2,000mm per annum, and the catchment is considered wet approximately 80% of the time. The percentage runoff is high (over 50%) and the catchment is typically steep.

Allt Anavig flows in a northerly direction and into a man-made reservoir located directly above the quarry before entering a culvert and flowing into Kyle Akin. Within the quarry there is also a lagoon which seems to be largely derived from surface water runoff.

### 3.1.2 Flood estimation

The Allt Anavig catchment is ungauged and design flood flows were estimated using both the Flood Estimation Handbook (FEH) statistical method and ReFH2 method. ReFH2 was the preferred method as this produced greater flows. Hydrographs were estimated using ReFH2. There is some uncertainty in the results from both methods. In the absence of gauged data within the catchment and a suitable donor station(s) for data transfer not explored at this stage, both methods are reliant on the catchment descriptors. In similar situations, SEPA has preferred to take the larger of the estimates, i.e. ReFH2 (2013) and the same approach is recommended for adoption here in the absence of monitoring information.

The 0.5% AEP flow is 30.1m$^3$/s and 0.5% plus climate change flow is 36.1m$^3$/s.

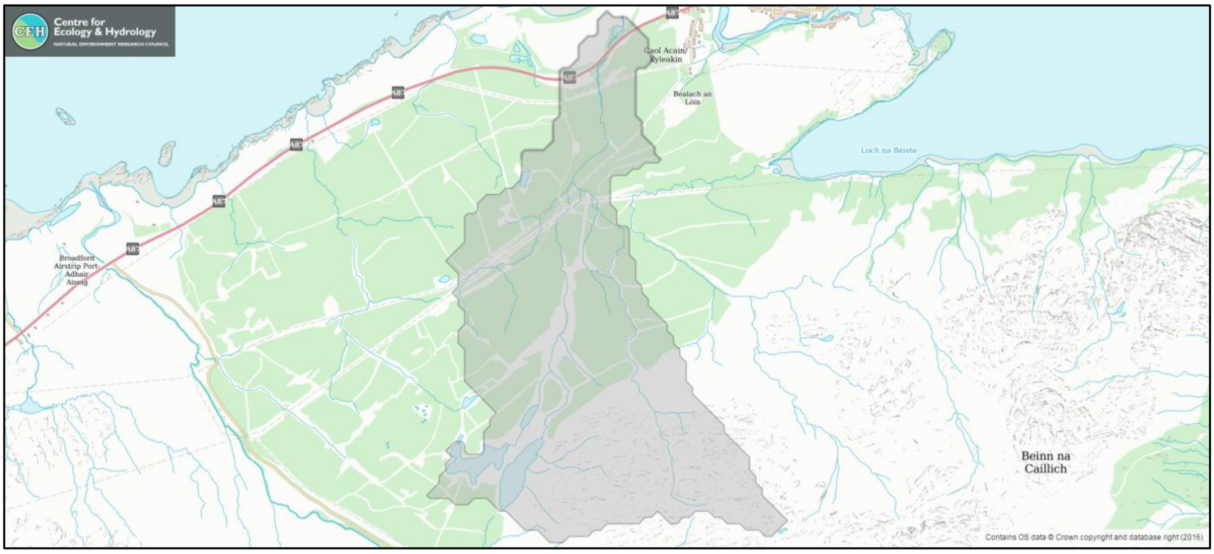



**Figure 2-1: Allt Anavig catchment area**

## 3.2 Coastal flood levels

Coastal flood levels for the site were derived in line with the Coastal Flood Boundary (CFB) method. Still water sea levels were obtained for two primary sites north and south of Kyleakin: Tobermory and Ullapool. Astronomical tide levels were obtained from the National Tide and Sea Level Facility (www.ntslf.org) (Table 3-1) and extreme levels from the *Coastal flood boundary conditions for UK mainland and islands* (Environment Agency, 2011) (Table 3-2). Levels for Kyleakin, which sits approximately half-way between the two primary sites, were obtained by interpolation.

**Table 3-1: Design sea levels**

| Site | MLWS | MLWN | MHWN | MHWS |
|---|---|---|---|---|
| Tobermory (mAOD) | -1.53 | -0.36 | 1.12 | 2.31 |
| Ullapool (mAOD) | -1.99 | -0.57 | 1.23 | 2.46 |
| *Kyleakin (mAOD)* | *-0.88* | *-0.47* | *1.18* | *2.39* |

1. Data from National Tide and Sea Level Facility.

**Table 3-2 Extreme still water sea levels**

| Site | Chainage (km) | 1% AEP | 0.5% AEP | 0.1% AEP |
|---|---|---|---|---|
| Tobermory[1] (mAOD) | 2,320 | 3.74 | 3.87 | 4.18 |
| Ullapool[1] (mAOD) | 2,564 | 3.76 | 3.82 | 3.96 |
| *Kyleakin (mAOD)* | *2,440* | *3.75*[2] | *3.85*[2] | *4.03*[3] |

1. Data for primary sites obtained from Environment Agency (2011) Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR2: Design sea levels, Table 4.1.
2. Interpolated.
3. SEPA.

**Table 3-3 Forecast sea level rise and water levels**

| Site | | 2095 central estimate | H++ scenario | Still water level |
|---|---|---|---|---|
| Edinburgh (mm) | | 230 to 390 | *928 to 2500* | |
| Belfast west coast (mm) | | 245 to 403 | | |
| *Kyleakin (mAOD)* | | | *2500* | *6.53* |

1. UKCP09

## 3.3 Design water levels

Design water levels for the Allt Anavig reservoir and lagoon were estimated using a 1D-2D hydraulic model (Flood Modeller-TuFlow) The hydraulic modelling report is given in Appendix C. The reservoir was modelled pre- and post-development whilst the lagoon was modelled post-development only. For the 0.5% AEP with climate change event, modelled water levels are given in Table 3-4.

**Table 3-4 Modelled water levels in reservoir and lagoon**

| Site | Pre-development | | Post-development | |
|---|---|---|---|---|
| | Initial | Peak | Initial | Peak |
| Reservoir (mAOD) | 12.19 | 13.49 | 11.69 (-0.50) | 12.95 (-0.54) |
| Lagoon (mAOD) | - | - | 5.6 | 7.44 |

## 3.4 Joint probability analysis

The new reservoir overflow will discharge to the sea via a new open channel with an invert level of 2.50 to 4.04mAOD. The 0.1% AEP still water sea level (4.03mAOD) is just below the invert level of the culvert outlet (4.04mAOD). The tailwater level will be insufficient to force a switch from inlet control to outlet control conditions and headwater level will not be influenced by tailwater conditions. As a result, joint probability analysis was not considered necessary.

## 3.5 Geology and hydrogeology

The underlying strata of the catchment consist of a mixture of sandstones and mudstones (Kinloch Formation) in the upper catchment and sandstones (Applecross Formation) in the lower catchment, intersected in places by Microgabbro and Basalt dykes. There are notable areas of till and morainic deposits, on the flanks of Beinne na Greine and in the valleys of the watercourses, as well as alluvium. The lower catchment also contains raised marine deposits of gravel, sand and silt.

The catchment in general is considered to have low permeability with little groundwater[1], presumably because of the combination of sandstone and overlying till, which in combination with the steep nature of the upper catchment is likely to contribute to a rapid and potentially high runoff response to rainfall. It is also likely that the sands and gravels in the lower catchment contribute to locally higher permeability.

Groundwater levels within the site vary between 1.3m bgl and 5.82m bgl, with the higher levels located to the south of the site. It is expected that groundwater may vary in response to the prevailing wetness of the seasons and may be close to the surface at times in the lower lying areas.

## 3.6 Sensitivity analysis

Sensitivity to flow was tested for 0.5% AEP and 0.5% AEP CC flows plus 20%. The proposed overflow channel itself is not sensitive to increased flow as the flow is controlled by the inlet to the 2.5 metre diameter pipe culvert. However, variations in flow alter the flow split from the reservoir and are reflected in the existing channel. Sensitivity testing showed that the proposed design has no capacity in excess of the 0.5% AEP CC event ($36m^3/s$). Any increase in flow above this will result in flooding of the site. The design is therefore sensitive to uncertainties in the flow and the site is at risk of flooding as a consequence. The surrounding ground levels should be set to prevent overtopping and bypassing of the existing culvert.

Sensitivity to roughness was tested for Manning roughness coefficient +/- 20%. Peak water levels in the model are moderately sensitive to roughness but the proposed design remains in-bank during the design scenario plus 20% and therefore flood risk at the site is not sensitive to roughness.

To test the sensitivity of the model results to the downstream tidal boundary, an extreme tidal event was run for the pre- and post-development scenarios. A tidal cycle with peak levels corresponding to a 0.5% AEP event (4.03mAOD) plus 2095 climate change allowances were tested. The central estimate climate change is 0.2m and H++ estimate is 2.5m, giving still water levels of 4.23mAOD and 6.53mAOD respectively. The site is elevated at +8mAOD so no influence on flood risk from the downstream boundary was identified.

Culvert blockage at 10% and 50% was carried out separately for the existing and proposed culverts; no combined blockage was tested. Blocking the proposed pipe culvert by 10% alters the flow split from the reservoir, sending more flow down the existing channel which results in flooding to the external areas of the site but not buildings. At 50% blockage, both site and buildings are flooded. The findings are similar for a blockage of the existing culvert: 10% blockage results in some flooding of the site and 50% flooding of both buildings and site.

In conclusion, the design has been optimised at 0.5% AEP CC flood ($36m^3/s$) and there is no capacity within the design to deviate from this. Furthermore the design is highly sensitive to the parameters controlling the correct flow split from the reservoir (option inlet design and invert as well as the existing outlet weir). Variations in design and or model parameters may result in flooding of the site.

[1] https://fehweb.hydrosolutions.co.uk/GB/map

# 4. Assessment of Flood Risk

## 4.1 Introduction

Jacobs has undertaken hydrological and hydraulic flood modelling[2] (Appendix C) in order to understand the potential risk to the proposed development. This section considers the existing flood risk to the proposed works areas from all sources. Table 4-1 summarises flood risk from all sources.

**Table 4.1 Summary of Flood Risk**

| Source of flooding | Existing Flood Risk to the Development | Flood Risk from the Development | Residual Flood Risk |
|---|---|---|---|
| Coastal | Low | Low | Low |
| Fluvial | High | Low | High |
| Pluvial | Low | Low | Low |
| Groundwater | Low | Low | Low |
| Reservoir | Low | Low | Moderate |
| Canal | Low | Low | Low |
| Sewer and artificial drainage | Low | Low | Low |

The site is located away from urban areas and does not appear to have any manholes at the site signalling presence of utilities or a sewer network. Therefore flood risk from sewers will not be considered further.

There is no canal within close proximity to the site, hence this source of flooding will not be considered further.

## 4.2 Coastal Flood Risk

Coastal flooding can be caused by high astronomical tides, particularly when these coincide with a low-pressure storm system which locally raises sea and coastal water levels (tidal surge), overwhelming coastal and river defences. These factors can be made worse by strong winds blowing the raised body of water up coastal river basins some distance from the coast. Such flooding may become more frequent in future years due to rising sea levels.

**Existing Flood Risk**

SEPA's online flood risk maps indicate that the site is considered to be at low risk of coastal flooding, however, the presence of the site adjacent to and open to the coast to the north, presents a possible risk of flooding from this source. SEPA has provided an indication of the still water flood level of 4.03m AOD at this point for the 0.5% AEP[3] coastal flood event. This level is lower than the land levels within the quarry, which are a minimum of 6.0m AOD, the exception being the outlet and inlet of the existing culvert, which lie below 4.0m AOD.

It is possible therefore, that high tide levels could propagate up the existing culvert and cause localised ponding within the channel at this point. However, with a difference in level of 1.97m between the still sea water level and the ground levels within the quarry, there would be no inundation of the quarry. This means that coastal flooding is not an issue by itself.

[2] Allt Anavig Hydraulic Modelling Report, Jacobs (2016)
[3] Flood frequency can alternatively be expressed in terms of an annual exceedance probability (AEP), the probability of an event being equalled or exceeded in any given year, which is the inverse of the return period. For example, the 1 in 100 year flood can be expressed as the 1% AEP flood, which has a 1% chance of occurring in any year.

A slightly more extreme coastal scenario has been hydraulically modelled that considers the joint probability of reasonably high flows within the Allt Anavig, equivalent to the 10% AEP fluvial flood event, and the 0.5% AEP sea level, with an allowance for climate change. In this scenario, hydraulic modelling indicated that the existing site would be at risk from flooding, principally due to a reduction in capacity of the culvert, with fluvial flows overtopping the culvert inlet and inundating the site. The results of this modelling are presented in Figure 11 of the Modelling Report, which is provided in Appendix C.

The jetty and assets located on the jetty may be at higher risk of coastal flooding, however, these are designed to be at a level of 8.0m AOD or higher and therefore well above extreme coastal levels and climate change levels as explained in Section 6.

This report has also considered the joint probability flood risk from combined fluvial and coastal flows as highlighted in Figure 5-3.

**Development Impact on Coastal Flood Risk**

Both ground levels (8mAOD) and finished floor level (8.25mAOD) of the site are of a sufficient level as to not lie within the extreme sea level of 6.53mAOD. Based on the information discussed above, it is reasonable to conclude that the proposed development will not increase the coastal flood risk to the surrounding area and that the risk to the proposed site is low.

## 4.3 Fluvial Flood Risk

Fluvial flooding typically occurs when a river's capacity is exceeded and the excess water overtops the river banks. It can also occur when the watercourse has a high level downstream, perhaps due to structures or blockage, thus limiting the amount of discharge. This creates a back-up of water and again water overtops the banks. Typical flooding issues occur when the natural floodplain has been urbanised and where the river has been constrained.

**Existing Flood Risk**

The main risk of flooding to the site comes from the Allt Anavig. Whilst the catchment is not large, the steep nature of the catchment, the geology of the upper catchment and the significant rainfall volumes result in large flows and the potential for flooding within the site.

The existing culvert is shown to have insufficient capacity to pass the flow for the full range of flood events, which results in overtopping at the inlet and flow across the site. Figure 4.1 shows the fluvial flood extents that may occur during a 0.5%AEP + climate change event. The culvert capacity as modelled was found to be $13.9m^3/s$, which is below the estimated peak flow for the 20% AEP event of $14.2m^3/s$.

The risk of flooding from the watercourse is therefore considered to be high and the modelled extents presented within Figure 4-1 would suggest that the entire quarry floor could be at risk under extreme conditions. Blockages poses a residual risk, particularly in light of the wooded nature of the streambanks, though given the mechanisms of flooding it is unlikely to increase the consequences of flooding, only the likelihood of its occurrence.

Development of the site will need to consider the risk posed from the Allt Anavig and manage the risk accordingly to ensure that the buildings, materials and other assets and receptors are not at risk.

**Development Impact on Fluvial Flood Risk**

The proposed new channel has been designed to be flood free in a 0.5% AEP plus climate change event. The predicted catchment flows in a 0.5% AEP plus Climate Change and 0.1% AEP events are similar and therefore the new channel design predicts the site to be flood free in a 0.1% AEP flood event. Although there is a significant reduction in flood extent during the 0.5% AEP + climate change event there is still some slight flooding adjacent to the existing watercourse due to the water slowing down as it enters the culvert causing a slight back up of water (see Figure 4-2).

As a result of the design of the diversion, the fluvial flood risk posed to the new site is considered to be low. There remains, however, a residual risk from blockage of culverts and failure of the reservoir, which is discussed below.

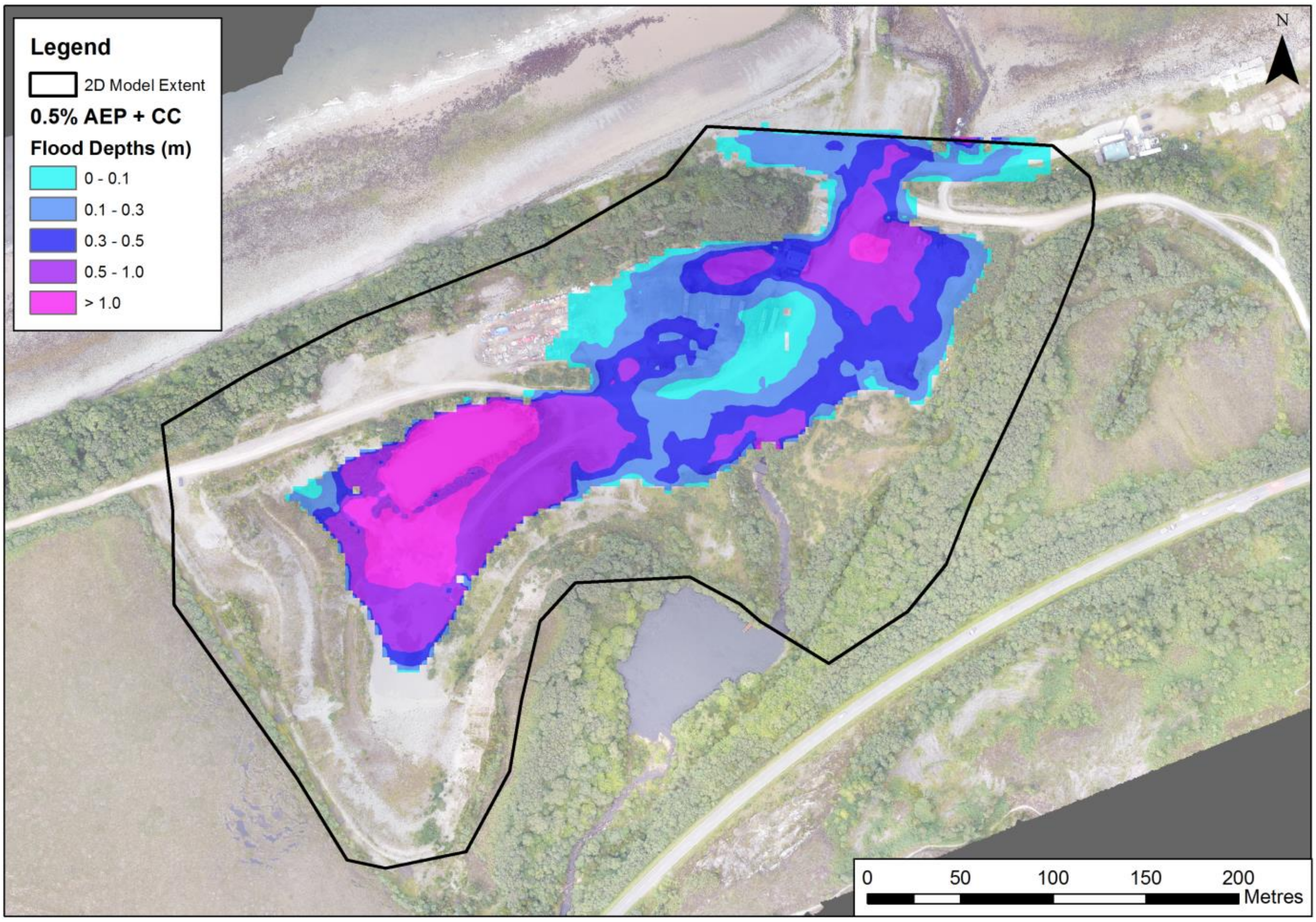



**Figure 4-1: Fluvial flood extents from a 0.5% AEP + Climate Change flood event (Source: Allt Anavig Hydraulic Modelling Report)**

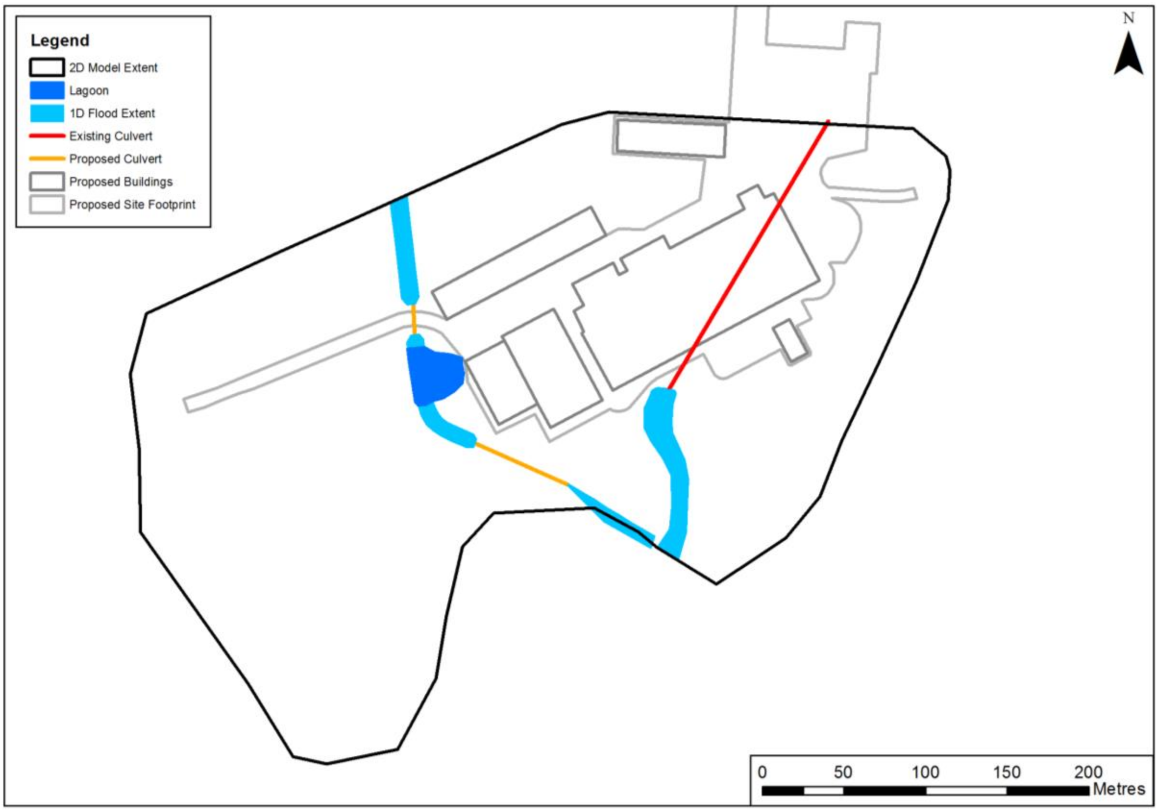



**Figure 4-2: Design Option 0.5% AEP + Climate Change Fluvial Flood Extent**

## 4.4 Pluvial Flood Risk

Pluvial flooding is defined as water flowing over the ground that has not yet entered a drainage channel or similar. It usually occurs as a result of an intense period of rainfall which exceeds the infiltration capacity of the ground. Typically, runoff occurs on sloping land or where the ground surface is relatively impermeable. The ground can be impermeable either naturally due to the soil type or geology, or due to development which places impervious material over the ground surface (e.g. paving and roads).

**Existing Flood Risk**

SEPA's flood maps show minor pockets of ponding within the site associated with the pond in the west of the quarry and depressions elsewhere. The extent of this flooding is the same whether considering high risk areas that flood frequently (i.e. those that flood in events equivalent to a 1 in 10 year storm), or low risk areas (i.e. those that flood in a 1 in 1000 year storm). This suggests that the risk from surface water is limited, which may reflect the permeability of land within which the quarry is constructed.

As the nature of the flood risk shown in the SEPA maps is limited to lower lying areas within the site, it is considered that surface water flood risk is generally low.

**Development Impact on Pluvial Flood Risk**

There may be temporary changes in surface water flood risk across the area during construction. For example, heavy vehicles may compact permeable ground, thereby decreasing permeability and increasing surface water runoff, and creating surface water flow paths leading to and from the construction areas. Site traffic may also mobilise sediment, increasing sediment load and sedimentation along flow paths.

A surface water drainage system, managing runoff for events up to and including the 0.5% AEP event, is proposed within the site that will capture runoff and direct it to the watercourse in the vicinity of the lagoon to the west of the site. The western section of the lagoon is also being proposed to be used SUD's in order to accommodate excess overland flows. Surface water within the site will therefore be managed to prevent impacts within the site.

## 4.5 Flooding from Groundwater

Groundwater flooding is caused by the emergence of water from beneath the ground at either point or diffuse locations when the natural level of the water table rises above ground level. This can result in deep and long lasting flooding of low-lying or below ground infrastructure such as underpasses and basements. Groundwater flooding can cause significant damage to property, especially in urban areas, and can pose further risks to the environment and ground stability.

**Existing Flood Risk**

Groundwater is present, though its variability over time is unknown. It has been assumed that any emergence of groundwater, were it to occur, would typically flow towards lower ground in the same way as surface water flooding is shown to do by SEPA's surface water flood maps. Consequently, groundwater is not expected to pose a significant risk within the current site and management of this source of flooding would typically be dealt with in the same manner as for surface water flooding.

**Development Impact on Groundwater Flood Risk**

The proposed new development and associated infrastructure (e.g. foundations, drainage) may provide a small barrier to groundwater movement but it is not anticipated to create a measureable increase in groundwater flood risk to nearby receptors. Groundwater flood risk to the development is considered to remain low, particularly as the proposed diversion channel would create a new route to enable groundwater to leave the site without impacting the buildings proposed.

## 4.6 Flood Risk from Reservoirs

Reservoir failure can be a particularly dangerous form of flooding as it results in the sudden release of large volumes of water that can travel at high velocity. This can result in deep and widespread flooding, potentially resulting in significant damage. The likelihood of reservoir flooding occurring is generally extremely low given that all large reservoirs are managed in accordance with the Reservoirs (Scotland) Act 2011 (which supersedes the Reservoir Act 1975).

**Existing Flood Risk**

The on-line waterbody immediately upstream of the quarry through which the Allt Anavig flows has been referred to as a reservoir. It should be noted that this is not classified under the Reservoirs (Scotland) Act 2011. As such, SEPA's online reservoir inundation maps do not cover the waterbody and no flood risk extent is identified as a result. However, despite the small size of the waterbody, there is a risk that failure of the spillway could increase flows within the Allt Anavig and the existing site would be flooded.

No modelling of this scenario has been undertaken, however, failure of the reservoir has been simulated as part of an assessment of residual risks for the proposed site. This is discussed further in Section 5.

**Development Impact on Reservoir Flood Risk**

The size, location and function of the proposed development within the area will not increase the risk of reservoir flooding to the surrounding area or to any existing infrastructure in this area. Therefore, the impact of the development on reservoir flood risk remains low. Nevertheless, caution must be taken when constructing next to the Reservoir to avoid disturbing the structure or foundations and increasing or causing weaknesses.

Regular inspection and maintenance of the reservoir, as well as the diversion channel and new structures will ensure any structural issues are identified early and a flood risk emergency plan to manage the residual risks from this source is recommended.

## 4.7 Impacts of Climate Change on Flood Risk to the Site

Climate change modelling and guidance indicates that the frequency and severity of storm events will increase in the future and consequently higher rainfall intensities are expected.

Rising sea levels and more frequent / severe storm events focus wave energy closer to the shore and cliff faces, which leads to increased levels of coastal erosion. Resting sea levels will naturally be higher therefore the risk of flooding in coastal or storm events will be greater. With the increased rainfall intensities expected due to climate change, the risk of fluvial and surface water flooding can be expected to increase without mitigation.

This is also likely to affect any drainage or sewer networks in the area, as increased rainfall is likely to result in an increased chance of an overloaded drainage network or sewer surcharge.

The UKCP09 projections indicate that it is likely there will be much more variability in rainfall patterns which could lead to very high rainfall but also periods of very low rainfall. This will impact on groundwater as well and is likely to result in increased frequency and severity of groundwater related floods.

Whilst reservoirs are judged to be fairly resilient to climate change due to stringent monitoring and construction checks, the threat of increased erosion and fluctuating water levels could result in increased flood risk. This is particularly applicable if the reservoir has existing weaknesses in the structure. Monitoring of the structure on a regular basis is recommended in order to understand if there are any weaknesses which could be exploited in the process of climate change.

## 4.8 Design Mitigation

The dominating factor in the flood risk to the proposed fish feed factory is the exceedance of culvert capacity. The design involves diversion of the watercourse around the west of the quarry, via the existing lagoon, and then towards the coast in a northern direction. The open channels and culverts have been designed to convey flows up to the 0.5% AEP plus climate change. The nature of the diversion and details are shown in Figure 4 of Appendix D.

In addition, floor levels within the factory are to be raised above surrounding ground level, to a level of 8.25m AOD. The proposed final ground levels for the site allow for dry safe access and egress from the building and away from site. It is important for the access track to be of a sufficient height to allow for this to take place however; residual risk places this at risk (see Section 5.7).

## 4.9 Structures affecting local hydraulics

### 4.9.1 Allt Anavig Culvert

The existing Allt Anavig culvert conveys overflows from the reservoir to the sea. The culvert comprises a 210 metres long , 2.0m diameter culvert with a fall of approximately 4.9 metres (from 5.3mAOD to 0.4mAOD) giving a bed slope of 0.023 or 1 in 43.

The Allt Anavig culvert has a discharge capacity of 13.9m$^3$/s, significantly less than the 0.5% AEP flow of 30.1m$^3$/s and 0.5% plus climate change flow of 36.1m$^3$/s.

The proposed second reservoir overflow will provide some relief to the Allt Anavig culvert, which post-development will be required to convey no more than 13.9m$^3$/s.

### 4.9.2 Proposed Pipe Culvert

The proposed 2.5m diameter pipe culvert will convey flows from the new reservoir overflow to the quarry floor. The 64.3 metre long pipe will drop from 9.711mAOD to 5.925mAOD with a uniform bed slope of 0.059 or 1 in 17. From the quarry floor, the flow will be conveyed in a trapezoidal open channel to the lagoon.

For the design flow of 22$m^3$/s, the culvert will operate under submerged flow inlet control conditions with a submerged inlet and free flow in the barrel. The barrel will start overtopping at a level of 14.00m AOD and a flow of 25.2$m^3$/s.

### 4.9.3 Proposed Box Culvert

The proposed 3.6 x 2.4m box culvert will convey flows from the lagoon beneath the quarry access track, before discharging to the sea via a 60 metre long two-stage channel. The 18 metre long culvert will drop from 5.09mAOD to 4.04mAOD with a uniform bed slope of 0.0583 or 1 in 17.

For the design flow of 22$m^3$/s, the culvert will operate under submerged flow inlet control conditions with a submerged inlet and free flow in the barrel. The barrel will start overtopping at a level of 8.00m AOD and for a flow of 30$m^3$/s.

### 4.9.4 Open Channel to Marine Outfall

The open channel between the culvert and marine outfall will be semi-natural to provide habitat and encourage fish passage. A natural scour hole will be allowed to develop immediately downstream of the box culvert with a constructed step-pool series further downstream towards the shoreline. The step-pool series will have a step height of 0.2m suitable for salmonids and a pool length of 8.2m. The steps will be created by installing lines of boulders to act as weirs, curved in plan to create a slightly longer effective weir crest and to ensure that any scour is focussed in the centre of the channel.

At the marine outfall, an armourstone apron will be constructed to dissipate energy and prevent erosion of the foreshore, whilst providing habitat.

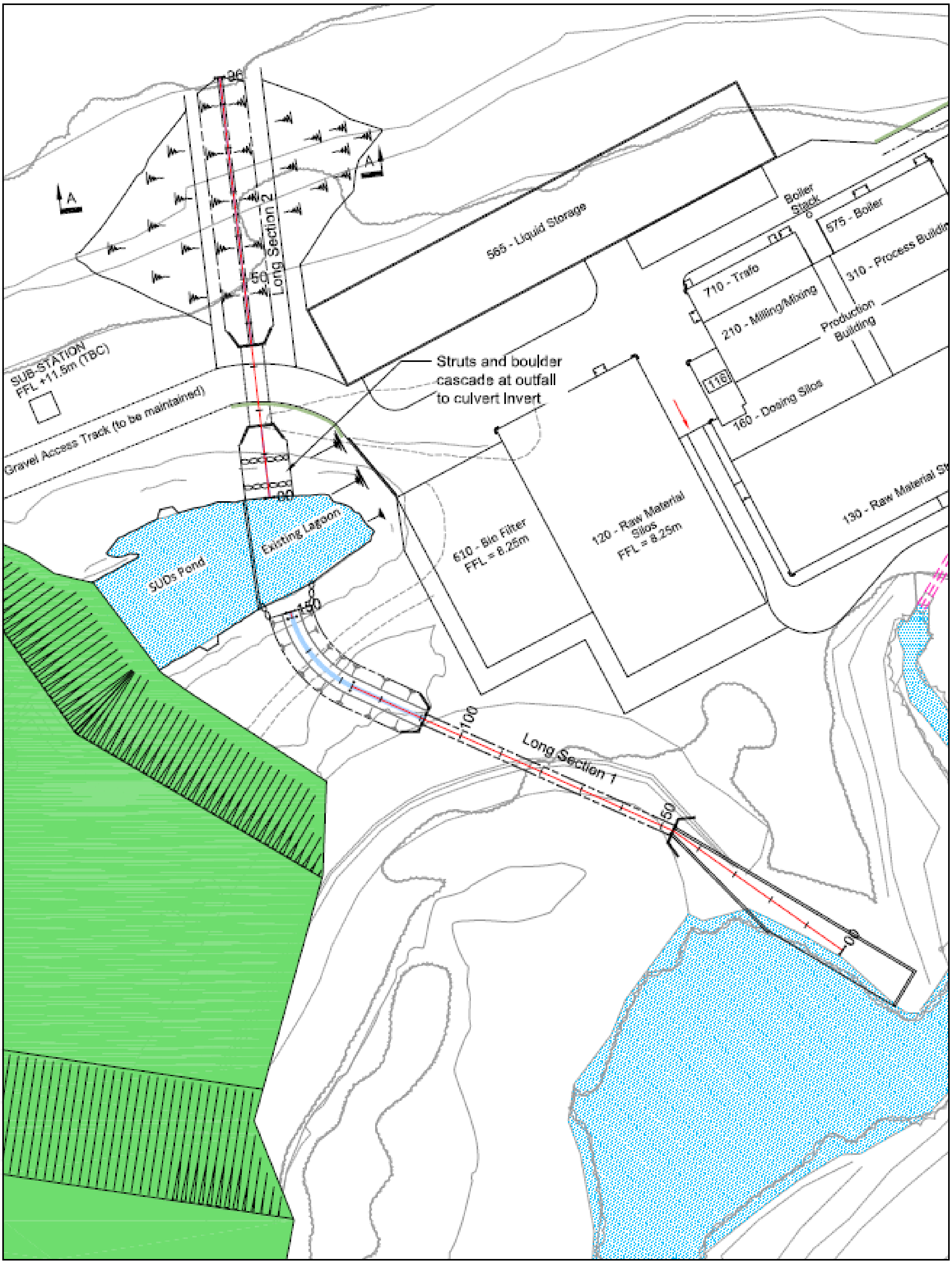



**Figure 4-3: Proposed 2.5m diameter pipe culvert and 3.6 x 2.4m box culvert**

## 4.10 Impacts of culverts

The impacts of 10% blockage were assessed using hydraulic modelling. This showed that for both the existing culvert and the proposed 2.5m diameter pipe culvert the external areas of the site would flood in a 0.5% AEP plus CC flood event.

The impacts of 50% blockage were assessed using hydraulic modelling. This showed that for both the existing culvert and the proposed 2.5m diameter pipe culvert the external areas of the site and buildings would flood in a 0.5% AEP plus CC flood event.

Further detail is given in Section 5.

## 4.11 Potential impacts on morphology, habitat and ecology

### 4.11.1 Reservoir to lagoon

The proposed reservoir spillway, pipe culvert and stilling basin will operate under conditions of high flow velocities and turbulence. These sections will be constructed using hard engineering (concrete) to dissipate energy and prevent scour, and will provide little habitat.

Between the stilling basin and lagoon, the proposed trapezoidal open channel with 1:2 sloping sides will convey flow at slower velocities. This reach has a mild bed slope and the channel lining will be relatively smooth and hydraulically efficient to convey the flow. Nevertheless, this reach will be more naturalised and has the potential to provide some habitat. Coarse sediment deposition is likely as the flow velocity decreases.

The open channel will discharge to the lagoon with an estimated flow velocity of 2.2m/s for the 0.5% AEP with climate event. The lagoon is assumed to be shallow and susceptible to scour, hence the perimeter will be lined with a riprap revetment to prevent local scour of erodible bed and bank material. Riprap uses layers of natural stone and is the most natural form of grey bank protection. Green bank protection (e.g. willow spiling) is unsuitable for prolonged flow velocities exceeding 1m/s.

### 4.11.2 Lagoon to marine outfall

Between the lagoon and box culvert, the flow will be conveyed in an open channel boulder cascade with vertical sides with a V-shaped cross-profile across the channel bed, to maximise efficiency of the transport of fine sediments within a low flow channel and allow movement of aquatic species during lower flows.

The box culvert beneath the access road will comprise a pre-cast concrete box culvert with vertical concrete headwalls and wingwalls. The culvert barrel will be smooth to allow efficient conveyance of flow. Downstream of the culvert, a natural scour hole will be allowed to develop, providing potential refuge for fish. An embedded cut-off wall at the culvert outlet will protect the box culvert and liquid storage from undermining.

Downstream of the natural scour hole, the open channel section to the marine outfall will comprise a two-stage channel with sloping sides and side berms for ease of access. The lower channel will be a constructed step-pool series, with a step height of 0.2m and pool length of about 8.2m. The steps will be constructed using large, embedded boulders, whilst the pools will be lined with an erosion-resistant clay-gravel-cobble mix. A shallow V invert will provide suitable flow depths for fish passage during low flow conditions. The channel will provide an intertidal habitat environment with potential to act as a refuge for marine species. The introduction of coarse sediment and inclusion of a low flow channel would provide habitat diversity and prevent smothering of the bed with fine sediments.

At the marine outfall, armour stone will be placed to dissipate energy and prevent scour of the foreshore. The large voids between the stones will provide habitat.

# 5. Residual Risks

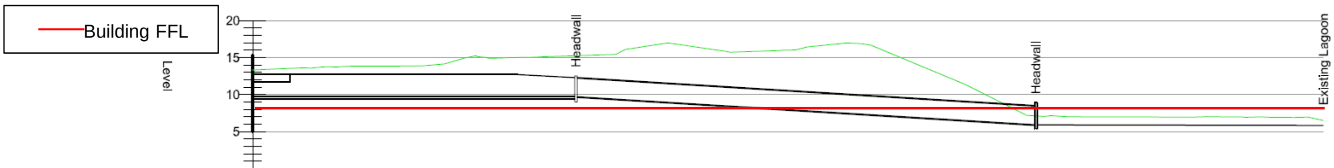
## 5.1 Introduction

Residual risk is the remaining risk left over after inherent risks have been reduced by risk control measures to a practicable level.

## 5.2 Coastal flood risk

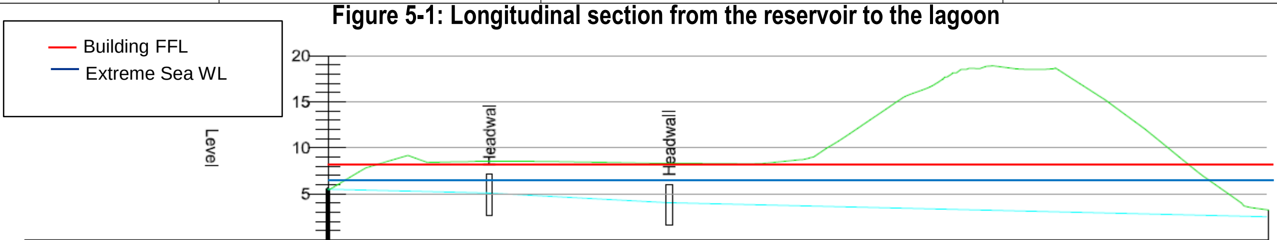
High astronomical tide levels, tidal surges and adverse wind conditions could cause water discharging from the Allt Anavig culvert to back up. This could cause the site to become inundated from the Allt Anavig. The 2095 H++ climate change scenario, which consists of a combination of sea level rise and surge that is beyond the likely range but physically plausible, resulted in sea level rise estimates between 928mm and 2500mm. When added to the extreme still water sea level for this location (4.03m AOD), this gives 6.53m AOD (Table 3.3). The proposed floor level of the buildings is 8.25m AOD, whilst the quarry floor is typically 8.0mAOD (6.0m AOD at the mouth of the site). Climate change impacts on sea levels are not expected to result in a direct risk of coastal flooding, as the finished floor levels have a freeboard of 1.72m and the quarry floor 1.47m.

Longitudinal sections along the watercourse in Figures 5-1 and 5-2 show the finished floor and extreme sea levels. It can be seen that the building would be above extreme sea level.




| Chainage | 00.00 | 10.00 | 20.00 | 30.00 | 40.00 | 44.82 | 50.00 | 60.00 | 70.00 | 80.00 | 90.00 | 100.00 | 109.16 | 110.00 | 120.00 | 130.00 | 140.00 | 149.84 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing Levels (mAOD) | 13.32 | 13.74 | 13.85 | 15.03 | 15.09 | 15.25 | 15.42 | 16.66 | 15.84 | 16.71 | 15.07 | 10.96 | 7.10 | 7.04 | 6.98 | 6.96 | 6.97 | 6.31 |
| Invert Levels (mAOD) | 11.69 11.19 | | | | | 9.711 | | | | | | | 5.925 | | | | | 5.823 |
| Gradient | 1 in 30 | | | | | | 1 in 17 | | | | | | | 1 in 390 | | | | |
| Description | Concrete Spillway | | | | | | 2.5mØ Thrustbore concrete pipe | | | | | | | Open channel | | | | |

**Figure 5-1: Longitudinal section from the reservoir to the lagoon**




| Chainage | 00.00 | 10.00 | 20.00 | 30.00 | 40.00 | 50.00 | 60.00 | 70.00 | 80.00 | 90.00 | 96.203 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing Levels (mAOD) | 5.49 | 8.46 | 8.53 | 8.37 | 8.27 | 9.23 | 15.96 | 18.63 | 14.83 | 6.59 | 3.25 |
| Invert Levels (mAOD) | | | 5.09 | | 4.04 | | | | | | 2.50 |
| Gradient | 1 in 40 | | | 1 in 17.5 | | 1 in 40 | | | | | |
| Description | Open Channel (See Channel Detail below) | | | Culvert (Concrete box culvert 3.6 x 2.4m) | | Open Channel (See Channel Detail below) | | | | | |

**Figure 5-2: Longitudinal section from the lagoon to the marine outfall**

Figure 5-3 shows modelled flood extents for a 0.1% AEP tidal event in conjunction with 10% AEP fluvial flows and shows that the majority of the site would remain flood free.

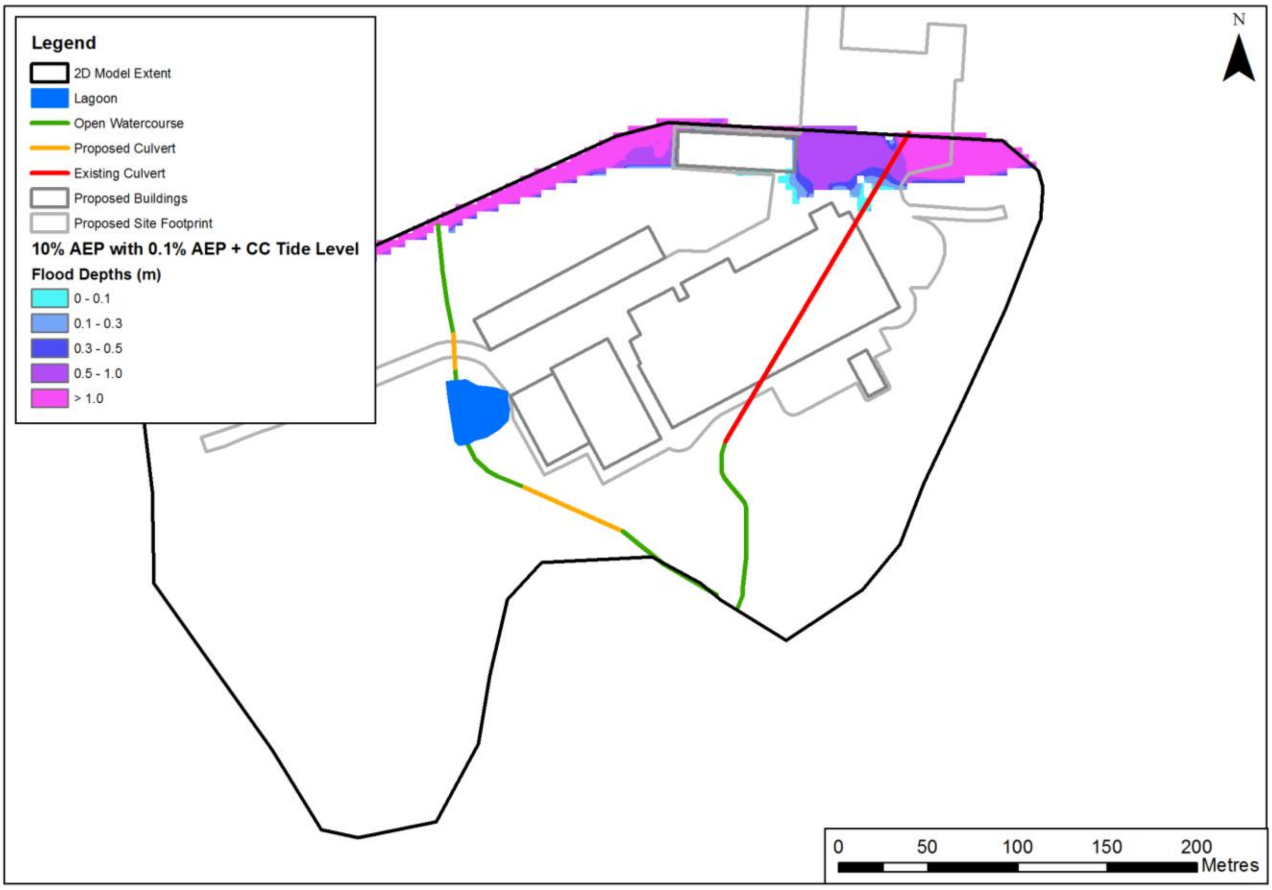



**Figure 5-3: Design Option 0.1% AEP Tidal and 10% AEP Fluvial Flood Extent**

## 5.3 Fluvial flood risk

The proposed design requires the diverted watercourse to be culverted under the site, as at present. The residual flood risk associated with a blockage of both existing and proposed culverts has been modelled.

**Error! Reference source not found.**shows the extent of flooding across the site for the 10% and 50% blockage scenarios in the existing culvert. Similarly Figure 5-5 shows the flood extents for blockages in the proposed 2.5m culvert. Blocking the proposed culvert by 10% alters the flow split from the reservoir, sending more flow down the existing channel. This results in flooding to the external areas of the site but not buildings. At 50% blockage, both external areas of the site and buildings are flooded. It is similar for blockage of the existing culvert: 10% results in some flooding of the external areas of the site and 50% blockage causes flooding of both buildings and external areas. No combined blockage has been tested.


**Figure 5-4: Modelled flood extent for Blockage in the Existing Culvert (0.5% AEP plus CC Flood)**


**Figure 5-5: Modelled flood extent for Blockage in the Proposed 2.5m Culvert (0.5% AEP plus CC Flood)**

Further to the above flood extents, hazard outputs (defined by Defra's Flood Hazard to People classification, FD2321) have been extracted from the TUFLOW modelling results to understand more about the specific issues from the blockage events analysed. Figures 5.6 and 5.7 present the hazard results for the 10% blockage scenario and the 50% blockage scenario, which supplement Figures 13 and 14 of the Hydraulic Modelling Report (Appendix C).

The results presented in Figure 5.6 indicate that with the exception of localised areas around the periphery of the building, but not immediately adjacent to it, the hazards posed by flood water under a 10% blockage scenario are Low, meaning that the combination of shallow depths and the velocity of water are insufficient to pose a significant hazard to the occupiers of the site.

The results of a 50% blockage are presented in Figure 5.7, and these indicate that the hazard to the south side of the building is increased such that there is a Significant hazard by virtue of depth, velocity or a combination of both. Importantly, the hazard in areas to the north of the building remain Low immediately adjacent to the building and key access routes.

In light of the above additional information, the following will be undertaken:

- A site Emergency Flood Risk Plan will specifically include the additional information provided by the Flood Hazard maps presented overleaf so that that the actions and procedures presented take note of areas of likely Significant Hazard and seek to avoid the need for site staff or emergency responders to be in those areas.

- Where appropriate, land levels surrounding the building will be reprofiled to further ensure that areas of greater hazard are located away from the buildings.


**Figure 5-6: Flood hazard (defined by Defra FD2321) for the 0.5% AEP plus climate change scenario with 10% blockage**


**Figure 5-7: Flood hazard (defined by Defra FD2321) for the 0.5% AEP plus climate change scenario with 50% blockage**

## 5.4 Pluvial flood risk

Modern surface water drainage systems are designed to accommodate a design storm (with an allowance for climate change) and minimise the consequences when the design capacity is exceeded. The addition of a climate change uplift factor increases the resilience of the surface water drainage system to anticipated future events (i.e. increase in rainfall intensities). The local land drainage system has not been assessed for this desk study and therefore its design capacity is not known. Any new development will be expected to meet the national design criteria using appropriate uplift factors and exceedance criteria. In addition National Infrastructure will also need to be designed for resilience to inundation.

## 5.5 Flood risk from reservoirs

Construction of the development is not expected to affect the reservoir; however, there remains a residual risk of failure. Although the reservoir has a low risk of flooding the site, there is a high residual risk if the dam is breached or overtopped. The severity of the risk would be increased due to the short response time for site workers in a breach scenario. The management regime of the feature is currently unknown, as is its form of construction, date of construction and under what basis of design it was constructed. As part of the operation of the site, routine inspection and maintenance of the reservoir should be undertaken to ensure that it remains structurally sound, with mitigation implemented if issues are identified.

## 5.6 Rate of inundation

Due to the steep nature of the catchment, if any flooding were to occur via blockage of culverts or failure of the dam, there would be very little warning of flooding and the site would become inundated fairly quickly. Based on the modelled results, inundation of the site is likely to occur in less than two hours.

## 5.7 Access and egress

In the event that a blockage occurs within the existing and proposed culverts, it is evident from Figures 5-2 and 5-3 that there would be no dry access and egress route from the building for a 0.5% AEP CC event. With a 10%

blockage the site can expect flood depths of approximately 62.5cm however, this varies across the site depending on the ground level.

It is recommended that the access track from the building to the north east of the site is raised more in order to provide a safe route for access and egress.

# 6. Planning Policy

## 6.1 Flood Risk Management (Scotland) Act 2009 (FRM Act)

The FRM (Scotland) Act 2009 places a duty on responsible authorities (Scottish Ministers, SEPA, Scottish Water and local authorities) to manage and reduce flood risk, promote sustainable flood risk management. The main elements relevant to the planning system are assessing flood risks and undertaking structural and non-structural flood management measures.

With reference to the proposed scheme, local authorities are required to have a regard to flood risk management plans that are produced under the act. In relation to proposed developments, applicants must assess flood risk in respect of the development. This amends the Town and Country Planning Regulations (Scotland) 2009 so that planning authorities require applicants to provide an assessment of flood risk where a development is likely to result in the material increase in the number of properties at risk of flooding.

## 6.2 Scottish Planning Policy (SPP)

SPP (Scottish Government, 2010) requires planning authorities to consider all sources of flooding (coastal, fluvial, pluvial, groundwater, sewers and blocked culverts) and their associated risks when preparing development plans and reviewing planning applications.

The aims of SPP in relation to flooding are:

- to prevent developments which would be at significant risk of being affected by flooding;
- to prevent developments which would increase the probability of flooding elsewhere; and
- to provide a risk framework from which to identify a site's flood risk category and the related appropriate planning response.

This approach places planning in the wider context of Scottish Government aims and policies, however the SPP does not reinstate policy and guidance used elsewhere but should take into account the wider policy framework including the National Planning Framework in decision making.

# 7. Conclusions and recommendations

## 7.1 Conclusions

Jacobs UK Ltd has been commissioned by Marine Harvest to undertake a Level 1 Flood Risk Assessment (FRA) for a proposed Fish Feed Factory to be constructed within a quarry located in the Allt Anavig catchment.

Flood risks have been considered from a range of sources including coastal, river, pluvial, groundwater, sewers and artificial drainage systems, canals and reservoirs. It has been found that the residual risk of blockages and reservoir failure pose the greatest risk to the site.

The conclusions are as follows:

- Coastal flooding even in its most extreme form does not pose a great risk to site. The ground level across the site and a floor level of 8.25mAOD is seen as sufficient enough in that the site does not become inundated with flood waters.

- Fluvial flooding is considered to be a high risk in its existing state as the current culvert capacity as modelled was found to be 13.9m³/s, which is below the estimated peak flow for the 20% AEP event of 14.2m³/. The proposed reservoir overflow and new channel has been designed to accommodate additional flows and increase capacity to 0.5% AEP plus climate change.

- Pluvial flooding is not seen to pose a risk to the existing site. However there may be temporary changes in surface water flood risk across the area during construction. A surface water drainage system, managing runoff for events up to and including the 0.5% AEP event, is proposed within the site that will capture runoff and direct it to the watercourse in the vicinity of the lagoon in the west of the site.

- Groundwater is not seen to pose a risk to the site and it has been assumed that any groundwater reaching the surface will follow any pluvial flow paths. The construction of a larger impermeable area may provide a small barrier to groundwater however, groundwater flood risk to the development is considered to remain low, particularly as the diversion channel creates a new route to enable groundwater to leave the site without impacting the proposed buildings.

- The management regime of the reservoir, form of construction, date of construction and under what basis of design it was constructed are all currently unknown. The existing risk of reservoir failure (breach or overtopping) is moderate but could be reduced to low through routing inspection and maintenance of the reservoir to ensure that it remains structurally sound, with mitigation implemented if issues are identified.

- Canals and artificial drainage systems were not seen to pose a risk to site.

Residual risk poses the greatest risk to the site with risk of culvert blockages (high risk) and reservoir failure (moderate risk). Modelling has been undertaken for different blockage scenarios (10% and 50%) both of which indicated that the site was at risk of flooding ion both all scenarios however only at a 50% blockage was the building at risk of flooding. Access and egress would be made difficult as a result of the flooding that would take place.

## 7.2 Recommendations

Through the assessment of flood risk across the proposed development site, a list of recommendations have been compiled below in order to facilitate further assessment of flood risk at key points of interest. The list is based on the indicative proposals and hydraulic modelling results provided.

1) Consider using coarse inlet screens to prevent large debris from entering the culvert and reduce the residual risk of blockage. This could be supplemented by debris management within the reservoir and its catchment to reduce debris load.

2) An operational maintenance plan for the culvert, watercourse and reservoirs will be implemented to maintain the low level of flood risk.

3) Floor levels are to be set no lower than 8.25 mAOD in order to maintain a freeboard level of 1.73m above extreme sea level if the 2095 H++ climate change scenario occurred.

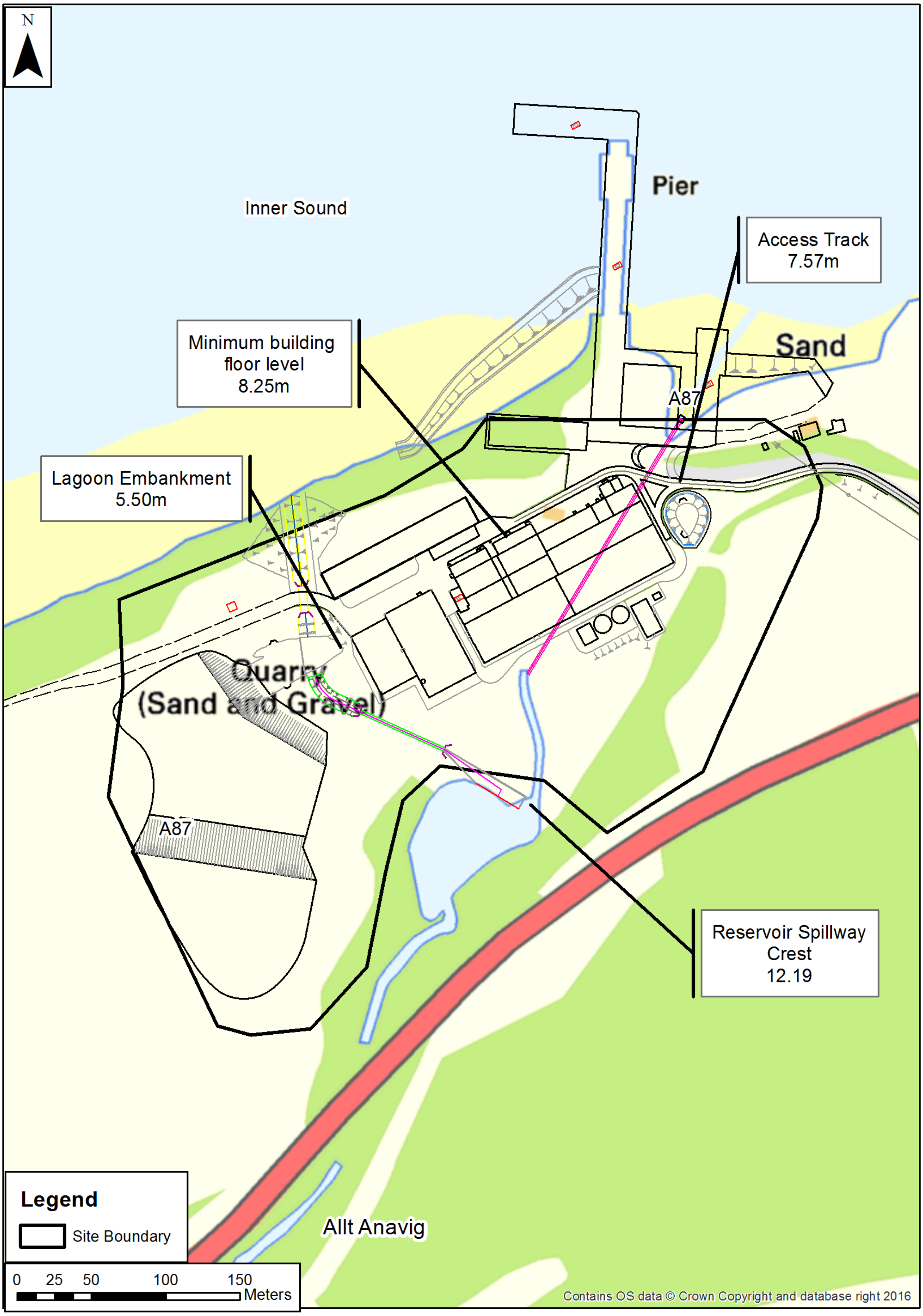
4) Water level monitoring of reservoir and/or audible alarms based on rapid or significant water level change should be implemented due to the short response time occupants of the building will have if the dam were to fail.

5) Emergency procedures to address actions, safe refuge and safe routes for access/egress in event of flooding should be assessed, including presentation of Flood Hazard outputs which should be incorporated into emergency procedures where appropriate.

6) Access from the building and the access track to the east of the site should be raised slightly in order to provide dry access and egress in the event of a blockage occurring within the existing and proposed culverts.

# Appendix A. Site levels Pre- and Post- Development

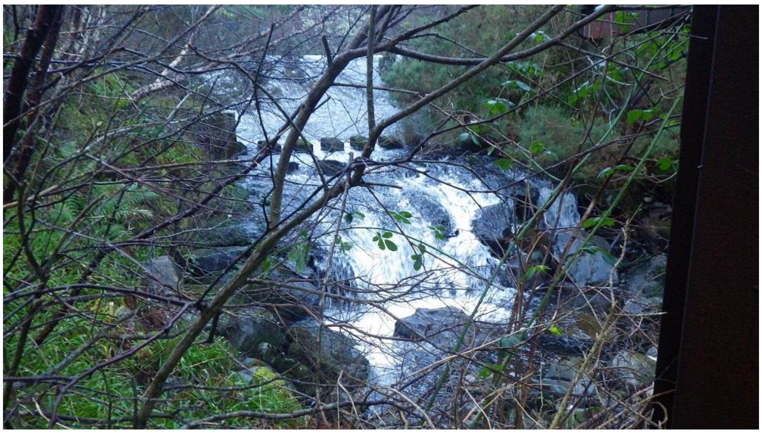

**Pre development levels**




**Post development levels**

# Appendix B. Photographs

| | |
|---|---|
|  | Figure A-1<br>Allt Anavig - general view of reservoir |
|  | Figure A-2<br>Allt Anavig reservoir overflow weir crest |
|  | Figure A-3<br>Allt Anavig reservoir overflow weir crest and spillway |

| | |
|---|---|
|  | Figure A-4<br>Allt Anavig reservoir overflow weir crest and spillway |
|  | Figure A-5<br>Allt Anavig culvert inlet |
|  | Figure A-6<br>General view of quarry and lagoon; the proposed channel will run from the left-hand side of the hill, pass through the lagoon towards the photographer |
|  | Figure A-6<br>General view of quarry and lagoon; proposed channel will cut through the embankment on the left |

# Appendix C. Hydraulic Modelling Report

# Kyleakin Fish Feed Factory

Marine Harvest

## Allt Anavig Hydraulic Modelling Report

B2261401_002 | 02

03 March 2017

### Document history and status

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| Rev 1 | 03/03/2017 | Allt Anavig Hydraulic Modelling Report | Miriam Tailby | Mathieu Valois | Alan Brown |
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# Kyleakin Fish Feed Factory

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Jacobs U.K. Limited

4th Floor, Metro
33 Trafford Road
Salford
M5 3NN
United Kingdom
T +44 (0)161 873 8500
F +44 (0)161 873 7115
www.jacobs.com

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## Contents

# 1. Introduction

Hydraulic modelling is required to support the flood risk assessment for the site of a proposed new fish factory near Kyleakin on the Isle of Skye. The existing Allt Anavig watercourse drains under the A87 road and into a reservoir above the site. This spills into an open channel which is culverted beneath the proposed site to an outlet at the coast.

Figure 1 shows an overview of the site layout, with the proposed site footprint lying at the base of the quarry, adjacent to the coast. Ground levels rise steeply around the site as shown in Figure 2. The Allt Anavig catchment at the inlet of the culvert is approximately 5.5km² with mountainous terrain in the upper catchment and forestry in the lower catchment. It receives a high amount of rainfall.

The purpose of the hydraulic modelling is to assess the flood risk at the site from the existing culvert and with a proposed new additional culvert and channel to the west. This requires a 1-dimensional (1D) model of the Allt Anavig channel to be built using Flood Modeller version 4.2 software and linked to a 2-dimensional (2D) model of its adjacent floodplain using TUFLOW version 2016_03. The 1D and 2D extents of the hydraulic model are shown on Figure 1.

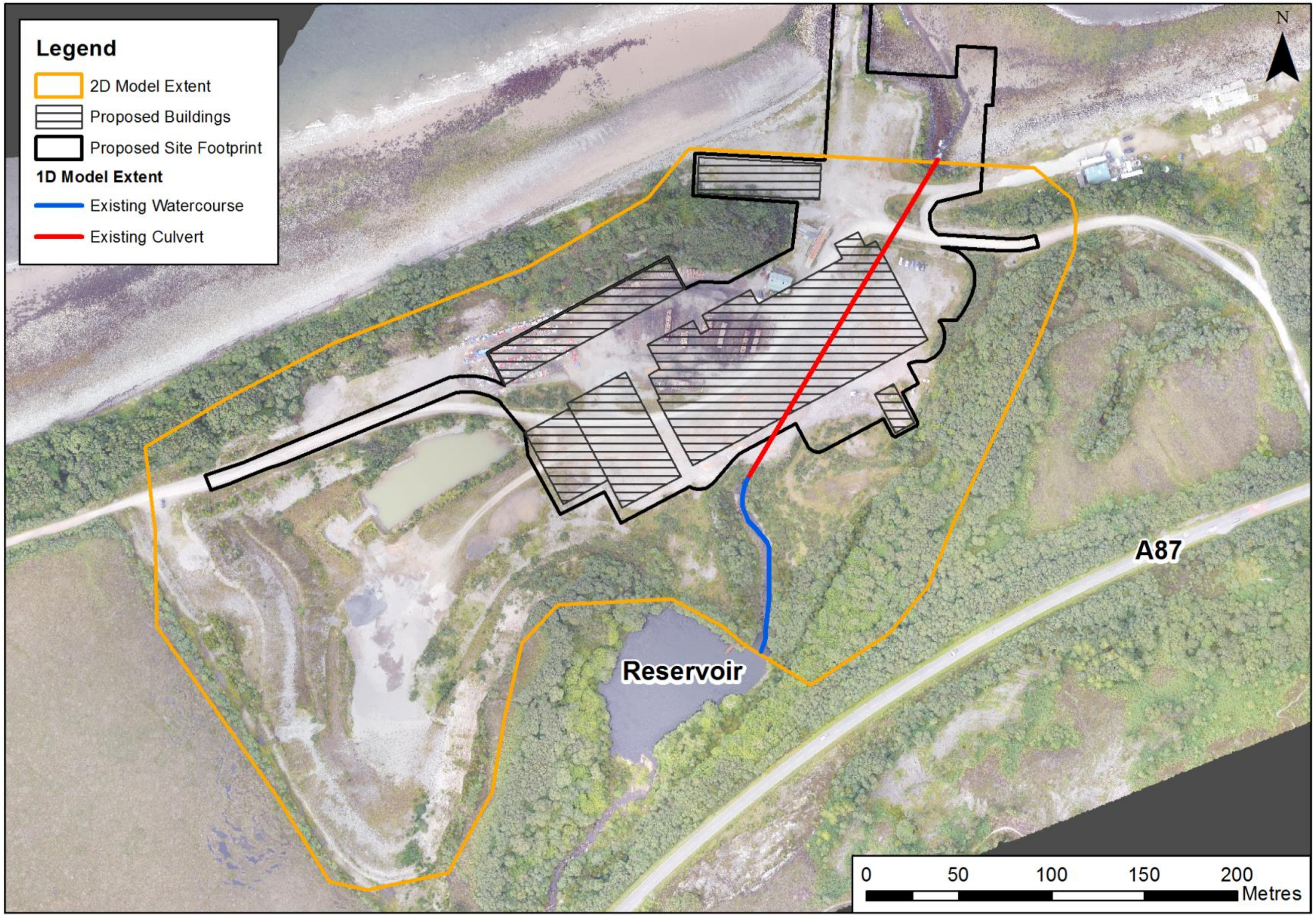


**Figure 1: Site Overview**

# 2. Hydrology

The details of the analysis carried out to produce inflows for the hydraulic model are provided in the hydrology report included in Appendix A.

Peak inflows have been estimated for the 10%, 0.5% and 0.1% Annual Exceedance Probability (AEP) flood events at the upstream end of the existing culvert using the 2013 Revitalised Rainfall Runoff method (ReFH2).

In order to assess the impact of Climate Change (CC), a 20% uplift of the hydrological inflows was applied on the 0.5% AEP event.

Table 1 shows the peak inflow values applied at the upstream end of the model.

**Table 1: Peak Hydrological Inflows**

| Location | Peak Flow ($m^3/s$) | | | |
|---|---|---|---|---|
| | AEP 10% | AEP 0.5% | AEP 0.5% + CC | AEP 0.1% |
| Upstream model extent | 17.1 | 30.1 | 36.1 | 37.5 |

# 3. Baseline Modelling

## 3.1 Input Data

The data used to construct the hydraulic model are summarised in Table 2.

**Table 2: Data Used to Build the Hydraulic Model**

| Data | Description | Source |
|---|---|---|
| LiDAR | Light Detecting And Ranging (LiDAR) digital terrain model at 1m resolution. The dataset was used to inform the hydraulic model with ground elevation information. | Provided by Marine Harvest |
| Survey Drawings | Provided invert levels and diameter for existing culvert. | Provided by Marine Harvest |
| Proposed Design Drawings | Informed watercourse alignment, cross section shape, culvert dimensions, design levels and roughness. | Jacobs |

## 3.2 1D Schematisation

### 3.2.1 In-Channel Geometry

Cross sections were extracted from the LiDAR at 15m intervals along the open channel downstream of the reservoir. Some adjustments were made around the culvert entrance to match the surveyed invert level at the inlet. To aid model performance additional interpolated cross sections were added at 2-4m intervals.

Figure 2 shows the 1D model extent. Table 3 shows the Flood Modeller nodes associated with the modelled watercourse.

The reservoir was included in the 1D model as a reservoir unit using geometry extracted from the LiDAR data. This was connected to the cross section downstream by a spill unit with a weir coefficient of 1.5 and a spill level of 12.19mAOD.

**Table 3: Flood Modeller Nodes**

| Reach | Upstream Node | Downstream Node |
|---|---|---|
| Allt Anavig | ANA01_0309r | ANA01_0010 |

### 3.2.2 In-Channel Hydraulic Friction

Hydraulic roughness (Manning’s ‘n’ coefficient) values were determined primarily from aerial imagery. The channel is steep and rocky but is well vegetated along the banks with trees and shrubs. The Manning’s ‘n’ coefficients used are shown in Table 4.

**Table 4: Manning's 'n' Coefficients - 1D Domain**

| Flood Modeller Nodes | Bed Manning’s ‘n’ | Bed Material | Bank Manning’s ‘n’ | Bank Material |
|---|---|---|---|---|
| ANA01_0309 to ANA01_0210 | 0.05 | Steep, stony channel | 0.10 | Trees and shrubs |

### 3.2.3 In-Channel Hydraulic Structures

The 200m long, 2m diameter culvert under the site was the only hydraulic structure included in the model, as shown in Table 5. Invert and crown levels were taken from survey drawings provided. As no other data was available roughness and inlet characteristics were assumed.

**Table 5: In-Channel Hydraulic Structures**

| Structure | Flood Modeller Node | Model Schematisation |
|---|---|---|
| Culvert under proposed site | ANA01_0210 | Circular conduit + Inlet/Outlet units |

**Table 6: Culvert Inlet Loss Parameters**

| Structure | Unsubmerged Inlet Control Loss Coefficient (K) | Exponent of Flow Intensity for Inlet Control (M) | Submerged Inlet Control Loss Coefficient (c) | Submerged Inlet Control Adjustment Factor (Y) | Outlet Control Loss Coefficient (Ki) |
|---|---|---|---|---|---|
| Culvert under proposed site | 0.0098 | 2 | 0.0398 | 0.67 | 0.5 |

### 3.2.4 Boundary Conditions – 1D Domain

The upstream and downstream boundaries applied to the 1D model are listed in Table 7. A static Mean High Water Spring (MHWS) tidal level was applied to the downstream boundary. This was calculated using Admiralty Tide Tables[1]. Sea level rise was applied to the tidal boundary for the climate change scenario. Levels were increased by 0.2m to account for sea level rise in line with UKCIP09 guidance.

**Table 7: Boundary Conditions - 1D Domain**

| Type of Boundary | Flood Modeller Node | Description |
|---|---|---|
| Flow-Time Boundary | ANA01_0309r | Inflow hydrograph |
| Head-Time Boundary | ANA01_0010 | Mean High Water Spring |

## 3.3 2D Schematisation

### 3.3.1 Flood Plain Topography

Figure 2 shows the location of the 2D model extent. The topography was represented using a 4m grid size with levels based on the LiDAR data provided.

### 3.3.2 Flood Plain Hydraulic Friction

A constant Manning's 'n' roughness value of 0.055 was applied across the 2D domain.

### 3.3.3 Boundary Conditions – 2D Domain

No inflow was applied to the 2D model. Table 8 describes the downstream boundary conditions applied in the 2D domain.

[1] 2006. Admiralty Tide Tables - United Kingdom and Ireland (Including Channel Ports): vol. 1. United Kingdom Hydrographic Office

**Table 8: Boundary Conditions - 2D Domain**

| Type of Boundary | TUFLOW Feature | Description |
|---|---|---|
| Stage-Time | HT Boundary | Mean High Water Spring water level defined along coastline. |

### 3.3.4 1D/2D Linking

The 1D and 2D domains were linked along the length of Allt Anavig and across the top of the culvert, where care was taken to ensure the link was defined at the maximum level of the spill (7.80m AOD).

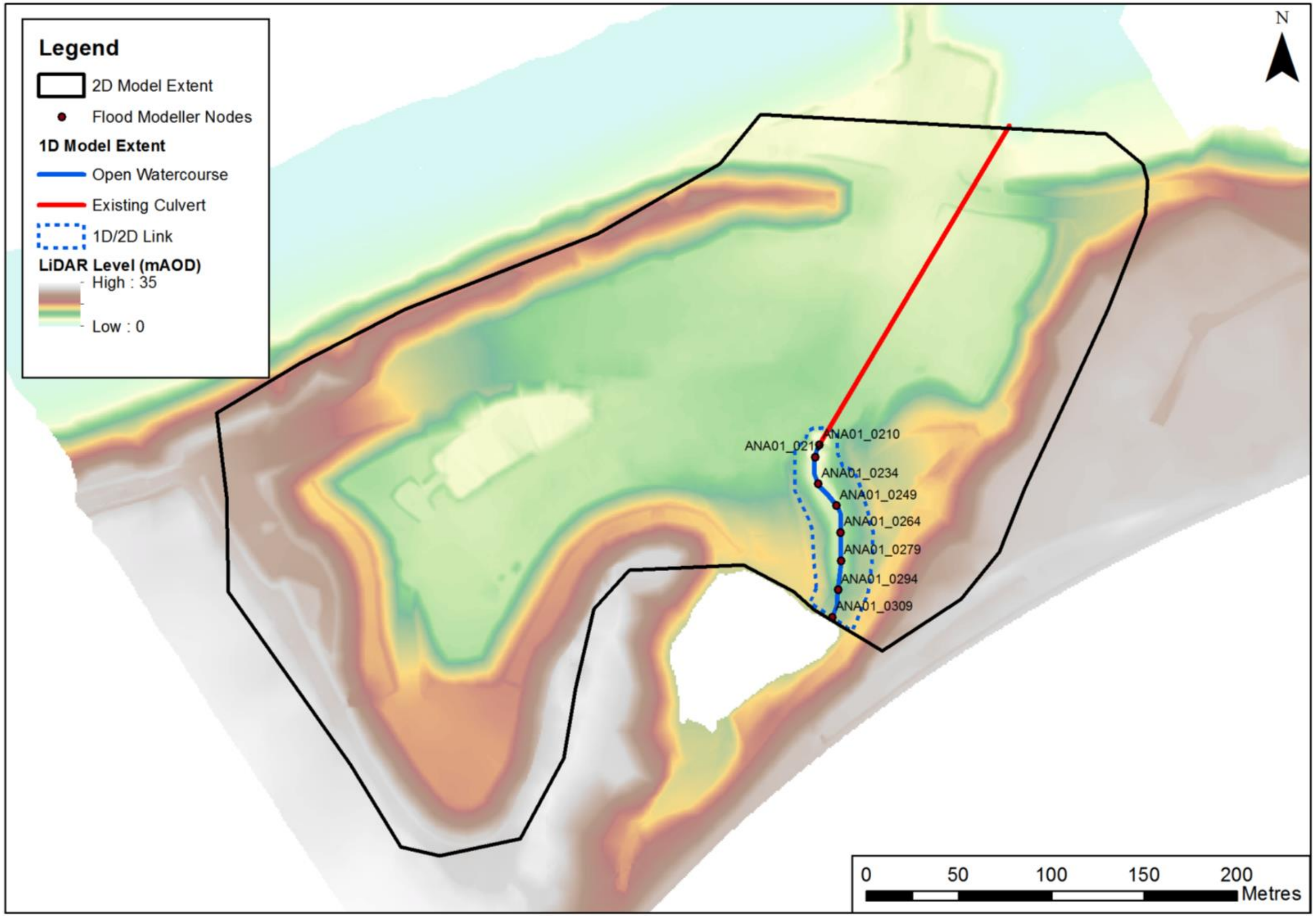


**Figure 2: Baseline Model Extent**

# 4. Design Option Modelling

## 4.1 Design Option

The preferred design option consists of an additional spillway from the reservoir to the west of the existing channel. This new spillway connects to a 2.5 m diameter culvert, then to a section of open channel flowing into an existing lagoon. Another new section of open watercourse would then connect to the lagoon and the sea, with a box culvert under an existing access track. This new flood relief channel has been designed to take a peak flow of 22m³/s in the 0.5% AEP plus Climate Change event and allow only 14m³/s down the existing channel so that the culvert capacity in the existing channel is not exceeded.

Ground levels will be modified over much of the site with the development area having a proposed finished ground level of 8.00m AOD and the majority of the buildings a finished floor level of 8.25 AOD.

Figure 4 shows the proposed design layout.

## 4.2 1D Model Updates

### 4.2.1 Channel Realignment

The new channel has been designed to convey the design flow (0.5% AEP plus 20% Climate Change) so the site is flood free in a design scenario. The baseline 1D model was modified to include a new 33m long spillway at a level of 11.69mAOD, 0.5m below the existing spill level. This connects into a 62m long 2.5m diameter culvert that drains to 34m of open channel with a linear gradient of 2% connecting to the lagoon. Downstream of the lagoon 96m of watercourse connects the lagoon to the sea at a gradient of 3%. Examples of the proposed channel sections for the new open channel are shown in Figure 3. The typical shape has a base width of 4m and side slopes of 1:2.

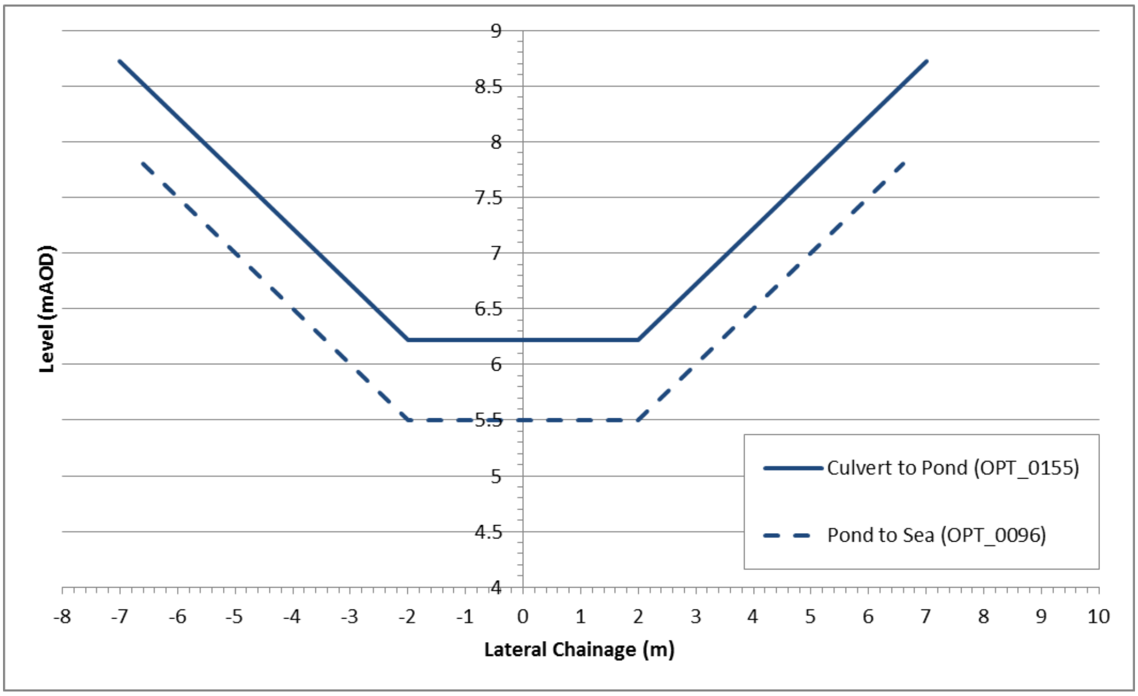


**Figure 3: Proposed Realignment Cross Section**

A Manning's 'n' channel roughness of 0.035 was used for the new open channel upstream of the lagoon. Downstream of the lagoon the proposed channel will be naturalised and a roughness value of 0.05 was used.

#### 4.2.2 Hydraulic Structures

Details of the culverts are shown in Table 9 and Table 10. The inlet of Culvert 1 is assumed to be a standard headwall end of pipe. Under design flow conditions this structure is inlet controlled. The resultant headwater level has been optimised by adjusting the inlet invert levels until the required flow split between the two channels from the upstream reservoir was reached. Culvert 2 is assumed to have an improved inlet structure with 30° flared wingwalls with top edge bevelled 45°. The option model includes a revised head wall for the existing channel culvert with a crest level at 8.30 m AOD to prevent flows spilling over the headwall and onto the site.

**Table 9: Design Structures**

| Structure | Flood Modeller Node | Model Schematisation | Height (m) | Width (m) | Manning's 'n' - Walls and Soffit | Manning's 'n' - Invert |
|---|---|---|---|---|---|---|
| Culvert 1 | OPT_0217 | Circular conduits | 2.5 | NA | 0.013 | 0.035 |
| Culvert 2 | OP2_0080 | Box conduit | 2.4 | 3.6 | 0.013 | 0.05 |

**Table 10: Culvert Inlet Loss Parameters**

| Structure | Unsubmerged Inlet Control Loss Coefficient (K) | Exponent of Flow Intensity for Inlet Control (M) | Submerged Inlet Control Loss Coefficient (c) | Submerged Inlet Control Adjustment Factor (Y) | Outlet Control Loss Coefficient (Ki) |
|---|---|---|---|---|---|
| Culvert 1 | 0.0078 | 2 | 0.0292 | 0.74 | 0.3 |
| Culvert 2 | 0.005 | 1.05 | 0.04 | 0.48 | 0.26 |

#### 4.2.3 Lagoon

A reservoir unit was used to model the lagoon. A level of 5.40mAOD was extracted from the LiDAR for the base elevation and the area was measured using LiDAR and the design drawings. The proposed site level of 8.00 m AOD was used as the top elevation.

### 4.3 2D Model Updates

The 2D component of the baseline model was updated to raise the site footprint area shown in Figure 4 to a minimum level of 8.00 m AOD. The building footprints were lifted to the proposed floor levels, predominantly 8.25mAOD for the buildings in the main site area.

Roughness values were modified to use a Manning's 'n' value of 1 within the building footprints and a value of 0.025 within the rest of the site footprint.

The 1D/2D links were updated for the additional channel and lagoon.

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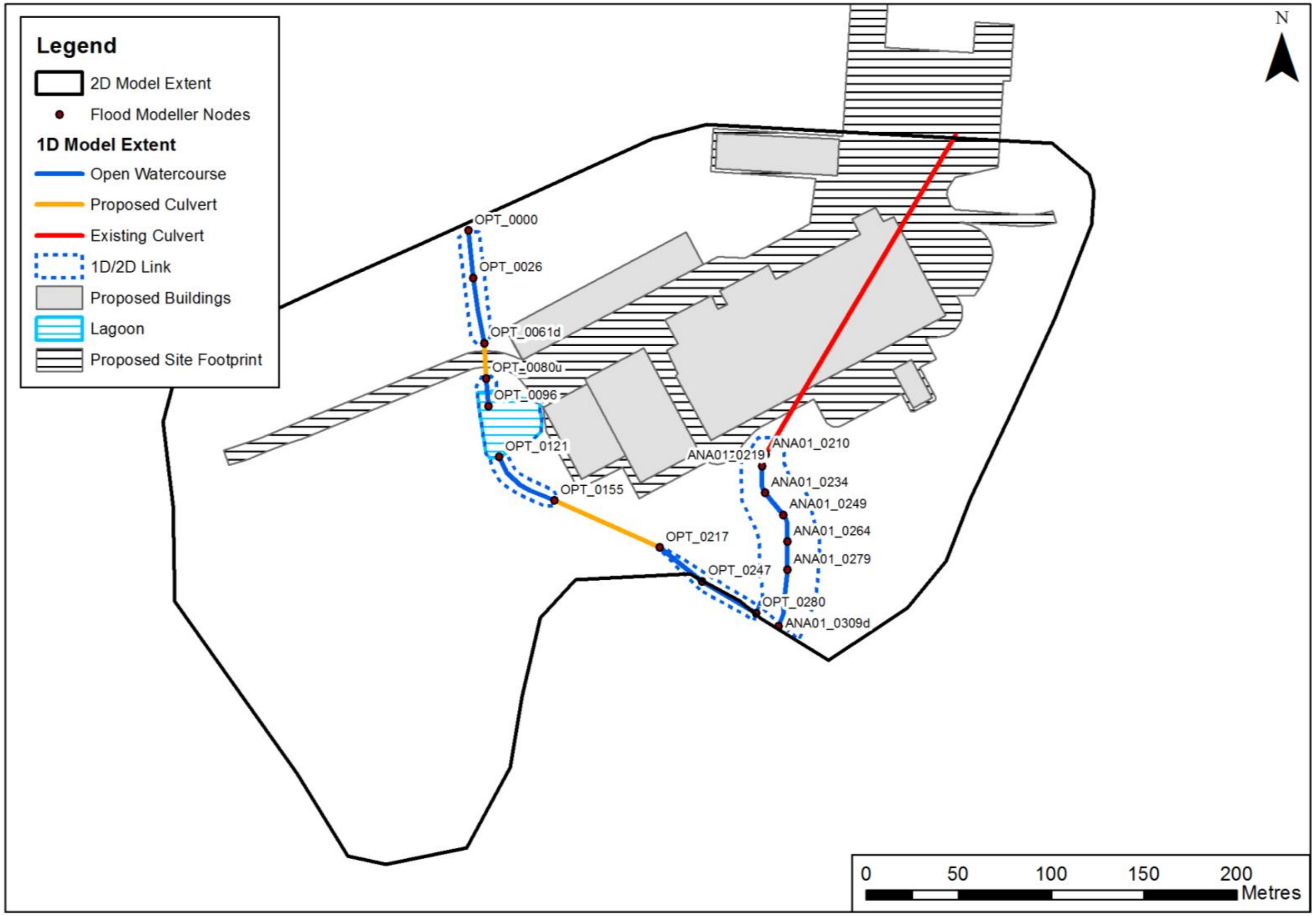


**Figure 4: Design Model Extent and Proposed Design Layout**

# 5. Modelled Events

Table 11 shows the AEP events and model scenarios that have been simulated. Baseline and design option scenarios were run for the 0.5%, 0.5% plus Climate Change and 0.1% AEP events.

Scenarios were then run to check the sensitivity of the model to roughness coefficients and tidal levels as well as to test the effects of culvert blockages.

**Table 11: Modelled Events**

| Scenario | AEP Event | | |
|---|---|---|---|
| | 0.5% | 0.5%+CC | 0.1% |
| Baseline | ✓ | ✓ | ✓ |
| Design Option | ✓ | ✓ | ✓ |
| Design Roughness Sensitivity | | ✓ | |
| Design Tidal Boundary Sensitivity | | ✓ | |
| Design Flow Sensitivity | | ✓ | |
| 10% Culvert Blockage | | ✓ | |
| 50% Culvert Blockage | | ✓ | |

# 6. Model Proving

## 6.1 Model Performance

Run performance was monitored throughout the model build process and during each subsequent simulation to ensure a suitable model convergence was achieved.  Convergence refers to the ability of the modelling software to arrive at a solution that is close to the exact solution within a pre-specified error tolerance. Typical convergence plots for the baseline and design scenarios are shown in Figure 5 below; good convergence was achieved throughout the runs.

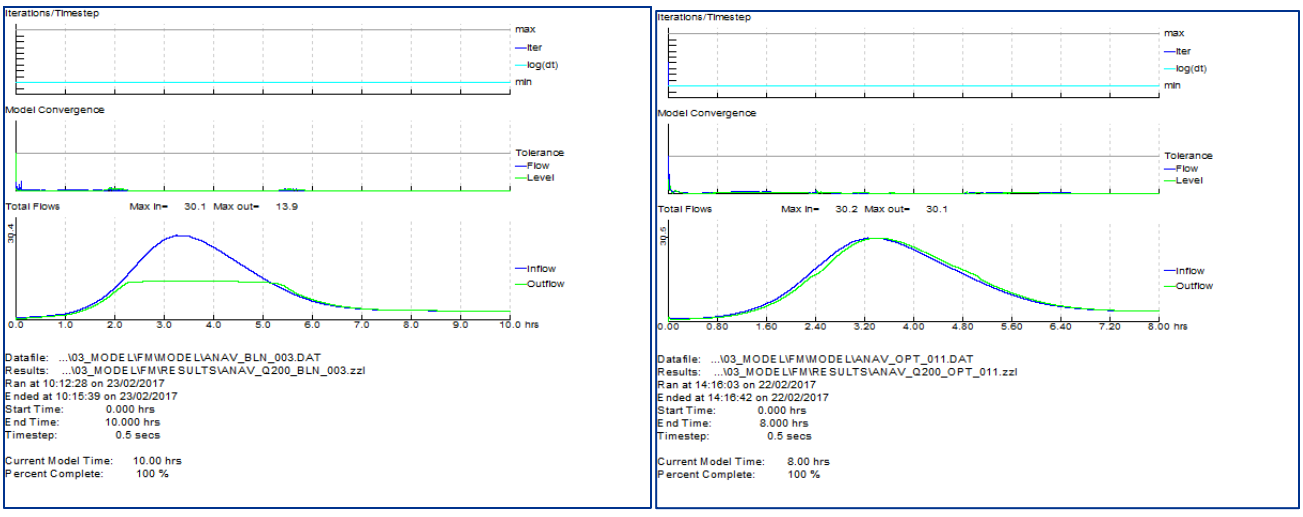


**Figure 5: 1D Model Convergence for the baseline (left) and the design (right) scenarios - 0.5% AEP Event**

The cumulative mass error reports output from the TUFLOW 2D model have been checked.  The recommended tolerance range is +/- 1% Mass Balance error. Figure 6 shows that the mass balance is within tolerance for the baseline 0.5% AEP event except for an initial spike when there is very low volume in the 2D model. This is typical for all the modelled scenarios.

The change in volume through the model simulation was also checked and was found to vary smoothly, which is also an indicator of good convergence of the 2D model.

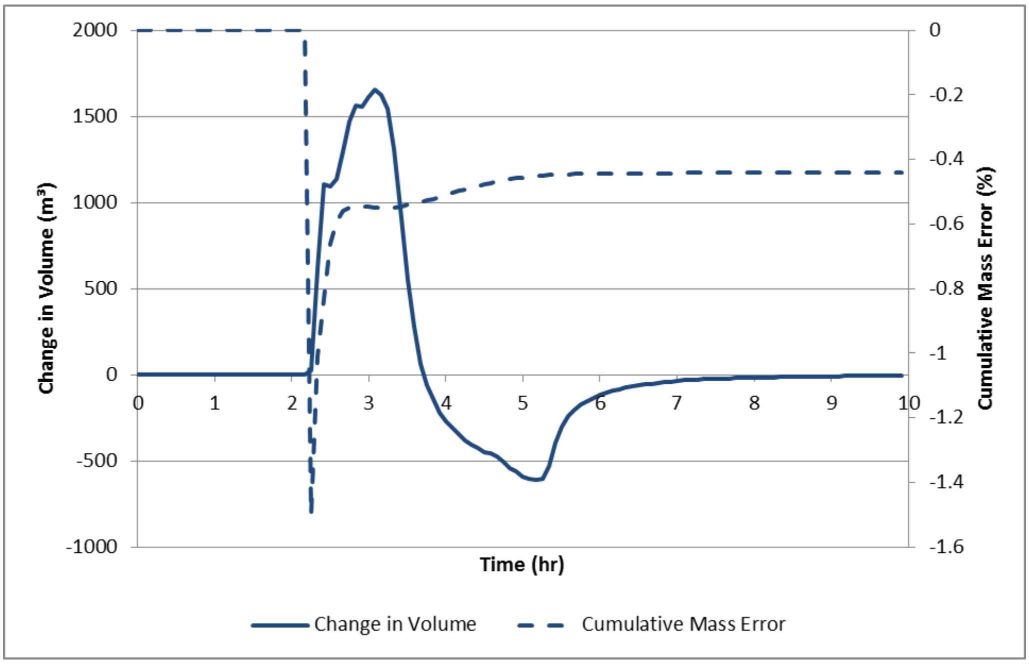


**Figure 6: 2D Cumulative Mass Error and Change in Volume - 0.5% AEP Event**

## 6.2 Calibration

The Allt Anavig watercourse is ungauged. In addition no historic flood information has been found available. Therefore the baseline hydraulic model has not been calibrated.

## 6.3 Sensitivity Analysis

### 6.3.1 Roughness

To test the sensitivity of the model results to uncertainties in the in-channel roughness, Manning's 'n' coefficients were modified by +/-20% in the design option model and the model was run for a 0.5% AEP plus Climate Change event. The resulting differences in water level in the 1D channel are summarised in Table 12. The results show that the in-channel water levels are moderately sensitive to changes in roughness however flow remains within bank and therefore the flood risk to the design is not sensitive to these uncertainties.

**Table 12: Roughness Sensitivity Results**

| Sensitivity Scenario | Maximum Water Level Difference (m) | | Average Water Level Difference (m) | |
|---|---|---|---|---|
| | Existing Channel | New Channel | Existing Channel | New Channel |
| Design + 20% Roughness | 0.120 | 0.112 | 0.064 | 0.025 |
| Design -20% Roughness | -0.180 | -0.129 | -0.090 | -0.024 |

### 6.3.2 Flow

The model sensitivity to flow has been tested using +/-20% variation on the 0.5% AEP plus Climate Change flows. The results (Table 13) show the option design channel itself is not sensitive to increased flow as the flow is controlled by the inlet to the 2.5 m diameter option culvert. However variations in flow do alter the flow split from the upstream reservoir and this is reflected in the discharge in the existing channel. The option design has been optimised for the 0.5% AEP plus Climate Change flow and therefore any variation, particularly an increase is likely to result in flooding of the site. The 20% increased flow scenario showed flooding across the site and above the building floor levels. The design is therefore sensitive to uncertainties in the flow and the site is at risk of flooding as a consequence.

Table 13: Flow Sensitivity Results

| Sensitivity Scenario | Maximum Water Level Difference (m) | | Average Water Level Difference (m) | |
|---|---|---|---|---|
| | Existing Channel | New Channel | Existing Channel | New Channel |
| Design + 20% flow | 0.39 | 0.19 | 0.26 | 0.12 |
| Design -20% flow | -2.73 | -0.18 | -1.03 | -0.08 |

#### 6.3.3 Structure Parameters

The option model has two weir/spillway structures controlling flow out of the upstream reservoir. Details of these structures are uncertain so sensitivity testing of the weir coefficient has been undertaken.

Model results have shown that the new spillway design is not sensitive to the weir coefficient primarily because the weir is drowned by tail water conditions from the culvert inlet under design flows. The weir coefficient for the spill to the existing channel does however impact the flow split between the flood relief channel and the existing channel.

It should be noted that the modelled flow split from the reservoir has been optimised by altering the invert of the new culvert, the correct operation of the flood relief channel is therefore dependent on the culvert invert and inlet design and the subsequent impact this has on upstream water levels.

#### 6.3.4 Tidal Boundary

To test the sensitivity of the option model results to the downstream boundary conditions, the tidal boundary at the downstream end of the model was increased by 1m and the model was run for a 0.5% AEP plus Climate Change event. The results show the design not to be sensitive to variations in the tidal boundary.

#### 6.3.5 Blockage Runs

To test the design to the effects of culvert blockage the design option has been run for six different blockage scenarios. Blockages of 10% and 50% were tested separately on the existing culvert, the new 2.5m diameter culvert and the new 3.6 x 2.4m box culvert. The results of the blockage scenarios show the design to be sensitive to blockage. The results of the blockage scenarios are further discussed in section 7.3.

# 7. Model Results

## 7.1 Baseline

Flood maps showing the maximum flood depths across the site in the baseline scenario are shown in Figure 7 to Figure 9. The existing culvert is shown to have insufficient capacity resulting in flow overtopping at the inlet and flowing across the proposed site. The culvert capacity as modelled was found to be 13.9$m^3$/s. This is below the estimated peak flow for the 20% AEP event of 14.2$m^3$/s.

The resultant flood depths in the 0.5% AEP plus Climate Change event are mostly less than 0.5m across the central area of the site, with depths increasing to greater than 1m in areas to the east and west. A maximum depth of 2.4m is reached in the existing site lagoon. Velocities vary between 0.5 and 1m/s across the majority of the site but exceed 2m/s in localised areas in front of the culvert inlet and in the access way to the coast.

Tabulated water level, flow and velocity results for each node in the watercourse channel are provided in Table 14 to Table 16 in Appendix B.

## 7.2 Design Option

The proposed flood relief channel has been designed to be flood free in a 0.5% AEP plus Climate Change event. However in the 0.1% AEP event the channel overtops at the existing culvert inlet and spills across the site. Flood levels are shallow across the main site area and below building floor levels. The 1D flood extents for the 0.5% AEP plus Climate Change event and 0.1% AEP event are shown in Figure 10 and Figure 11.

A tidal flood scenario using a 0.1% AEP plus sea level rise (Climate Change H++ scenario +2.5 m sea level rise) tidal boundary at 6.53m AOD was run with a coincident 10% AEP fluvial flow. The results from this model scenario (Figure 12) show the site to be predominantly flood free with some tidal inundation around the low-lying coastal areas.

Tabulated water level, flow and velocity results for each node in the watercourse channel are provided in Table 17 to Table 19 in Appendix B.

## 7.3 Blockage Scenarios

As with the existing situation the design requires the diverted watercourse to be culverted. The residual flood risk associated with a blockage of these culverts has been modelled as described in section 6.3.5. Figure 13 shows the extent of flooding across the site for the 10% and 50% blockage scenarios in the existing culvert. Similarly Figure 14 shows the flood extents for blockages in the proposed 2.5m culvert and Figure 15 the 3.6 x 2.4 m box culvert. Blocking the option design 2.5m diameter culvert by 10% alters the flow split from the reservoir sending more flow down the existing channel which results in flooding to the site but not the buildings. At 50% blockage, both site and buildings are flooded. It is the same case for a blockage of the existing culvert, 10% blockage results in some flooding of the site and 50% results in flooding of buildings and site. A 10% blockage of the 3.6 x 2.4 m culvert results in no flooding of the site however a 50% blockage results in some flooding of the site. Given the size of the culvert structures the risk of a significant blockage is considered low and may be mitigated through use of suitable inlet screens. No combined blockage scenario has been tested.

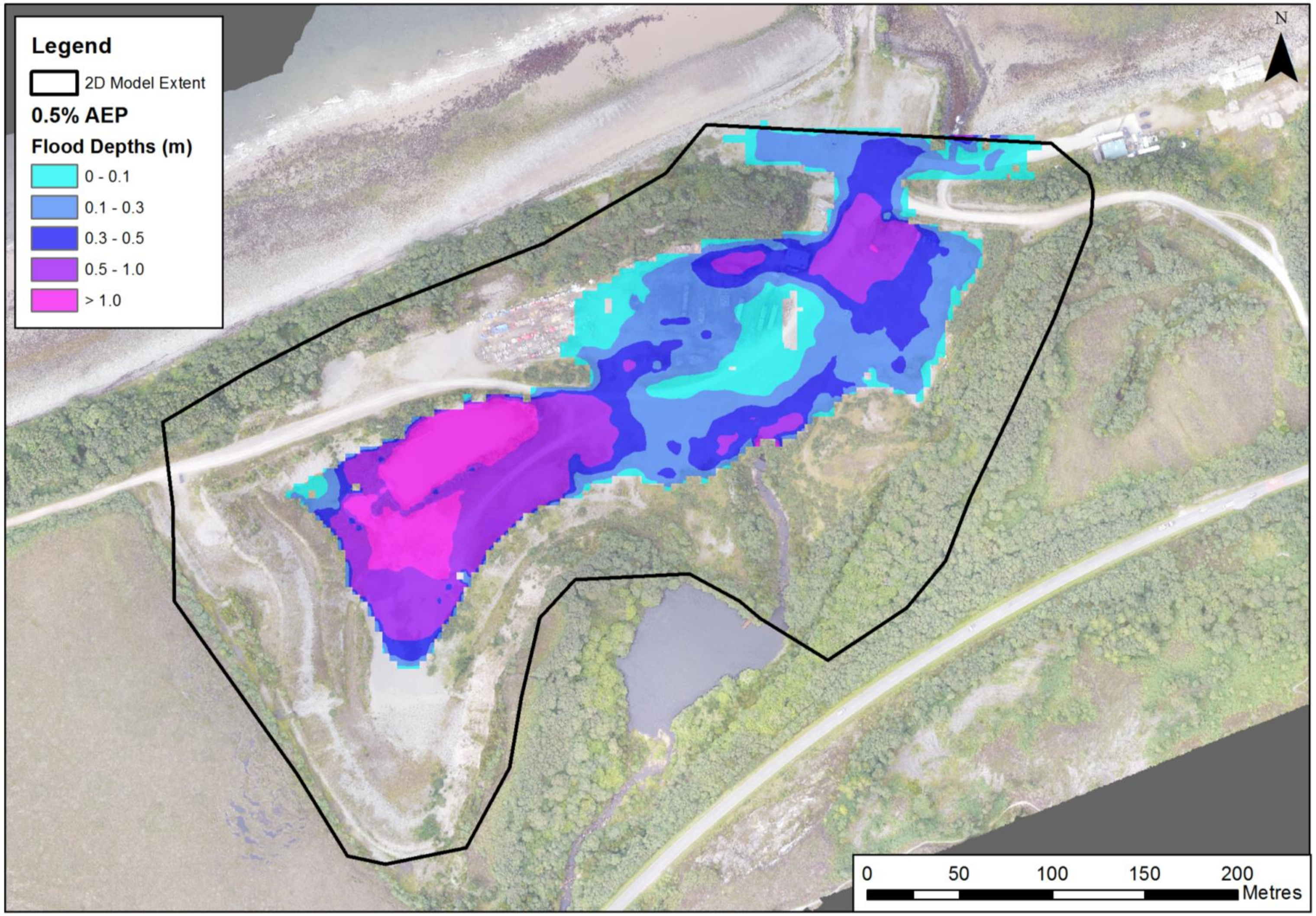


**Figure 7: Baseline 0.5% AEP Maximum Flood Depths**

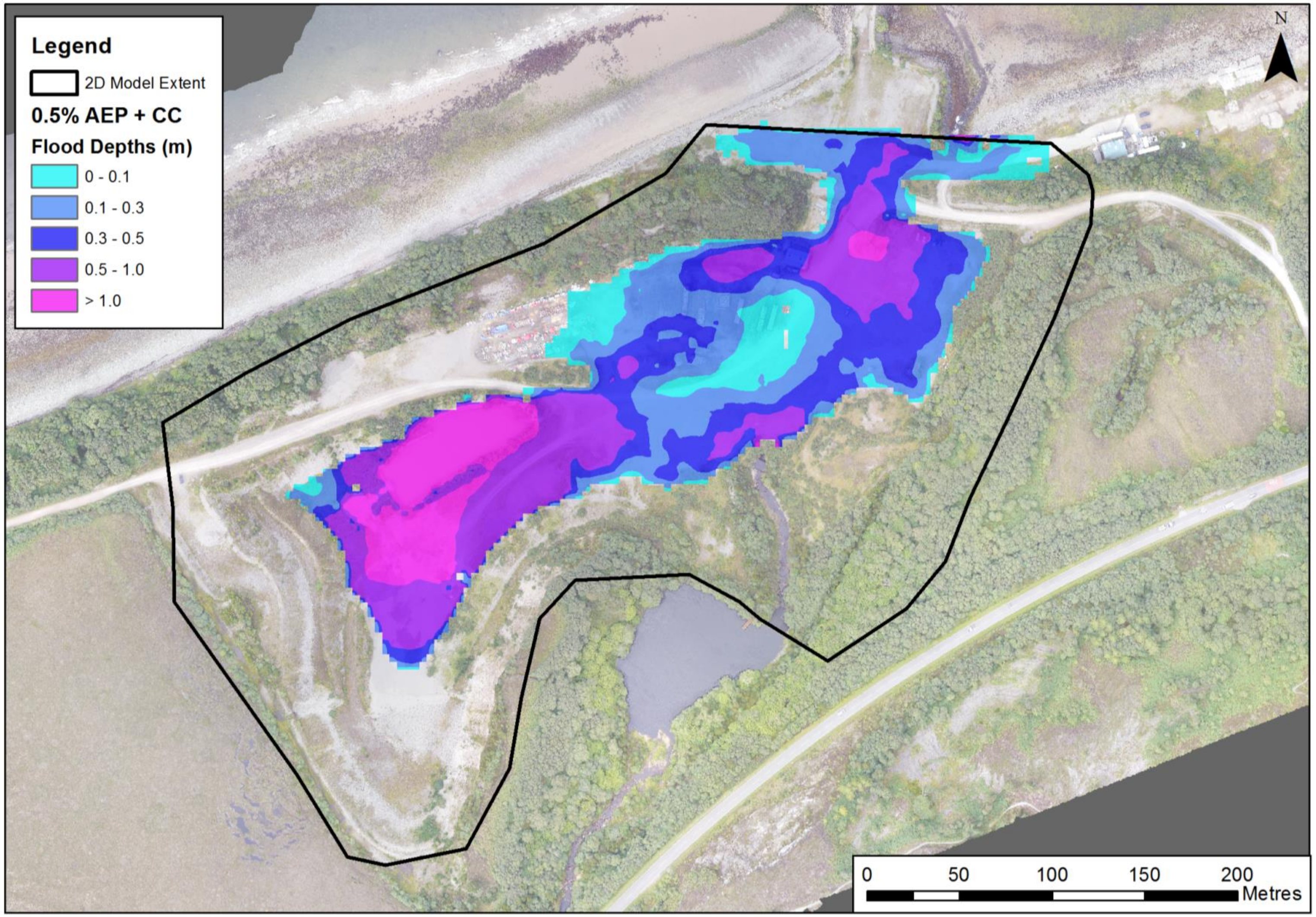


**Figure 8: Baseline 0.5% AEP + Climate Change Maximum Flood Depths**

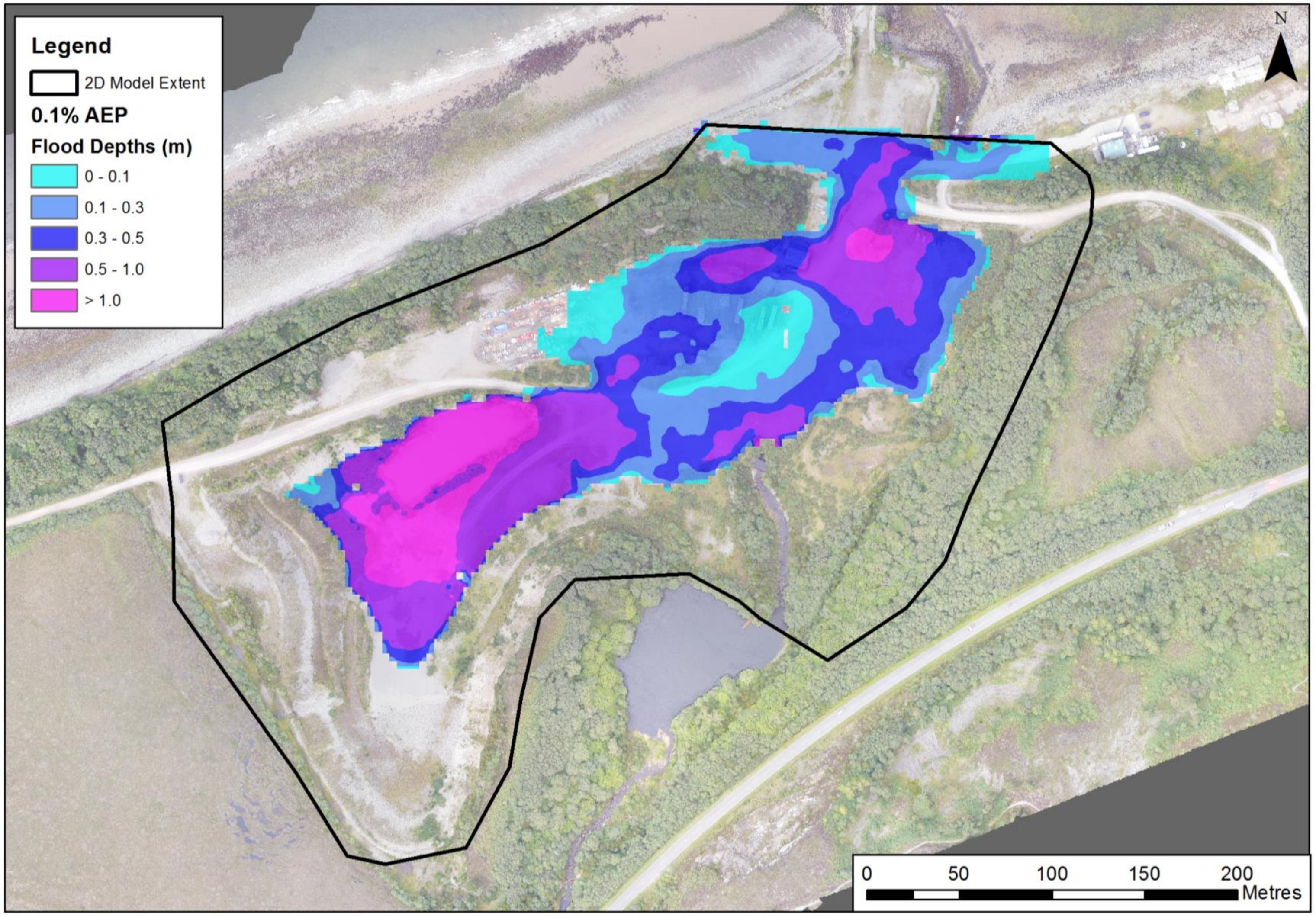


**Figure 9: Baseline 0.1% AEP Maximum Flood Depths**

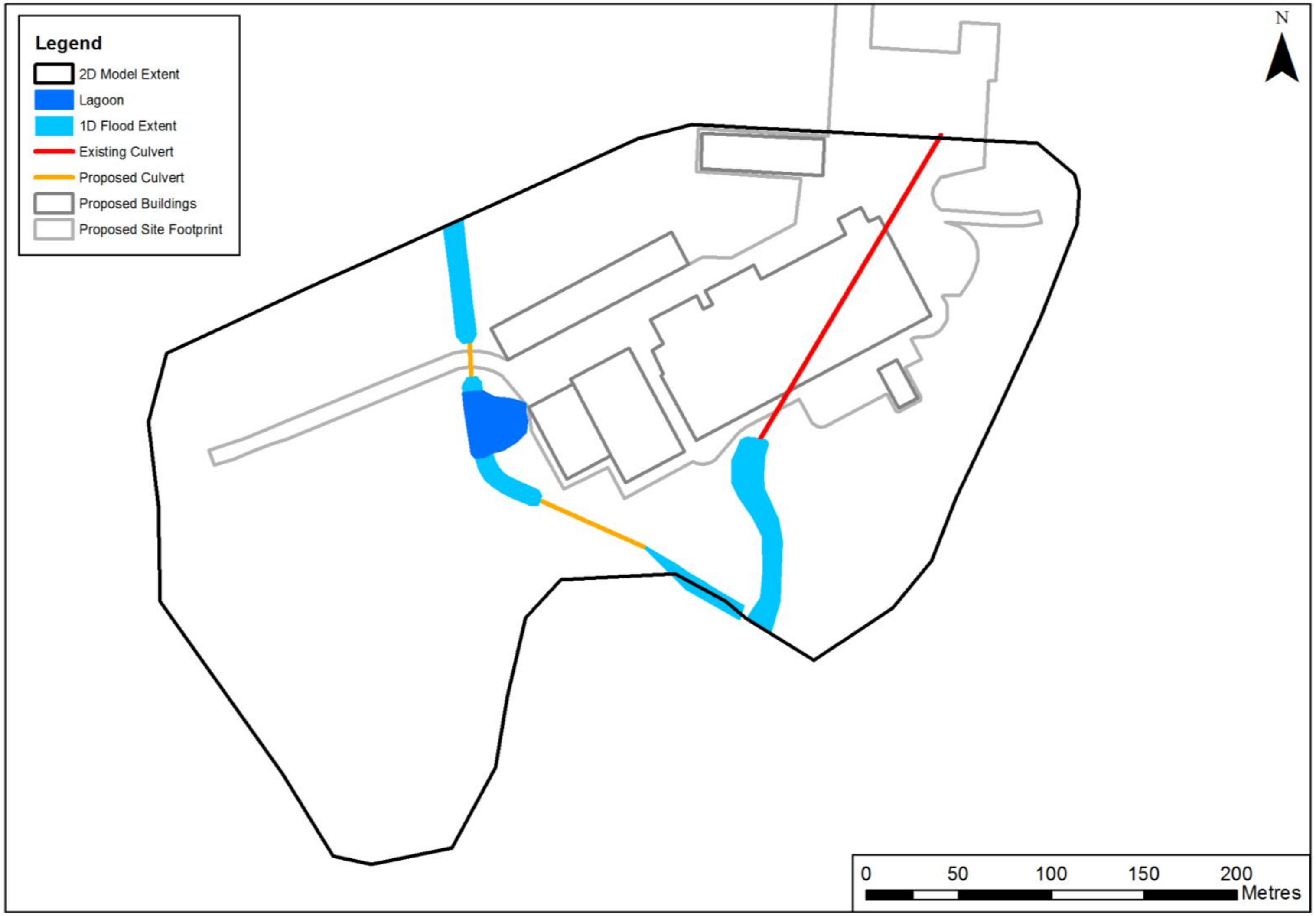


**Figure 10: Design Option 0.5% AEP + Climate Change Fluvial Flood Extent**


Figure 11: Design Option 0.1% AEP Fluvial Flood Extent

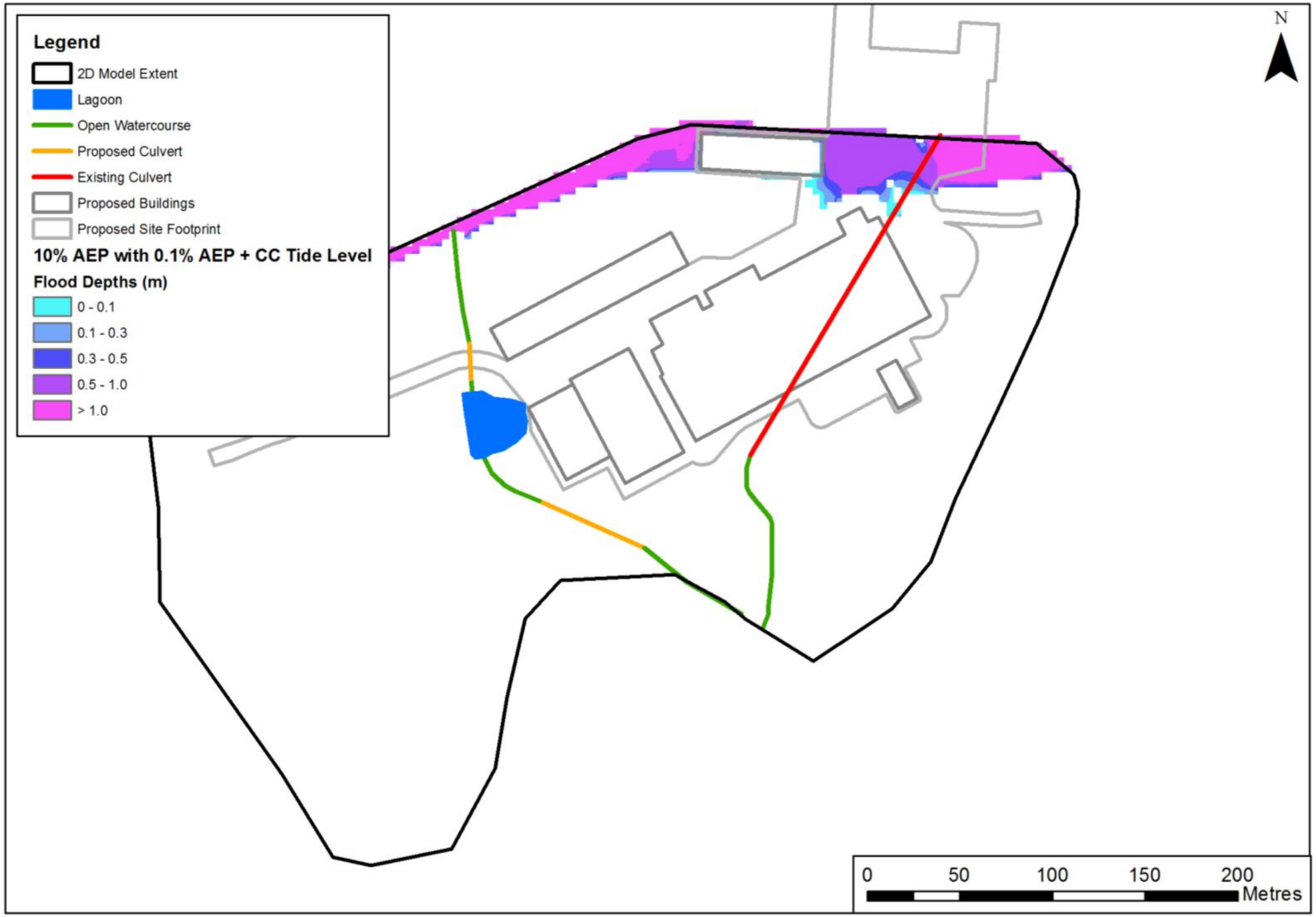


**Figure 12: Design Option 0.1% AEP Tidal and 10% AEP Fluvial Flood Extent**


**Figure 13: Culvert Blockage to the Existing Culvert - 0.5% AEP plus CC Flood Extent**


Figure 14: Culvert Blockage to the Proposed 2.5m Culvert - 0.5% AEP plus CC Flood Extent


**Figure 15: Culvert Blockage to the Proposed 2.5m Culvert - 0.5% AEP plus CC Flood Extent**

# 8. Assumptions and Limitations

The accuracy and validity of the hydraulic model results is heavily dependent on the accuracy of the hydrological and topographic data included in the model. Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process and the assumptions made are considered to be generally conservative for modelled water levels at the proposed site.

The key sources of uncertainty in the model are summarised below:

- Baseline channel geometry and flood plain topography were obtained from 1m LiDAR as no topographic survey data was available.
- The existing channel culvert inlet characteristics were assumed.
- The existing channel roughness was assigned using aerial photography.
- No data was available for model calibration or validation.
- The design is modelled in 1D and therefore the results will only reflect the depth average and node interpolated results. The complex hydraulics that are likely to be present in the culverts and channels may not be accurately represented.

# 9. Conclusions and Recommendations

The hydraulic modelling results for the baseline scenario show that the proposed site is at risk of flooding for a 0.5% AEP flood event and events of larger magnitude due to flows exceeding the capacity of the existing culvert and spilling across the site.

The proposed design option has been shown to be effective at mitigating the flood risk to the site by keeping flows within the designed channel for the 0.5% AEP plus Climate Change event. However some residual risk remains in the case of a culvert blockage and larger flow events than the 0.5% AEP plus Climate Change.

Due to the assumptions and limitations presented in Section 8 and the sensitivity of the design to the structure parameters it is recommended that at detailed design stage:

- Topographic survey is undertaken for the existing channel,
- The design is appropriately tested so the flow split from the reservoir works in practice.

# Appendix A. Hydrology Report

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# Kyleakin Fish Feed Factory

## Allt Anavig Hydrology Report

01 | 01

21 February 2017

# Kyleakin Fish Feed Factory

| | |
|---|---|
| Project No: | B2261401 |
| Document Title: | Allt Anavig Hydrology Report |
| Document No: | 01 |
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Jacobs
7th Floor, 2 Colmore Square
38 Colmore Circus, Queensway
Birmingham, B4 6BN
United Kingdom
T +44 (0)121 237 4000
F +44 (0)121 237 4001
www.jacobs.com

**Document history and status**

| Revision | Date | Description | By | Review | Approved |
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| Rev 0 | 07/10/2016 | Allt Anavig Kyleakin Hydrology Report | Alison Janes | Phil Raynor | A Brown |
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## Contents

# 1. Introduction

## 1.1 Objectives

The objectives for this assessment are for the downstream end of the Allt Anavig to provide:

1) Flood flow hydrographs for the 1 in 2 year, 5 year, 10 year, 25 year, 50 year, 100 year and 200 year return period events (i.e. 50%, 20%, 10%, 4%, 2%, 1%, and 0.5% Annual Exceedance Probability (AEP)).
2) Estimates of the 2nd, 5th, 50th, 80th, 90th 95th and 99th percentile flows.

## 1.2 Catchment Description

The Allt Anavig drains an area of 5.56km$^2$ to the east Isle of Skye. The watercourse runs northwards to join the see to the west of the town of Kyleakin. There are a few small locks in the upper catchment causing attenuation of flows. The catchment receives a high amount of rainfall averaging over 2,000mm per annum and the catchment is considered wet approximately 80% of the time. The percentage runoff is high (over 50%). The Bedrock is Applecross Formation of Sandstone in the most part with Kinloch Formation of Sandstone and Mudstone in the upper catchment. The superficial deposits are Till and Morainic deposits with raised Marine deposits in the lower catchment. Soils are peaty gleys, peaty gleys and peaty gley podzols.

**Table 1.1: Key catchment descriptors, taken from the Flood Estimation Handbook (FEH) and CD-ROM**

| Catchment Descriptor | Allt Anavig | Glossary |
|---|---|---|
| **Catchment Area (km²)** | 5.56 | Catchment area as derived from the FEH CD-ROM IHDTM. |
| **BFIHOST** | 0.303 | This base flow index (BFI) is a measure of catchment responsiveness derived using the 29-class Hydrology Of Soil Types (HOST) classification. The HOST dataset is available as a 1 km grid which records, for each grid square, the percentage associated with each HOST class present. Using IHDTM boundaries for each gauged catchment, the soil characteristics of the catchment can be indexed and by exploiting the relationship between soil typologies and runoff response an aggregated assessment of BFIHOST for the catchment can be derived.<br>The BFIHOST value indicates that approximately 30% of the baseflow is supplied via groundwater. |
| **SPRHOST** | 52.97 | Standard Percentage Runoff (SPR) (%) associated with each Hydrology Of Soil Types (HOST) soil class. The lower the value, the more permeable the catchment.<br>The SPRHOST value for this catchment does not indicate a high level of permeability. |
| **PROPWET** | 0.79 | This is a catchment wetness index (PROPortion of time soils are WET) and provides a measure of the proportion of time that catchment soils are defined as wet (in this context, when soil moisture deficits are less than 6 mm). |
| **DPSBAR (m/km)** | 149.3 | This landform descriptor (mean Drainage Path Slope) provides an index of overall catchment steepness |
| **FARL**<br>**(Flood Attenuation by Reservoirs and Lakes)** | 0.906 | The FARL index provides a guide to the degree of flood attenuation attributable to reservoirs and lakes in the catchment above a gauging station.<br>The FARL score indicates some flood attenuation resulting from the lochs identified in the upper catchment. |

| | | |
|---|---|---|
| **SAAR (61-90) (mm)** | 2166 | Standard Average Annual Rainfall for the period between 1961 and 1990, in millimetres. |
| **$URBEXT_{2010}$** | 0.000 | Extent of urban and suburban land cover in the year 2000 expressed as a fraction. The catchment is entirely rural. |

## 1.3 Flow Estimation Location

Flow estimates are required at the outflow location NG 73850 26400.

# 2. Flood Flows

## 2.1 Methodology

The following bullet points describe the methodology used for the estimation of the flood flows:

- The catchment area for the outflow location was derived from the FEH CD-ROM.
- The Median Annual Maximum Flow (QMED) was calculated from the FEH catchment descriptors.
- A pooling group analysis was undertaken using FEH CD-ROM Version 3.0 (2009) and WINFAP-FEH Version 3.0.003 (2009). The Jacobs WINFAP-FEH database currently uses Peak Flow data version 4.1 dated April 2016, published on the Centre for Hydrology and Ecology (CEH) website.

  WINFAP-FEH allows for pooled analysis to be completed from a group of hydrologically similar catchments to generate flood growth curves. The growth curve estimates were used to establish peak flows for the watercourse.
- For comparison, a Revitalised Flood Hydrograph (ReFH2) analysis was undertaken within the ReFH2 software. The flow results were compared with the results using the FEH statistical method.

## 2.2 Flood Flows – Results

### 2.2.1 QMED Results

Table 2.1 shows the QMED value calculated for the downstream extent of Allt Anavig using the FEH statistical approach.

**Table 2.1: Catchment QMED value**

| Outflow Point | Area ($km^2$) | $QMED_{rural}$ ($m^3/s$) |
|---|---|---|
| NG 73850 26400 | 5.56 | 8.1 |

### 2.2.2 FEH Pooling and ReFH2 Analysis – Design Flows

Table 2.2 shows the growth factors from the WINFAP-FEH pooling group analysis, corresponding FEH peak flows and the ReFH2 peak flows estimated for the downstream location on Allt Anavig using both the FEH rainfall statistics from 1999 and 2013. Details of the pooling group analysis is provided in Appendix A.

**Table 2.2: Growth factors and flow estimates for Allt Anavig**

| Return Period (years) | Growth factors | Flow Estimates ($m^3/s$) FEH Statistical | Flow Estimates ($m^3/s$) ReFH2 (FEH99) | Flow Estimates ($m^3/s$) ReFH2 (FEH13) |
|---|---|---|---|---|
| 2 | 1.000 | 8.1 | 7.0 | 9.9 |
| 5 | 1.331 | 10.8 | 9.3 | 14.2 |
| 10 | 1.561 | 12.6 | 11.2 | 17.1 |
| 50 | 1.882 | 17.4 | 17.5 | 24.0 |
| 100 | 2.149 | 19.7 | 21.3 | 27.0 |
| 200 | 2.443 | 22.4 | 26.0 | 30.1 |

## 2.3 Discussion and Conclusions

The flood flows have been produced using both the FEH statistical and ReFH2 methodologies. Results show the ReFH2 method estimated a much steeper growth than for the statistical method (See Table 2.2). At the lower return periods the statistical method is produces higher flows than the ReFH2 using FEH99 rainfall, but above the 50 year return period the ReFH2 model produces the larger flows. The ReFH2 (2013) method, which utilises more up to date rainfall and slightly different algorithms, results in a higher flow estimate across the board. There is some uncertainty in the results from both methods. In the absence of gauged data within the catchment and a suitable donor station(s) not explored at this stage to use for data transfer, both methods are reliant on the catchment descriptors. In similar situations, SEPA's has preferred to take the larger of the estimates, i.e. ReFH2 (2013) and the same approach is recommended for adoption here in the absence of monitoring information.

# 3. Low Flow Analysis

## 3.1 Gauging Station Overview

There are no gauging stations within the study catchment and no continuous flow data available for Allt Anavig.

**Table 3.1: Key catchment descriptors taken from the FEH CD-ROM**

| Catchment Descriptor | Allt Anavig |
|---|---|
| Catchment Area ($km^2$) | 5.56 |
| BFIHOST | 0.303 |
| SPRHOST | 52.97 |
| PROPWET | 0.79 |
| DPSBAR (m/km) | 149.3 |
| FARL | 0.906 |
| SAAR (61-90) (mm) | 2166 |
| $URBEXT_{2000}$ | 0.00 |

### 3.1.1 Stream Flow Data

At the time of writing this report, there were no continuous or spot gauged flow data available for the Allt Anavig or its tributaries. Given the lack of flow data for Allt Anavig, estimates were made using standard statistical methods and by drawing high level conclusions on the catchment characteristics.

## 3.2 Estimation of Catchment Low Flows

For this study it is important to gain an understanding of the wider hydrological regime and the likely flows within the Allt Anavig, particularly under the lower flow regimes. With the lack of gauged flow data on Allt Anavig an estimate of the flow regime can be made using the catchment characteristics, climatic averages and the Institute of Hydrology Report 108 (IH108) Catchment Water Balance method[1] (Table 3.2). This methodology is applicable to natural catchments and does not consider the impacts from artificial influences, however, in the case of Allt Anavig is considered to be an appropriate technique.

### 3.2.1 Climatic Averages

Average catchment rainfall was exported from the FEH CD-ROM catchment descriptors (See Table 1.1).

Potential Evaporation (PE) was sourced from the Centre for Ecology and Hydrology (CEH) hydrological summary report 2008[2]. This report contained ranges for total PE for each MORECS square along with a corresponding percentage in relation to the 1971 – 2000 average.

The Allt Anavig catchment is on the border of MORECS squares 26 and 27. The Long Term Average was estimated for each square in turn and then the mean of these two values used in the mean daily flow calculations. The results are displayed in Table 3.3.

[1] Gustard, A., Bullock, A. & Dixon, J.M. (1992). Low Flow Estimation in the UK. IH Report No. 108. Institute of Hydrology. Available online at: http://nora.nerc.ac.uk/6050/1/IH_108.pdf
[2] http://nrfa.ceh.ac.uk/hydrological-review-year

**Table 3.2: Estimated average MORECS PE values**

| MORECS Square | Minimum PE (mm) | Maximum PE (mm) | Average PE (mm) | Minimum of LTA (%) | Maximum LTA (%) | Average LTA (%) | LTA (mm) |
|---|---|---|---|---|---|---|---|
| **26** | 450 | 489 | 469.5 | 100 | 109 | 104.5 | 448 |
| **27** | 410 | 450 | 430.0 | 100 | 109 | 104.5 | 410 |
| | | | | | | **Average (mm)** | **429** |

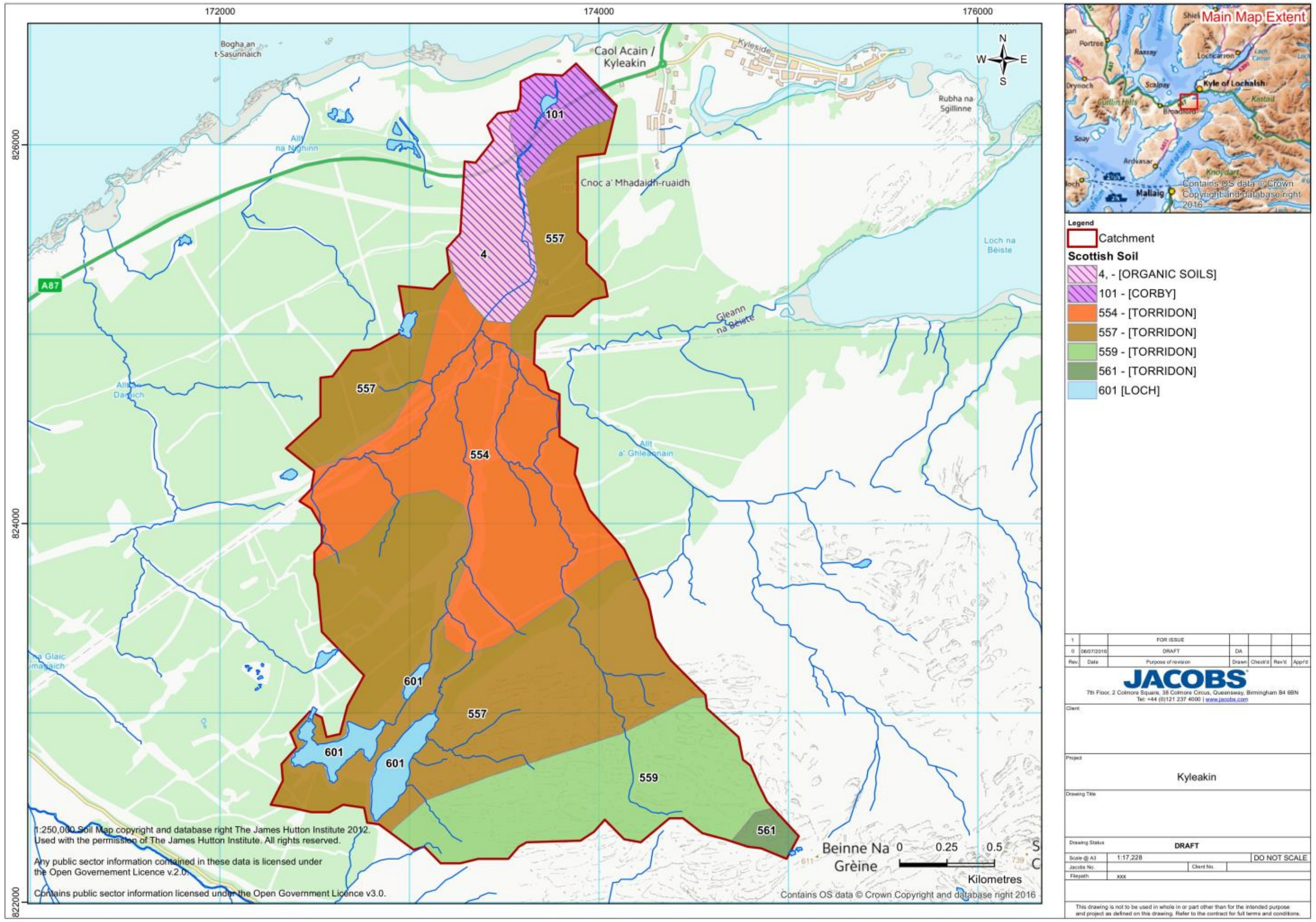



**Figure 3.1: Distribution of soil types within the Allt Anavig catchment**

### 3.2.2 Soils

Catchment soils were identified from the British Geological Survey (BGS) Soil and Drift maps[3] (Figure 3.1: and Table 3.3).

**Table 3.3: Proportions of soils in the Allt Anavig catchment**

| Soil Description | Soil Unit | Area (km²) | Proportion of catchment (%) |
|---|---|---|---|
| Organic Soils | 4 | 0.332 | 6.0% |
| Corby | 101 | 0.185 | 3.3% |
| Torridon | 554 | 1.440 | 26.1% |
| Torridon | 557 | 2.453 | 44.5% |
| Torridon | 559 | 0.892 | 16.2% |
| Torridon | 561 | 0.053 | 1.0% |
| Loch | 601 | 0.161 | 2.9% |
| ** measured from OS base mapping | | | |

### 3.2.3 Estimating Mean Daily Flow

Using the data derived in the above sections, summarised in Table 3.4, and the catchment water balance methodology in the IH report[1] (Equation 3-1) an estimate can be made of the mean daily flow.

$$MDF\ (m^3/s) = AARD \times AREA \times (3.17 \times 10^{-5})$$

**Equation 3-1: Equation to estimate the Mean daily flow**

**Table 3.4: Estimate of mean daily flow in Allt Anavig using IH108 method**

| Parameter | Allt Anavig |
|---|---|
| Catchment Area (km²) | 5.56 |
| Rainfall (SAAR 1961-1990) (mm) | 2,166 |
| Evaporation (MORECS 1971-2000) (mm) | 429 |
| Average Annual Runoff Depth (AARD) (mm) | 1737 |
| Mean Daily Flow (m³/s) | 0.306 |

### 3.2.4 Allt Anavig Flow Duration Curve

The IH108 method allows for the mean daily flow to be proportioned based on soil types to estimate the $Q_{95}$[4] flow. The soil types within the catchment were extracted from BGS soil maps and using IH108 the $Q_{95}$ proportions were calculated. The $Q_{95}$ for the catchment was estimated to be approximately 13% of the mean daily flow; this gave a $Q_{95}$ flow of 0.041 m³/s (3.54 Ml/day).

[3] BGS Soil and Drift map: Lawes Agricultural Trust (Soil Survey of England and Wales) 1983.
[4] A flow that is equalled or exceeded 95% of the time

Once the relationship between the mean daily flow and $Q_{95}$ flow has been established, additional percentile flows can be estimated based on a standardised curve. The IH108 guidance recommends simple interpolation between the reported factors. For the Allt Anavig the percentage of mean daily flow calculated suggested that the appropriate flow duration curve for the watercourse is between curves 11 and 12 and an interpolation was made between these. The resultant percentile flows are provided in Table 3.6.

**Table 3.5: Allt Anavig flow percentiles**

| Flow Percentile | Percentage of MDF | Flow ($m^3/s$) |
|---|---|---|
| $Q_2$ | 382.2 | 1.170 |
| $Q_5$ | 278.9 | 0.854 |
| $Q_{50}$ | 57.5 | 0.176 |
| $Q_{80}$ | 26.1 | 0.080 |
| $Q_{90}$ | 17.6 | 0.054 |
| $Q_{95}$ | 13.2 | 0.041 |
| $Q_{99}$ | 8.2 | 0.025 |
| **Mean Daily Flow** | | 0.306 |

# Appendix A. Audit Trail – Flood Flows Pooling Analysis

# Jacobs flood study audit trail
# FEH pooling group analysis

Allt Anavig @ Kyle Akin

Jacobs UK, 7th Floor, 2 Colmore Square, 38 Colmore Circus, Queensway,
Birmingham, B4 6BN
Tel  0121 237 4000   Fax  0121 237 4001

# Project details

**Project title:** Kyleakin
**Project number:**
**Work Stage:** Flood Risk Assessment
**Client:** Marine Harvest
**Flood study site:** Allt Anavig @ Kyle Akin

# Contents

# Jacobs flood study audit trail

FEH pooling group analysis

## 1 General

The following analysis was undertaken using the FEH CD-ROM Version 3.0 (2009) and Winfap-FEH Version 3.0.003 (2009).

The Jacobs Winfap-FEH database uses the HiFlows-UK database v4.1 dated May 2016, published on the CEH website.

## 2 Catchment description

Grid Reference of the outflow: NG 73850 26400

### 2.1 FEH catchment descriptors:

| | |
|---|---|
| AREA | 5.56 |
| ALTBAR | 195 |
| ASPBAR | 344 |
| ASPVAR | 0.55 |
| BFIHOST | 0.303 |
| DPLBAR | 3.31 |
| DPSBAR | 149.3 |
| FARL | 0.906 |
| FPEXT | 0.0274 |
| FPDBAR | 0.389 |
| FPLOC | 0.826 |
| LDP | 5.74 |
| PROPWET | 0.79 |
| RMED-1H | 11.9 |
| RMED-1D | 54.8 |
| RMED-2D | 77.5 |
| SAAR | 2166 |
| SAAR4170 | 1876 |
| SPRHOST | 52.97 |
| URBCONC2000 | -999999 |
| URBEXT2000 | 0.0000000 |
| URBLOC2000 | -999999 |

**2.2 Presence of significant land-use or catchment factors:**

| Factors | Comment | Potential Significance |
|---|---|---|
| Reservoir\lake | Small upland lochans are situated in the catchment including Lochan na Saile.<br>FARL = 0.906 | Attenuation |
| Urban | Catchment is essentially rural URBEXT 2016 =0.000 | Nothing that would significantly alter flood response |
| Land use | Forested in the lower catchment and moorlands in the upper catchment | NIL |
| Soils\Geology | Bedrock is Applecross Formation of Sandstone in the most part with Linloch Formation of Sandstone and Mudstone in the upper catchment. The superficial deposits are Till and Morainic deposits with raised Marine deposits in the lower catchment.<br>Soils are peaty gleys and peaty gleys and peaty gley podzols.<br>BFIHOST = 0.303, SPRHOST = 52.97. | Nothing that would significantly alter flood response |

**2.3 Flow record:**

Target site: Ungauged

## 3 Estimation of QMED

### 3.1 Approach used

| Used | Condition | Approach followed |
|---|---|---|
| | N >=30 | Estimate QMED using annual maxima |
| | 14=< N =<29 | Estimate QMED from annual maxima & optionally adjust for climatic variation |
| | 2=< N= <13 | Estimate QMED from POT data & adjust for climatic variation |
| | N <2 & suitable donor site with 20 years or more of record | Ignore record at subject site; transfer QMED from donor site |
| | N <2 & suitable donor with 10 to 19 years of record & 12 month overlap between records | Estimate QMED using procedure based on flood peak regression |
| | N <2 & suitable donor with 10 to 19 years of record but no 12 month overlap | Ignore record at subject site; transfer QMED from donor site |
| | N <2 & no long-record site nearby | Estimate QMED from very short POT record |
| | N <2 & no long-record site nearby | Treat site as ungauged catchment |
| | N <2 & no long-record site nearby | Defer analysis until longer flow record available |
| | N <2 & no long-record site nearby | (Abstract flood event information and apply the UH rainfall-runoff model as an alternative, to the pooling group procedure. Particularly recommended when site is urbanised) |
| ✓ | Ungauged catchment | Estimate QMED from catchment descriptors |
| | Ungauged catchment | Estimate QMED by data transfer from donor catchment |
| | Ungauged catchment | Estimate QMED by data transfer from analogue catchment |
| | Ungauged catchment | Estimate QMED from channel dimensions |
| | Ungauged catchment | Compare to regional pattern of mapped QMED adjustment factors |

(*preferred method)

### 3.2 QMED estimation from catchment descriptors

| Attribute | Value |
|---|---|
| AREA | 5.56 |
| SAAR | 2166 |
| FARL | 0.906 |
| SPRHOST | 52.97 |
| BFIHOST | 0.303 |
| URBEXT2016 | 0.000 |

$QMED_{\text{Catchment descriptors – rural}} = 8.3062\ \mathbf{AREA}^{0.8510}\ 0.1536^{(1000/\mathbf{SAAR})}\ \mathbf{FARL}^{3.4451}\ 0.0460^{\mathbf{BFIHOST}\text{^}2}$
$= 8.3062*5.56^{0.8510} * 0.1536^{(1000/2166)} * 0.906.^{3.4451} * 0.046^{0.303\text{^}2}$
$\mathbf{= 8.08.m^3/s}$

68% confidence interval = (5.65,11.56)
95% confidence interval = (3.95,16.54)

### 3.3 QMED estimation by data transfer

Nearby stations were checked for suitability of a donor catchment. Station 93001 is to the north east of the study catchment. It is deemed suitable for QMED calculation and pooling. However, the catchment area is much larger at 139km$^2$ and there is more attenuation of flows with the FARL value lower at 0.858.

On the Isle of Syke there is a station 105001. This station does not have any peak flow data and therefore could not be used to inform a data transfer.

No urban adjustment was made to the estimation of QMED as the catchment is rural with an URBEXT value of 0.000.

## Jacobs flood study audit trail

FEH pooling group analysis

**4 Steps involved in construction and analysis of a pooling group.**

**4.1 Pooling group construction**

Site of interest

| | | | |
|---|---|---|---|
| (a) Station Number | N/A | (b) Name | Kyleakin |
| Name of saved **.feh** group file | Kyleakin.feh | | |
| Target return period (years) for 5T rule | 100 | | |

**4.2 Initial Pooling group details**

Total number of sites: 16 — Total number of years: 523

| | |
|---|---|
| Total number of initial high discordancy sites | 1 |
| List them: 49006 | |
| Total number of short records (< 7 years) removed | 0 |
| List them: N/A | |
| Number of pooled years after sites removed | 523 |

**4.3 Subject Site Details**

Is subject site included as Rank 1 in pooled group: ~~yes~~ no

If no state reason why: Site is ungauged

**4.4 Test statistics on validity of pooling group for flood frequency analysis**

Heterogeneity test H2 value = 2.19

| Status | | |
|---|---|---|
| Review not necessary | | H2 < 1 |
| Review optional | | 1 < H2 < 2 |
| Review desirable | ✓ | 2 < H2 < 4 |
| Review essential | | H2 > 4 |

Goodness-of-fit test — Z values

| | | Value |
|---|---|---|
| GL | acceptable / ~~not acceptable~~ | 0.96 |
| GEV | acceptable / ~~not acceptable~~ | -0.82 |
| PT3 | ~~acceptable~~ / not acceptable | -1.41 |
| other | | |

(Note: in the FEH the GL is the generally favoured distribution for use)

ACTION is construction of flood frequency curve valid?

No ~~Yes~~

Check the suitability of sites in the pooling group

## 4.5 Revision of Pooling Group

**Revision No.** 1

| Station Number | Reason for changes in pooling group |
|---|---|
| 54022, 25003, 28033, 45816, 25011, 49006, 27051, 46005, 25012 | FARL = 1.0 |
| 48004, 73009, 72014, 48001, 73015, 76811, 89004, 47021, 51003 | Added to increase station years to 500 |
| | |

Number of sites: 16  Years: 523

Heterogeneity test  H2 value = 3.12

| Status | | | |
|---|---|---|---|
| | Review not necessary | | H2 < 1 |
| | Review optional | | 1 < H2 < 2 |
| | Review desirable | ✓ | 2 < H2 < 4 |
| | Review essential | | H2 > 4 |

Note: FEH Vol.3, chapter 16.3.2: "The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review."

| Goodness-of-fit test | Z values | | | Value |
|---|---|---|---|---|
| | | GL | acceptable / ~~not acceptable~~ | 0.55 |
| | | GEV | acceptable / ~~not acceptable~~ | -1.55 |
| | | PT3 | ~~acceptable~~ / not acceptable | -1.77 |
| | other | | | |

ACTION is construction of flood frequency curve valid?

~~No~~ Yes

Comment?

## 4.6 Flood frequency analysis of pooling group

| | | | | |
|---|---|---|---|---|
| Distributions selected | GL | ✓ | PT3 | |
| | GEV | ✓ | other | |

Standardisation method selected — Median (this acts as a check as median is the only method allowed within the pooling group method) ~~Mean~~

Construct flood frequency curve

| | | | If yes | | | |
|---|---|---|---|---|---|---|
| URBEXT updated | ~~yes~~ | no | from | | to | |
| Urban adjustment* | ~~yes~~ | no | | | | |
| Value of QMED = | 8.08 | $m^3/s$ | | | | |

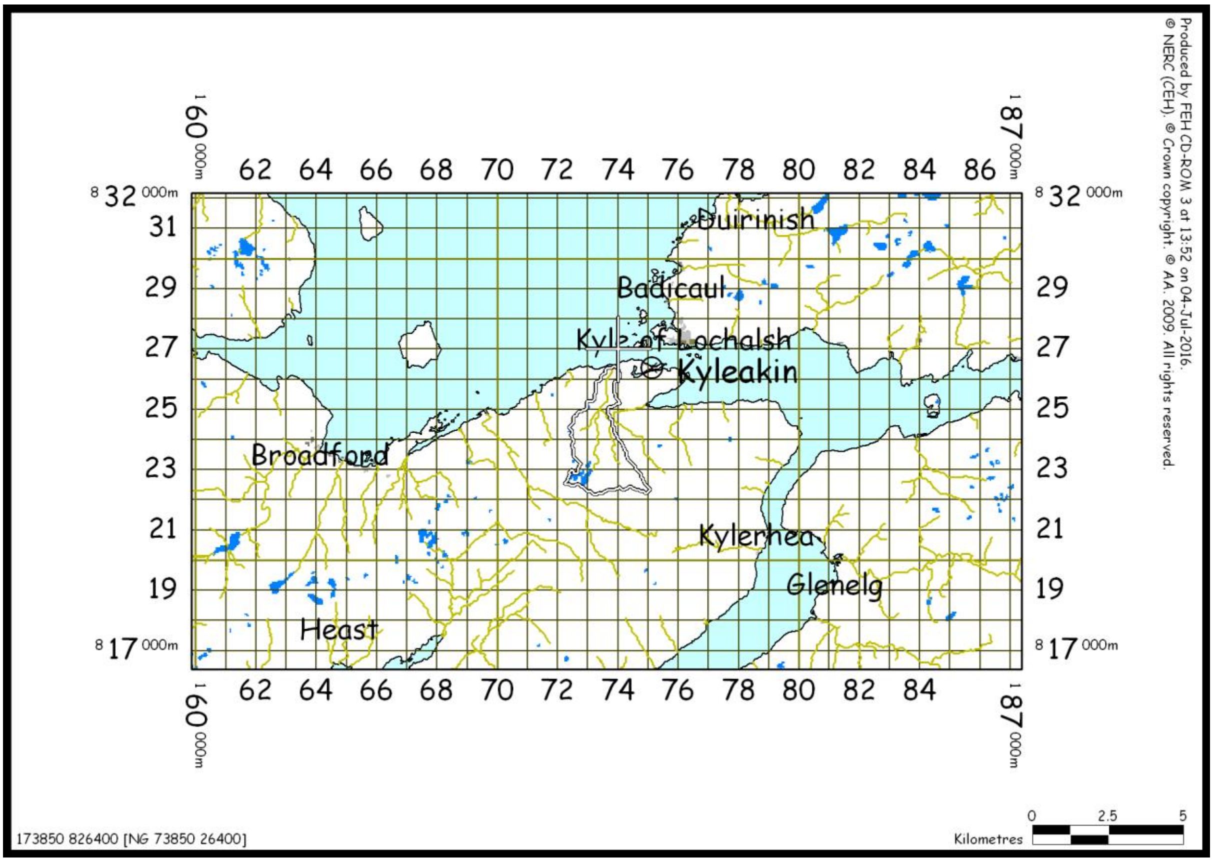
**GL**

| Return period (yrs) | Growth factors | Design flows ($m^3/s$) |
|---|---|---|
| 2 | 1.000 | 8.1 |
| 5 | 1.331 | 10.8 |
| 10 | 1.561 | 12.6 |
| 25 | 1.882 | 15.2 |
| 50 | 2.149 | 17.4 |
| 100 | 2.443 | 19.7 |
| 200 | 2.770 | 22.4 |
| 500 | 3.258 | 26.3 |

**GEV** ***for comparison***

| Return Period (yrs) | Growth factors | Design flows ($m^3/s$) |
|---|---|---|
| 2 | 1.000 | 8.1 |
| 5 | 1.366 | 11.0 |
| 10 | 1.605 | 13.0 |
| 25 | 1.901 | 15.4 |
| 50 | 2.118 | 17.1 |
| 100 | 2.330 | 18.8 |
| 200 | 2.539 | 20.5 |
| 500 | 2.810 | 22.7 |

**Appendix 1** **Location of catchment**



## Appendix 2 Pooling Group Details – Graphs

## Appendix 3 Pooling Group Details – Tables

| Station | Yrs | L-CV | L-Skew | L-Kurt | Discordancy | Distance |
|---|---|---|---|---|---|---|
| | | | | | | |
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| Station | Distance | AREA | SAAR | FPEXT | FARL | URBEXT 2000 |
|---|---|---|---|---|---|---|
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# Appendix B. Water Level Results

**Table 14: Baseline Peak Water Levels**

| Model Node | Baseline Peak Water Level (mAOD) | | |
|---|---|---|---|
| | 0.5% AEP | 0.5% AEP + CC | 0.1% AEP |
| ANA01_0309r | 13.368 | 13.496 | 13.524 |
| ANA01_0309wu | 13.368 | 13.496 | 13.524 |
| ANA01_0309wd | 12.682 | 12.735 | 12.747 |
| ANA01_0309 | 12.682 | 12.735 | 12.747 |
| ANA01_0294 | 10.407 | 10.489 | 10.507 |
| ANA01_0279 | 8.309 | 8.438 | 8.465 |
| ANA01_0264 | 8.056 | 8.163 | 8.186 |
| ANA01_0249 | 8.113 | 8.136 | 8.136 |
| ANA01_0234 | 8.172 | 8.223 | 8.230 |
| ANA01_0219 | 8.201 | 8.262 | 8.273 |
| ANA01_0210 | 8.238 | 8.321 | 8.337 |
| ANA01_0210c | 6.759 | 6.865 | 6.876 |
| ANA01_0010c | 4.833 | 4.971 | 4.976 |
| ANA01_0010 | 2.570 | 2.770 | 2.570 |

**Table 15: Baseline Peak Flows**

| Model Node | Baseline Flow ($m^3/s$) | | |
|---|---|---|---|
| | 0.5% AEP | 0.5% AEP + CC | 0.1% AEP |
| ANA01_0309r | 30.11 | 36.13 | 37.51 |
| ANA01_0309wu | 30.02 | 36.03 | 37.42 |
| ANA01_0309wd | 30.02 | 36.03 | 37.42 |
| ANA01_0309 | 30.02 | 36.03 | 37.42 |
| ANA01_0294 | 30.02 | 36.03 | 37.42 |
| ANA01_0279 | 30.02 | 36.03 | 37.42 |
| ANA01_0264 | 30.02 | 36.03 | 37.42 |
| ANA01_0249 | 30.02 | 36.03 | 37.42 |
| ANA01_0234 | 30.02 | 36.03 | 37.42 |
| ANA01_0219 | 30.02 | 36.03 | 37.42 |
| ANA01_0210 | 13.87 | 13.89 | 13.92 |
| ANA01_0210cu | 13.87 | 13.89 | 13.92 |
| ANA01_0210c | 13.87 | 13.89 | 13.92 |
| ANA01_0010c | 13.87 | 13.89 | 13.92 |
| ANA01_0010 | 13.87 | 13.89 | 13.92 |

**Table 16: Baseline Peak Velocities**

| Model Node | Baseline Velocity (m/s) | | |
|---|---|---|---|
| | 0.5% AEP | 0.5% AEP + CC | 0.1% AEP |
| ANA01_0309r | 1.02 | 1.02 | 1.02 |
| ANA01_0309wu | 1.02 | 1.02 | 1.02 |
| ANA01_0309wd | 1.02 | 1.02 | 1.02 |
| ANA01_0309 | 4.68 | 4.96 | 5.02 |
| ANA01_0294 | 5.44 | 5.73 | 5.79 |
| ANA01_0279 | 2.92 | 3.12 | 3.16 |
| ANA01_0264 | 2.80 | 3.07 | 3.13 |
| ANA01_0249 | 2.74 | 2.73 | 2.74 |
| ANA01_0234 | 3.06 | 3.06 | 3.06 |
| ANA01_0219 | 2.58 | 2.57 | 2.57 |
| ANA01_0210 | 1.44 | 1.44 | 1.44 |
| ANA01_0210cu | 2.52 | 2.52 | 2.52 |
| ANA01_0210c | 4.42 | 4.42 | 4.43 |
| ANA01_0010c | 4.42 | 4.42 | 4.43 |
| ANA01_0010 | 0.00 | 0.00 | 0.00 |

**Table 17: Design Option Peak Water Levels**

| Model Node | Design Option Peak Water Level (mAOD) | | |
|---|---|---|---|
| | 0.5% AEP | 0.5% AEP + CC | 0.1% AEP |
| ANA01_0309r | 12.776 | 12.949 | 12.986 |
| ANA01_0309wu | 12.776 | 12.949 | 12.986 |
| ANA01_0309wd | 12.440 | 12.512 | 12.527 |
| OPT_0280wu | 12.776 | 12.949 | 12.986 |
| OPT_0280wd | 11.690 | 11.690 | 11.690 |
| ANA01_0309d | 12.440 | 12.512 | 12.527 |
| ANA01_0294 | 10.026 | 10.142 | 10.165 |
| ANA01_0279 | 7.577 | 8.158 | 8.363 |
| ANA01_0264 | 7.202 | 8.150 | 8.359 |
| ANA01_0249 | 6.776 | 8.167 | 8.374 |
| ANA01_0234 | 5.929 | 8.177 | 8.384 |
| ANA01_0219 | 5.499 | 8.183 | 8.390 |
| ANA01_0210 | 5.449 | 8.175 | 8.385 |
| ANA01_0210c | 4.903 | 6.718 | 6.909 |
| ANA01_0010c | 4.087 | 4.819 | 4.991 |
| ANA01_0010 | 2.570 | 2.770 | 2.570 |
| OPT_0280 | 12.776 | 12.949 | 12.986 |
| OPT_0247 | 12.597 | 12.771 | 12.808 |
| OPT_0217 | 12.458 | 12.636 | 12.674 |
| OPT_0217c | 10.689 | 10.742 | 10.753 |
| OPT_0155c | 8.282 | 8.364 | 8.381 |
| OPT_0155 | 7.440 | 7.485 | 7.494 |
| OPT_0121 | 7.400 | 7.474 | 7.490 |
| OPT_0121s | 7.366 | 7.440 | 7.457 |
| OPT_0096s | 7.366 | 7.440 | 7.457 |
| OPT_0096 | 7.332 | 7.407 | 7.424 |
| OPT_0080u | 7.331 | 7.407 | 7.423 |
| OPT_0080c | 6.252 | 6.290 | 6.299 |
| OPT_0061c | 5.584 | 5.631 | 5.641 |
| OPT_0061d | 5.234 | 5.266 | 5.272 |
| OPT_0026 | 4.426 | 4.465 | 4.473 |
| OPT_0000 | 4.281 | 4.329 | 4.339 |

JACOBS

**Table 18: Design Option Peak Flows**

| Model Node | Design Option Peak Flow (m³/s) | | |
|---|---|---|---|
| | 0.5% AEP | 0.5% AEP + CC | 0.1% AEP |
| ANA01_0309r | 30.11 | 36.13 | 37.51 |
| ANA01_0309wu | 9.04 | 14.03 | 15.21 |
| ANA01_0309wd | 9.04 | 14.03 | 15.21 |
| OPT_0280wu | 3.85 | 3.70 | 3.82 |
| OPT_0280wd | 3.85 | 3.70 | 3.82 |
| ANA01_0309d | 9.04 | 14.03 | 15.21 |
| ANA01_0294 | 9.04 | 14.03 | 15.21 |
| ANA01_0279 | 9.04 | 14.01 | 15.21 |
| ANA01_0264 | 9.04 | 13.93 | 15.21 |
| ANA01_0249 | 9.04 | 13.86 | 15.21 |
| ANA01_0234 | 9.04 | 13.81 | 15.21 |
| ANA01_0219 | 9.03 | 13.79 | 15.21 |
| ANA01_0210 | 9.02 | 13.77 | 13.93 |
| ANA01_0210cu | 9.02 | 13.77 | 13.93 |
| ANA01_0210c | 9.02 | 13.77 | 13.93 |
| ANA01_0010c | 9.02 | 13.77 | 13.93 |
| ANA01_0010 | 9.02 | 13.77 | 13.93 |
| OPT_0247 | 21.05 | 22.09 | 22.30 |
| OPT_0217 | 21.05 | 22.09 | 22.30 |
| OPT_0217ci | 21.05 | 22.09 | 22.30 |
| OPT_0217c | 21.05 | 22.09 | 22.30 |
| OPT_0155c | 21.05 | 22.09 | 22.30 |
| OPT_0155co | 21.05 | 22.09 | 22.30 |
| OPT_0155 | 21.05 | 22.09 | 22.30 |
| OPT_0121 | 21.05 | 22.09 | 22.30 |
| OPT_0121s | 21.05 | 22.09 | 22.30 |
| OPT_0096s | 21.04 | 22.08 | 22.30 |
| OPT_0096 | 21.04 | 22.08 | 22.30 |
| OPT_0080u | 21.04 | 22.08 | 22.30 |
| OPT_0080ci | 21.04 | 22.08 | 22.30 |
| OPT_0080c | 21.04 | 22.08 | 22.30 |
| OPT_0061c | 21.04 | 22.08 | 22.30 |
| OPT_0061co | 21.04 | 22.08 | 22.30 |
| OPT_0061d | 21.04 | 22.08 | 22.30 |
| OPT_0026 | 21.04 | 22.08 | 22.30 |
| OPT_0000 | 21.04 | 22.08 | 22.30 |

**Table 19: Design Option Peak Velocities**

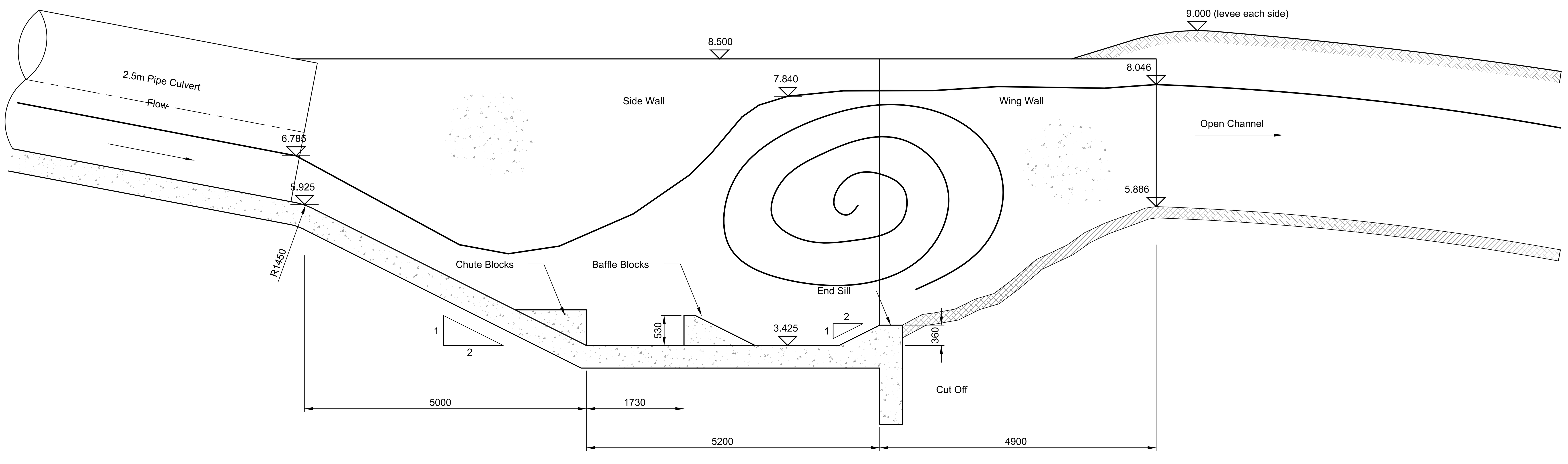
| Model Node | Design Option Peak Velocity (m/s) | | |
|---|---|---|---|
| | 0.5% AEP | 0.5% AEP + CC | 0.1% AEP |
| ANA01_0309r | 1.02 | 1.02 | 1.02 |
| ANA01_0309wu | 1.02 | 1.02 | 1.02 |
| ANA01_0309wd | 1.02 | 1.02 | 1.02 |
| OPT_0280wu | 1.02 | 1.02 | 1.02 |
| OPT_0280wd | 1.02 | 1.02 | 1.02 |
| ANA01_0309d | 3.02 | 3.52 | 3.63 |
| ANA01_0294 | 3.83 | 4.28 | 4.39 |
| ANA01_0279 | 2.20 | 2.42 | 2.44 |
| ANA01_0264 | 2.11 | 2.37 | 2.39 |
| ANA01_0249 | 2.55 | 2.69 | 2.69 |
| ANA01_0234 | 3.02 | 3.02 | 3.04 |
| ANA01_0219 | 2.52 | 2.51 | 2.49 |
| ANA01_0210 | 1.43 | 1.42 | 1.42 |
| ANA01_0210cu | 2.52 | 2.52 | 2.52 |
| ANA01_0210c | 3.26 | 4.38 | 4.44 |
| ANA01_0010c | 2.87 | 4.38 | 4.44 |
| ANA01_0010 | 0.00 | 0.00 | 0.00 |
| OPT_0247 | 1.32 | 1.32 | 1.32 |
| OPT_0217 | 1.79 | 1.79 | 1.79 |
| OPT_0217ci | 0.38 | 0.38 | 0.38 |
| OPT_0217c | 5.60 | 5.70 | 5.72 |
| OPT_0155c | 4.86 | 4.93 | 4.94 |
| OPT_0155co | 1.99 | 1.99 | 1.99 |
| OPT_0155 | 2.75 | 2.75 | 2.75 |
| OPT_0121 | 1.72 | 1.72 | 1.72 |
| OPT_0121s | 1.60 | 1.60 | 1.60 |
| OPT_0096s | 1.60 | 1.60 | 1.60 |
| OPT_0096 | 1.83 | 1.82 | 1.82 |
| OPT_0080u | 1.12 | 1.12 | 1.12 |
| OPT_0080ci | 1.60 | 1.60 | 1.60 |
| OPT_0080c | 5.03 | 5.11 | 5.13 |
| OPT_0061c | 3.79 | 3.85 | 3.87 |
| OPT_0061co | 1.60 | 1.60 | 1.60 |
| OPT_0061d | 2.74 | 2.77 | 2.78 |
| OPT_0026 | 2.51 | 2.53 | 2.53 |
| OPT_0000 | 1.56 | 1.57 | 1.57 |

# Appendix D. Design and Alignment Drawings

Drawing number / Rev

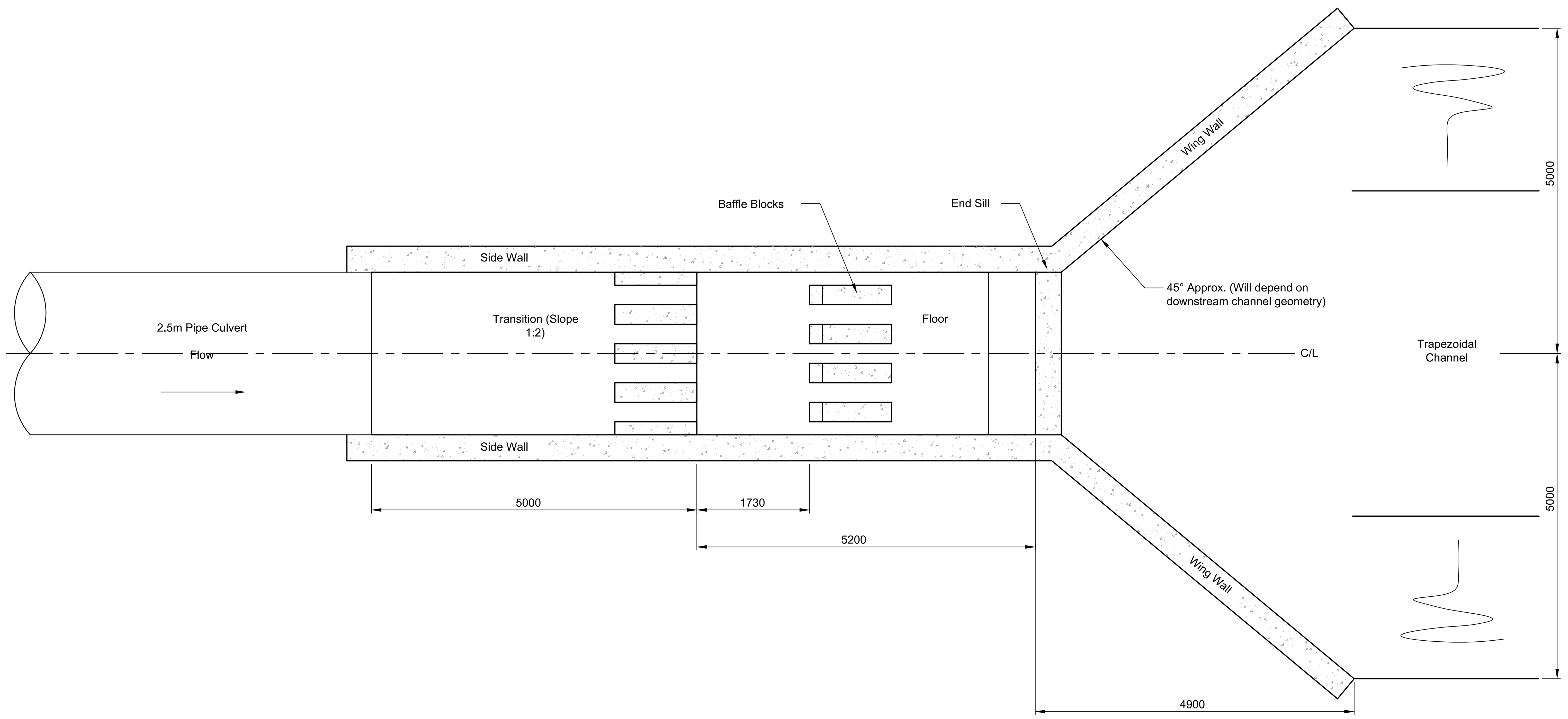
B2264800 (96) 601-1 Rev A

1. Do not scale from drawing - if in doubt ASK.
2. All dimensions in millimetres unless otherwise indicated.
3. This drawing is to be read in conjunction with all relevant architectural, civil, structural, and service engineer's drawings and specifications.




## LONGITUDINAL SECTION

Scale 1:50




## PLAN

Scale 1:50

0 1000 2000 3000 4000 5000 mm

SCALE 1 : 50

P:\Data\Engineering\Jobs\B2264800 - Kyleakin Fish Feed Factory\3. Technical\3.2 CAD\Working\(96) Drainage\B2264800-(96)-601-1 Rev A 2.5m Culvert.dwg - 03/03/2017 14:58:37 - A1Frame - Btaylor

| Rev | Rev. Date | Purpose of revision | Drawn | Checkd | Rev'd | Apprv'd |
|---|---|---|---|---|---|---|
| A | 03.03.17 | FOR INFORMATION | KMA | KS | KS | AB |

JACOBS

Client

MARINE HARVEST

Project

Fish Feed Plant, Allt Anavig Quarry, Kyleakin, Isle of Skye

Drawing title

ENERGY DISSIPATION STRUCTURE NORTH OF 2.5m CULVERT

Drawing status

FOR INFORMATION

| | | |
|---|---|---|
| Scale | 1:1000 @ A1 | DO NOT SCALE |
| Jacobs No. | B2264800 | |
| Client no. | | |

Drawing number

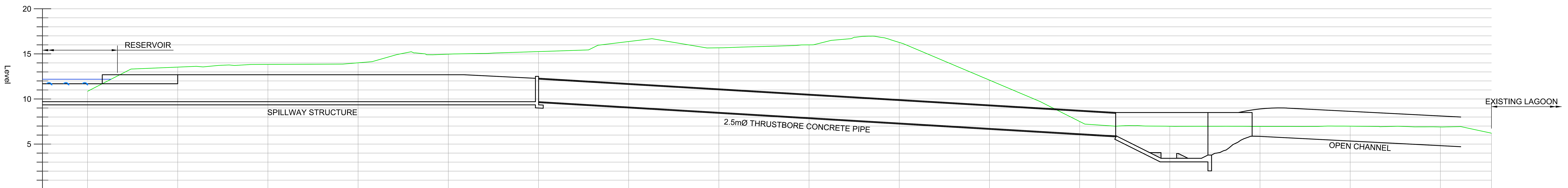
B2264800 (96) 601-1

Rev

A

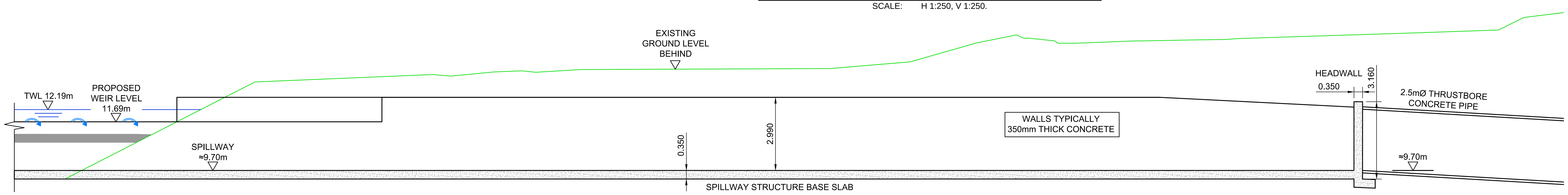
This drawing is not to be used in whole or part other than for the intended purpose and project as defined on this drawing. Refer to the contract for full terms and conditions.

Drawing number / Rev
B2264800 (96) 525-1 Rev B

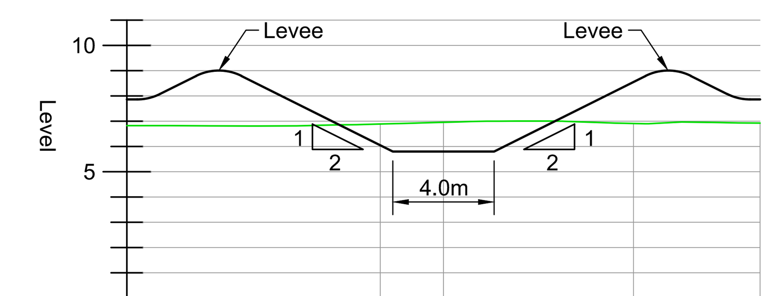



| Chainage | 00.00 | 10.00 | 20.00 | 30.00 | 40.00 | 50.00 | 60.00 | 70.00 | 80.00 | 90.00 | 100.00 | 110.00 | 114.00 | 120.00 | 130.00 | 140.00 | 150.00 | 155.67 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing Levels (mAOD) | 12.51 | 13.54 | 13.83 | 14.01 | 14.97 | 15.26 | 16.37 | 15.67 | 16.00 | 16.27 | 12.09 | 7.51 | 7.40 | 6.98 | 6.96 | 6.99 | 6.90 | 6.21 |
| Invert Level | 11.69 11.19 | | | | | 9.700 | | | | | | | 5.90 | 3.43 | | | | |

## LONG SECTION THROUGH SPILLWAY, THRUSTBORE PIPE AND OPEN CHANNEL
SCALE: H 1:250, V 1:250.

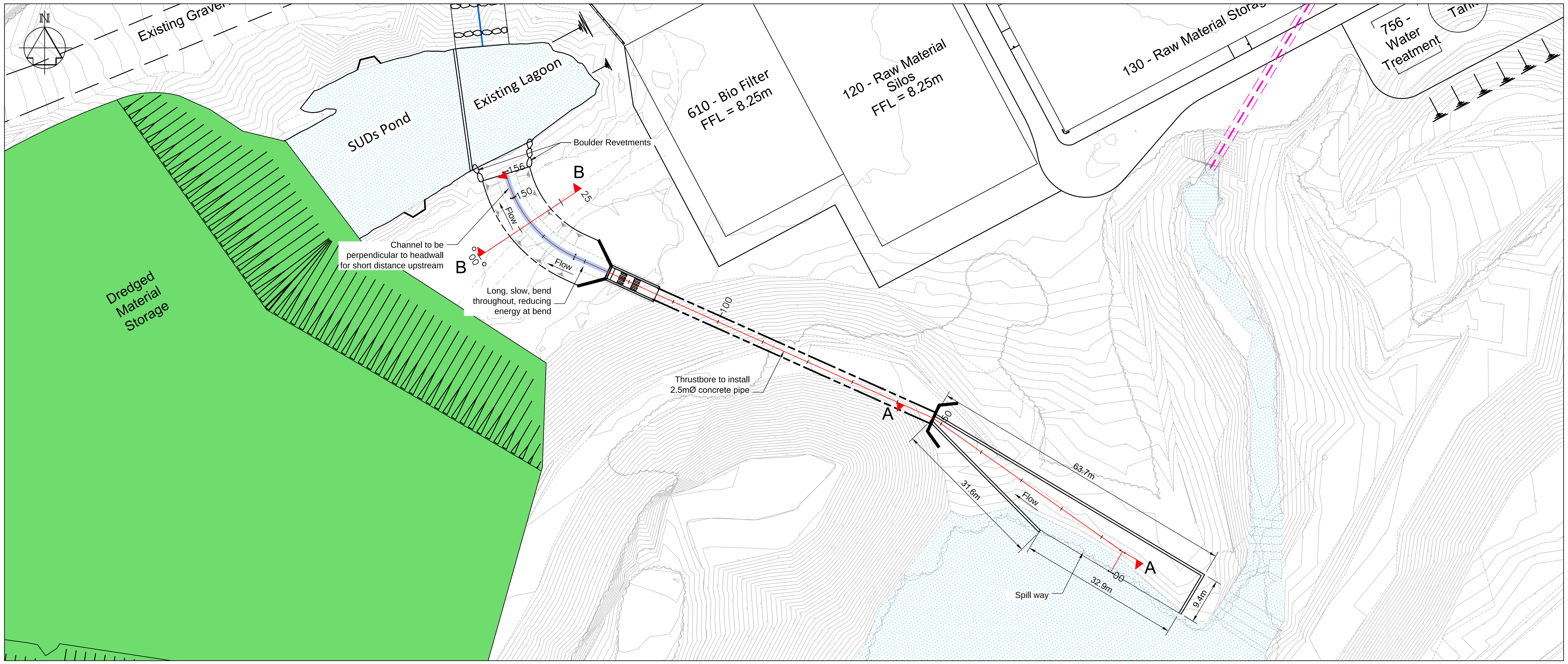



## SECTION A - A
Scale 1:100




| Chainage | 00.00 | 10.00 | 20.00 | 25.00 |
|---|---|---|---|---|
| Existing Levels (mAOD) | 6.825 | 6.889 | 6.909 | 6.925 |

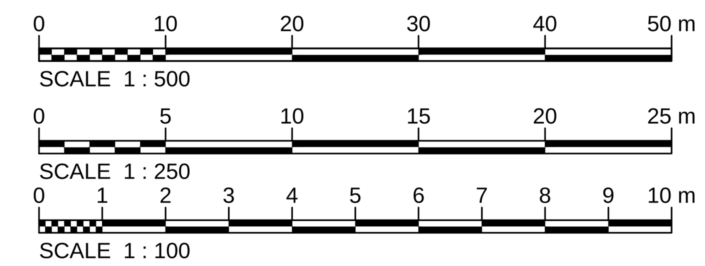
## CROSS SECTION THRO 'CHANNEL B - B
SCALE: H 1:250, V 1:250.




## SECTION PLAN
Scale 1:500

Notes:

1. Open channel to be lined with Terram NP geotextile installed below cobble mattress and boulder revetments.




| Rev | Rev. Date | Purpose of revision | Drawn | Checkd | Rev'd | Apprv'd |
|---|---|---|---|---|---|---|
| B | 03.03.17 | CULVERT END STRUCTURE ADDED | BT | KS | KS | AB |
| A | 09.02.17 | BUILDING WARRANT ISSUE | EM | KS | KS | AB |
| P1 | 07.02.17 | FOR INFORMATION | EM | KS | KS | AB |

JACOBS

Client
MARINE HARVEST

Project
Fish Feed Plant, Allt Anavig Quarry, Kyleakin, Isle of Skye

Drawing title
LONG SECTION THROUGH PROPOSED RESERVOIR OUTLET

Drawing status
FOR INFORMATION

| | | |
|---|---|---|
| Scale | As Shown @ A1 | DO NOT SCALE |
| Jacobs No. | B2264800 | |
| Client no. | | |

Drawing number
B2264800 (96) 525-1

Rev
B

This drawing is not to be used in whole or part other than for the intended purpose and project as defined on this drawing. Refer to the contract for full terms and conditions.

P:\Data\Engineering\Jobs\B2264800 - Kyleakin Fish Feed Factory\3. Technical\3.2 CAD\Working\(96) Drainage\B2264800-(96)-525-1 Rev B RESERVOIR OUTLET DRAINAGE.dwg - 03/03/2017 14:56:36 - A1 - Btaylor