Flood Risk Statement and Drainage Impact Assessment

Corshellach Battery Energy Storage System

Ref	04876-6315507

Revision History

Issue	Date	Name	Latest changes
01	15.04.24	William Miskelly	First created

Contents

1 Overview

1.1 Introduction

Corshellach BESS is a proposed battery-based energy storage system located 11km South of the town of Forres, Moray, Scotland.

This report sets out the flood risk screening and surface water management plan for the proposed Corshellach battery energy storage system, which will house battery enclosures along with associated infrastructure and electrical equipment.

The battery storage system comprises battery enclosures with associated power conversion systems, transformers and grid compliance equipment. All electrical equipment will be set on concrete foundations.

Drawing 04876-RES-LAY-DR-PT-001 included in Appendix A, shows the proposed project layout. The compound area within the fence measures 0.79 hectares, the total area enclosed by the red line boundary measures 3.84 hectares.

Relevant Moray Council compliance checklists and certificates are included in Appendix E.

2 Relevant Guidance and Legislation Requirements

This report uses best practice and conforms with the requirements of the relevant regulatory authorities.

The key legislation and guidance adhered to are as follows:

- Flood Risk and Drainage Impact Assessment for New Developments (Moray Council Supplementary Guidance).
- The EU Water Framework Directive (2000/60/EC).
- Scottish Planning Policy.
- The Water Environment (Controlled Activities) (Scotland) Regulations 2011.
- SEPA Pollution Prevention Guidance Notes (PPPGs).
- Engineering in the Water Environment, Good Practice Guide, Temporary Construction Methods, First Edition, March 2009.
- Sewers for Scotland 3rd Edition.
- The Sustainable Urban Drainage Scottish Working Party (SUDSWP) Water Assessment and Drainage Assessment Guide.
- Environmental Good Practice on Site, CIRIA C692, 3rd Edition.
- Control of Water Pollution on Construction Sites, CIRIA C532.
- The SUDS Manual 2015. CIRIA C753.
- British Geological Survey (BGS) Maps.
- Ciria C786 Culvert screen and outfall manual.

3 Existing Information

3.1 Site Location

The proposed site sits in Dunphail, approx. 11km south of Forres along the A940. Berryburn Wind Farm lies 3km south of the site and Logie Wind Farm lies 800m North of the site.

Access for construction will be taken from the North, from the junction at Tomnamoon. Light vehicle access for maintenance will be from the West from the junction adjacent to the Edinkillie Church onto the unnamed road heading East towards the site.

3.2 Existing Land Use and Topography

A walkover survey of the site has been undertaken, and a topographical survey of the site extents carried out to confirm the existing land use and topography. The existing site land use is for low intensity agricultural purposes.

Ground levels on site falls from approx. 244m AOD in the Northern corner to 215m AOD in the Southern corner. Levels fall at an approximate gradient of 1:13.

3.3 Ground Conditions

A review of the superficial deposits shows most of the site sits within the Beinn an Uain till formation, which is a diamicton with subsidiary clay, sand and silt. There are no publicly available borehole records near to the site.

3.4 Existing Hydrology / Drainage

The site appears to drain via overland flow into multiple watercourses that are located through the site and along the Southwestern boundary of the site, as well as a watercourse located parallel to the Southeastern boundary outside the site boundary.

The flows from the sites watercourse's flow into the Stripe of Corshellach, which subsequently flow into the Berry Burn / River Divie. SEPA classify the River Divie as a 'good' quality surface water body.

SEPA mapping classify the quality of groundwater underneath and around site as 'good'. The site does not fall in a protected area as defined by SEPA.

There are multiple watercourses on the site, these were found in a topographical survey. The topographical survey included a buried services survey which found no land drains, the survey was undertaken in 2023.

The topographical survey is included in Appendix D.

4 Flood Risk Screening

4.1 Overview

The proposed battery energy storage compound is deemed not at risk from flooding as set out in this flood screening section. As per Moray Council's Flood Risk and Drainage Impact Assessment for New Developments guidance, the Moray Council were contacted prior to designing the development, Moray Council confirmed that no flood risk assessment was required, correspondence record is provided in Appendix C.

The development boundary includes an existing private road that will be used to access the site during construction. The junction between this road and the public road U89E is located in an area at high risk of flooding. As per Moray Council's Flood Risk and Drainage Impact Assessment for New Developments guidance the development will not lead to an increased flood risk and safe access and egress can be achieved as detailed in section 4.2.

4.2 Flooding from Fluvial Sources and Surface Water

4.2.1 General

Figure 1 below depicts the SEPA flood risk map, with the proposed site compound and private access road boundary overlaid. As can be observed in Figure 1 the battery energy storage compound (shown in green) does not lie in an area at risk of flooding from fluvial sources (blue zones), or surface water (purple zones). The private access track shown in red in Figure 1 does lie in an area at high risk of flooding (defined as 10% Annual Exceedance Percentage) with a flood depth of up to 1m. This flood zone is located at the private access road junction with the U89E public road. There will be no construction taking place within the flood zone.

This access point will be used during construction, there are no suitable alternative access routes for construction traffic.

During the operational phase of the project, an alternative access route using the U88E Diverside Road may be used. This route is not at risk of any fluvial or surface water flooding.

4.2.2 Flood Warning & Flood Emergency Plan

With reference to the Scottish Environment Protection Agency's website, no general early notification of possible (fluvial and tidal) flooding, known as 'Floodline' is available for the land at risk of flooding from the Burn of Auldusack (name of watercourse causing the flooding). However, the Scottish Environment Protection Agency 'Floodline' flood warning service is available for the wider Findhorn Nairn Moray & Speyside area. The site will therefore subscribe to this service which aims to provide warning of an impending flood. During the construction phase, visitors and employees at the site will therefore have sufficient time to evacuate the site.

During the operational phase, the alternative access will be used to evacuate the site in the event of a flood warning.

The flood risk to the site entrance and any actions to take as a result of flooding will be documented in the form of a Flood Emergency Plan.

Figure 1 - Excerpt from SEPA surface water and fluvial flood risk map, with proposed site boundary overlaid.

4.3 Flooding from Groundwater

SEPA flood risk mapping shows the proposed development site does not lie in an area at risk of groundwater flooding.

4.4 Flooding from Tidal or Sea Flooding

The development site is located outside of any area of tidal influence based on its ground elevation above ordnance datum of >100m AOD. The proposed development is therefore not considered at risk of tidal or sea flooding.

4.5 Flooding from Sewers

There are no surface water sewers or highway drains on the development site. Therefore, the development is not considered at risk of flooding from sewers.

4.6 Flooding as a Result of the Development

The existing flow regime will remain unchanged as a result of the development as set out in Sections 5 and 6 of this report. Therefore, the development is not considered to exacerbate the flood risk of the surrounding area.

4.7 Historic Flooding

There are no known records of flooding at the location of the BESS compound according to the SEPA flood maps and to the knowledge of the Landowner.

5 Drainage Design Options

5.1 Foul Drainage

There will be no permanent foul drainage from the proposed development.

Any foul drainage from the temporary welfare facilities will be self-contained and disposed off-site appropriately.

5.2 Surface Water Drainage Discharge Options

5.2.1 General

As per Moray Council's Planning Policy EP5 as described in 'Flood Risk and Drainage Impact Assessment for New Developments', the proposed development should be drained by a sustainable urban drainage system. As such, the SUDS Hierarchy will be applied, and adequate infiltration testing to BRE 365 Digest will be undertaken to determine the viability of an infiltration-based drainage solution.

The drainage infrastructure is to be contructed prior to other permanent aspects of the BESS compound to reduce the likilhood of increased flood risk or decreased water quality during construction.

5.2.2 Rainwater Re-Use

Rainwater re-use is not applicable to this project; there are no facilities within the proposed development that have a demand for water.

5.2.3 Infiltration

Based on the hierarchy identified in Section 5.2.1, the preferred method of surface water discharge is via infiltration to the ground. However, the ground on site is not anticipated to support drainage by infiltration due to the following:

- Greenfield runoff rate estimation tool created by HR Wallingford supports this assumption as it identifies the land as soil type 5 indicating a clay soil and therefore lack of suitability of infiltration methods.
- The existing drainage systems on the site and surrounding the site utilise drainage channels to convey overland flows during storm events indicating a lack of suitability of infiltration methods.

5.2.4 Attenuate Rainwater in Ponds for Gradual Release

Refer to the infrastructure layout provided in Appendix A for details of the drainage layout.

If infiltration testing shows an infiltration-based drainage solution is not possible, the next preference in the SUDS Hierarchy is to attenuate flows in an on-site attenuation basin, discharging from site at a rate that does not exceed that of pre-existing greenfield conditions. Due to the low probability of infiltration capacity on site, it is assumed for design purposes that the attenuation basin design is the highest option on the SUDS Hierarchy that is viable for the proposed development site.

The surface water drainage will be designed in accordance with the guidance in Section 2, and Section 5.2.1. Flows will be restricted to Qbar and the attenuation basin will be sized to contain the 1 in 200 rainfall event plus a 42% allowance for climate change.

The preferred discharge point for the restricted flow will be to the existing watercourse located on the site, therefore matching existing drainage routes.

5.3 Water Channnel Crossings

As per Moray Council's Flood Risk and Drainage Impact Assessment for New Developments guidance, unnecessary engineering works in the water environment have been avoided as much as possible. To reach the site, it is necessary to cross a water channel that runs parallel to the private road with which the site is accessed from. This water channel will be crossed by installing two culverts, the design of these culverts are described in section 7.3.

6 Development Proposal

6.1 Site Preparation

As part of site preparation, existing topsoil on site will be scraped off and set aside for re-use in the landscaping scheme. For the proposed areas of permanent hardstanding on site (inside the compound) and the proposed tracks, the preferred surfacing will comprise permeable unbound granular material.

The compound and tracks will facilitate construction traffic and allow safe installation of the electrical infrastructure.

The compound will be graded appropriately in line with existing falls, ensuring a fall within the compound does not exceed 2%.

6.2 Management of Surface Water Flows

6.2.1 Post Development Surface Water Runoff

The proposed compound on the development will result in a permanent hardstanding area of in the order of 0.79 ha. To ensure adequate allowances are made at this stage in the project, it is assumed for storage calculations that permanent hardstanding will comprise asphalt, entirely impermeable with a runoff coefficient of 1.

Stormwater uphill of the site on the Northwestern side will be intercepted by a drainage channel which will run parallel to the site boundary. From this drainage channel, water will be conveyed to an outfall and discharged into an existing drainage channel running through the site. No change in ground permeability is proposed on the land located uphill of this drainage channel therefore no increase in storm flows is expected. Water intercepted by this Northwestern drainage channel will discharge into the existing drainage channel (see appendix A for drawing) without restriction imposed by flow control.

6.2.2 Proposed Attenuation Basin Design

It is proposed to use an attenuation basin to limit off-site surface water runoff from the permanent hardstanding areas on site. Ground levels on site fall from North to South. The proposed attenuation basin is located at the site's south-western corner.

The Corshellach Infrastructure Plan (included in Appendix A) shows the proposed attenuation basin design. The basin has been designed with a plan area and depth sufficient to accommodate storm flows generated on site during a 200-year event including an additional 42% allowance for climate change. To mitigate ground stability risk and slip / trip risk, basin slopes are limited to 1:3.

Attenuation calculations are summarised in Section 7 and included in Appendix B. Interception losses, such as those provided by on-site topsoil / grass, hedgerows, and vegetation, are neglected from these calculations as a conservative measure.

6.2.3 Water Quality and Treatment

In line with the requirements noted in the Moray Council Flood Risk and Drainage Impact Assessment for New Developments guidance document listed in Section 2, a Simple Index Approach is undertaken to ensure the proposed drainage strategy provides adequate water quality treatment, as per Section 26.7.1 of the SUDS Manual 2015 (CIRIA C753).

As a conservative approach, the proposed development is considered a medium pollution hazard level based on land use definitions provided in Table 26.2 of the SUDS Manual. The corresponding pollution hazard indices are denoted in Table 1.

Surface water within the proposed development will receive minimum three stages of treatment before being discharged to the existing ditch. The three main stages are listed below:

1. Filtration of water through filter drain stone upstream of basin; mitigation indices for filter drain: TSS = 0.4, metals = 0.4, hydrocarbons = 0.4.

2. Settlement in attenuation basin; mitigation indices for detention basin: TSS = 0.5, metals = 0.5, hydrocarbons = 0.6.

3. Settlement in swale with drainage stone check dam to increase pollutant retention; mitigation indices for detention basin: TSS = 0.5, metals = 0.6, hydrocarbons = 0.6.

Table 1 below demonstrates how the pollution hazard index for each contaminant is satisfied by the three stages of water treatment provided as part of the proposed drainage strategy.

Table 1 - Simple Index Calculation

Contaminant Type	Stage 1	Stage 2	Stage 3	Total SUDS Mitigation Index	Pollution Hazard Index	Utilisation
Total Suspended solids	0.4	0.5(0.5)=0.25	0.5(0.5)=0.25	0.9	0.7	1.29
Metals	0.4	0.5(0.5)=0.25	0.5(0.6)=0.3	0.95	0.6	1.58
Hydrocarbons	0.4	0.5(0.6)=0.3	0.5(0.6)=0.3	1	0.7	1.43

During the construction phase, temporary silts fences will be installed, providing an additional treatment stage of water filtration.

6.2.4 Exceedance Flow Design

In accordance with CIRIA Report 753, an exceedance route should be considered as part of the SUDS design.

The exceedance route will remain as per the existing scenario, i.e., over vegetation down towards the existing watercourse South of the site.

To mitigate flood risk in the event of an exceedance, the attenuation basin will be located downslope of the energy storage facility. The resultant site levels will be such that surface water from any extreme events will flow over the banks of the attenuation basin away from the energy storage facility and then downslope overland away from the site. The edges of the attenuation basin will be vegetated to reduce the risk of scour during an extreme event.

6.2.5 Water Channel Crossing

In accordance with Ciria C786 Culvert Screen and Outfall Manual, appropriate culverts have been designed beneath the access tracks to cross the water channel running parallel to the site. Further details of the water channel crossing culverts can be found in section 7.3 and attached appendices.

6.2.6 SUDS Layout and Typical Details

Refer to Appendix A for indicative details and layout of the SUDS and other drainage infrastructure proposed across the site.

7 Hydraulic Assessment

A preliminary runoff and attenuation calculation for compound has been undertaken using a HR Wallingford online design tool available from:

https://www.uksuds.com/tools/greenfield-runoff-rate-estimation

The inputs taken have been assumed as "worst case" and as such has determined the maximum drainage component extents required for the project. This includes assuming all permanent infrastructure (other than the access track) has an asphalt surface, and that drainage by infiltration is not possible.

A detailed drainage design will be performed following the ground investigation and compound earthing design (to determine surface finishes).

All methods and inputs are taken in accordance with the relevant guidance documents provided in Section 2.

7.1 Greenfield Peak Runoff Rates from Site

Current and future greenfield runoff rates for the development have been estimated using the IH124 Method. Using the mapping software within HR Wallingford Design Tool, the site-specific parameters have been established:

- Standard average annual rainfall (SAAR6190): 854mm;
- Standard percentage run-off: 53%;
- Total drained area: 0.79ha;

Total drained area is defined as the catchment area for the attenuation basin, which comprises the BESS compound area (0.79ha), area extents are shown on the Infrastructure Layout in Appendix A.

Refer to Appendix B for the Qbar design tool calculation summary.

The peak runoff rate calculated for a Qbar rainfall event is 6.25 l/s. It is proposed to match this discharge rate through use of a flow control device installed in a manhole positioned immediately downstream of the basin.

7.2 Attenuation Storage Required Post Development

The surface water storage volume estimation tool uses a storage assessment method developed by HR Wallingford based on correlations between storage requirements and hydrological and hydraulic characteristics of sites.

Attenuation storage will be provided to accommodate the peak runoff rate calculated up to the critical 1 in 200 storm plus a 42% allowance for climate change.

Refer to Appendix B for the storage volume calculation summary.

As per the calculation described in Section 7.1, allowable discharge from the basin is set to the calculated greenfield runoff rate of 6.25 l/s.

Due to site levels and basin positioning as described in Section 6.2.2, the catchment area for the basin is defined as the BESS compound area, 0.79ha.

The attention volume calculated based on the above criteria is approximately 806m^3. 3D modelling has been carried out to prove this volume can be accommodated within the site boundary. The attenuation volume should be considered a maximum volume, this assumes that all permanent infrastructure (other than the access track) has an asphalt surface and that drainage by infiltration methods is not possible.

As per the Moray Council planning response shown in Appendix C, it is required that following a critical 1 in 30 year event (including 42% climate change), any proposed attenuation basin is able to empty within 24 hours. From the storage volume calculation included in Appendix; it can be observed that for the 30 year return period calculation, a maximum attenuation volume of 491m^3 is required. Based on an outfall rate of 6.25 l/s, 491m^3 would take 21.82 hours to drain completely.

7.3 Water Channel Crossing Culvert Design

The preliminary water channel crossing culvert designs have been designed in accordance with Ciria C786 Culvert Screen and Outfall Manual and the Moray Council's Flood risk and Drainage Impact Assessment for New Developments guidance.

Similarly to the compound drainage described earlier in section 7.2, the culvert was to be designed to a 1 in 200 storm with a 42% climate change allowance.

The flows going into the culverts have been determined by using the peak greenfield runoff rate of the upstream catchment during a 1 in 200 year storm event with 42% climate change allowance.

Total upstream catchment area is defined as the catchment area that flows into the water channel. This has been calculated using OS mapping to define the catchment area. Figure 2 shows the upsteam catchment area laid over OS mapping showing the contours of the area. The flows from the BESS compound and above the BESS compound are discharged downstream of the culverts.

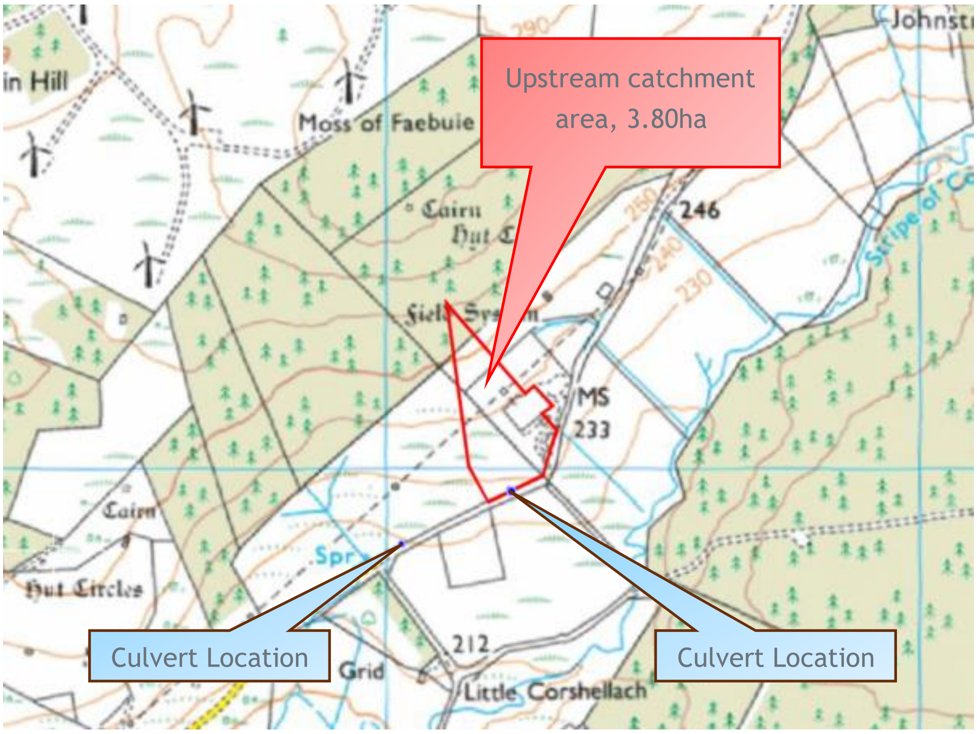

Figure 2 – Upstream Catchment Area & Culvert Locations

Current and future peak greenfield runoff rates for the upstream catchment have been estimated using the IH124 Method. Using the mapping software within HR Wallingford Design Tool, the site-specific parameters have been established:

- Standard average annual rainfall (SAAR6190): 854mm;
- Standard percentage run-off: 53%;
- Total upstream catchment area: 3.80ha;

Refer to Appendix B for the 1 in 200 design tool run off rate estimation summary. The 1 in 200 value of 85.33l/s then had the climate change factor of 42% applied resulting in a peak runoff rate of 121.2l/s which the culverts would need to be designed to accommodate.

Using Ciria C786 Culvert Screen and Outfall Manual, a preliminary calculation (attached in Appendix B) has been undertaken demonstrating a 450mm culvert could be placed in the water channel and not impede flows during a 1 in 200 plus climate change event at both crossings. Alternatively two smaller adjacent culverts could be used (i.e 2no. 300mm diameter culverts). The exact geometry (diameter, shape, material, etc.) of the culvert will be determined during the detailed design stage.

8 Operation and Maintenance Requirements

All surface water drainage and pollution control features associated with the site will remain private and will be maintained by the site operator.

The following section outlines the proposed maintenance for the various aspects of the drainage system. If necessary, these outline maintenance proposals will be refined when the site is operational to suit specific conditions.

A maintenance record log will be maintained for all maintenance work carried out. Where problems persist on each six-monthly inspection, advice will be sought from the SUDS designer on an alternative drainage solution.

8.1 Pipe & Catchpits

The anticipated maintenance plan for the site pipes and site compound catchpits is outlined in Table 2.

Table 2 – Typical Pipes and Catchpits Operation and Maintenance Requirements

Pipes, culverts and Catchpits Maintenance Schedule	
Maintenance Action	**Minimum Frequency**
Inspect manhole / pipe. Where pipe has become clogged with silt, the pipe will be cleared out.	Half yearly
Remove litter and debris.	Half yearly
Inspect inlets and outlets for blockages, and clear (if required).	Half yearly
Remove settled solids, litter and debris from catchpits.	Half yearly

8.2 Filter Drain

The anticipated maintenance plan for the filter drains at the site is outlined in Table 3.

Table 3 – Typical Filter Drain Maintenance Requirements

Filter Drain Maintenance Schedule	
Maintenance Action	**Minimum Frequency**
Inspect filter drain for silt contamination.	Half yearly
Replace drainage stone where necessary.	Half yearly
Remove litter and debris	Half yearly

8.3 Infiltration / Attenuation Basin

The anticipated maintenance plan for the basin at the site is outlined in Table 4.

Table 4 – Typical Basin Operation and Maintenance Requirements

Basin Maintenance Schedule	
Maintenance Action	**Minimum Frequency**
Remove litter and debris.	Half yearly
Inspect inlets and outlets for blockages, and clear (if required).	Half yearly
Inspect inlets and outlets for noticeable effects of erosion, suitable erosion protection measures such as reno-mattress or placement of large stones (>150mm) to dissipate water energy levels will be installed at the area affected.	Half yearly
Inspect silt accumulation rates in any forebay and in main body of the pond and establish appropriate removal frequencies.	Half yearly
Reseed areas of poor vegetation growth, alter plant types to better suit conditions (if required).	As required, or if bare soil is exposed over 10% or more of the basin treatment area

8.4 Swale

The anticipated maintenance plan for the swale at the site is outlined in Table 5.

Table 5 - Typical Swale Maintenance Requirements

Swale Maintenance Schedule	
Maintenance Action	**Minimum Frequency**
Inspect swale for silt contamination.	Half yearly
Remove litter and debris.	Half yearly
Cut grass along swale banks.	Half yearly

8.5 Water Channel Crossing Culverts

The anticipated maintenance plan for the water channel crossing culverts is outlined in Table 6.

Table 6 – Water Channel Culvert Maintenance Requirements

Water Channel Culvert Maintenance Schedule	
Maintenance Action	**Minimum Frequency**
Where pipe has become clogged with silt, the pipe will be cleared out.	Half yearly
Remove litter and debris.	Half yearly
Inspect inlets and outlets for blockages, and clear (if required).	Half yearly

9 Conclusion

A flood risk assessment has been undertaken across the site. The site has been deemed at low risk of flooding.

An assessment of the drainage options has also been undertaken, and it has been concluded that drainage by infiltration is unlikely to be a viable option. As such, the current proposal is to drain the site via an attenuation basin, with a restricted discharge rate into an existing drainage ditch. Infiltration testing will be undertaken on site prior to detail design, and should acceptable infiltration rates be found, an infiltration solution will be adopted during detail design.

The required attenuation volume has been calculated as $806m^3$. This should be considered a maximum volume, based on the assumption that all permanent infrastructure (other than the access track) has an asphalt surface and that drainage by infiltration methods is not possible.

A site investigation, 3D earthworks design, earthing design, and a further assessment of the proposed discharge will be undertaken to inform the detailed design of the site drainage.

The drainage strategy proposed will provide sufficient water quality treatment as demonstrated using the Simple Index Approach.

Appendix A Project Drawings

A.1 Infrastructure Layout – 04876-RES-LAY-DR-PT-001

A.2 Typical Drainage Details – 04876-RES-DRN-DR-PT-001

A.3 Typical Water Crossing Details – 04876-RES-DRN-DR-PT-002

A.4 Location Plan – 04876-RES-MAP-DR-XX-001

Appendix B Calculations & Supporting Information

B.1 Corshellach Greenfield Runoff Rate - 04876-7517994

B.2 Attenuation Pond – UK Storage Volume - 04876-7341126

B.3 Upstream Catchment Greenfield Runoff Rate - 04876-7332087

B.4 Corshellach Culvert Design - 04876-7344914

B.5 Corshellach Catchment Descriptor - 04876-7517999

Appendix C Correspondence with Moray Council

C.1 23.00239.PEMAJ – Drainage Response – 04876-7518657

C.2 RE_Corshellach – 04876-7223967

Appendix D Surveys

D.1 313053-SU01 Topographic 2D-A1 Landscape - 313053 SU001

Appendix E Compliance Documents

E.1 Level 1 Flood Risk Statement Checklist & Level 2 Drainage Impact Assessment Checklist

E.2 DIA Compliance Checklist

E.3 Indemnity Insurance Evidence