Gleann Astaile Dam - Design and Construction of a Geomembrane Lined Embankment Dam

M. J. HILL, MWH UK Ltd
I. C. CARTER, MWH UK Ltd

SYNOPSIS Gleann Astaile Reservoir was formed by raising a natural lochan to provide storage in support of a 2 MW hydropower scheme on the Isle of Jura, Scotland. The scheme was promoted by Inver LLP and is thought to be one of the largest privately developed hydropower scheme in the UK in recent times. The reservoir will have a storage volume of $0.7Mm^3$ and is impounded by a 9m high and 280m long earthfill embankment dam.

The geology at the site comprises Peat overlying Glacial Till and Quartzite bedrock - the Glacial Till being characterised by a silty-sandy gravel and cobbles. The lack of cohesive soil materials at the site meant that dam designs were based around artificial waterproof elements, including concrete core wall and geomembranes. Early contractor involvement was initiated by the Client to provide constructability, schedule and cost input at the conceptual design stage. An embankment dam incorporating an upstream geomembrane was ultimately selected on economic and constructability grounds.

The dam incorporates an upstream sloping HDPE geomembrane connected at the upstream toe via a concrete plinth founded on bedrock. The geomembrane was covered by a transition layer of fill and rip rap to provide greater protection against puncture and aging.

This paper describes the field investigation, design and construction of the dam and appurtenant works, with particular emphasis on the waterproof geomembrane works.

PROJECT DESCRIPTION

Gleann Astaile Reservoir was formed by enlargement of an existing lochan and construction of an earthfill embankment dam a short distance downstream of the neck of the lochan. The Full Supply Level (FSL) of the existing lochan was increased by 3m to 135.33mOD.

Dams: Engineering in a Social & Environmental Context. Thomas Telford, London, 2012

The embankment dam is approximately 280m long and incorporates a 15m long overflow weir and spillway over the central section of the embankment. The intake for the power penstock pipeline was formed within a shaft located within the embankment with an inlet culvert extending into the reservoir. The waterproof element of the embankment dam was formed by an upstream sloping HDPE geomembrane, which was secured at the crest within an anchor trench and connected at the upstream toe to a concrete plinth, which was cast against a prepared bedrock surface.

The general arrangement of the dam and ancillary works is shown in Figure 1.

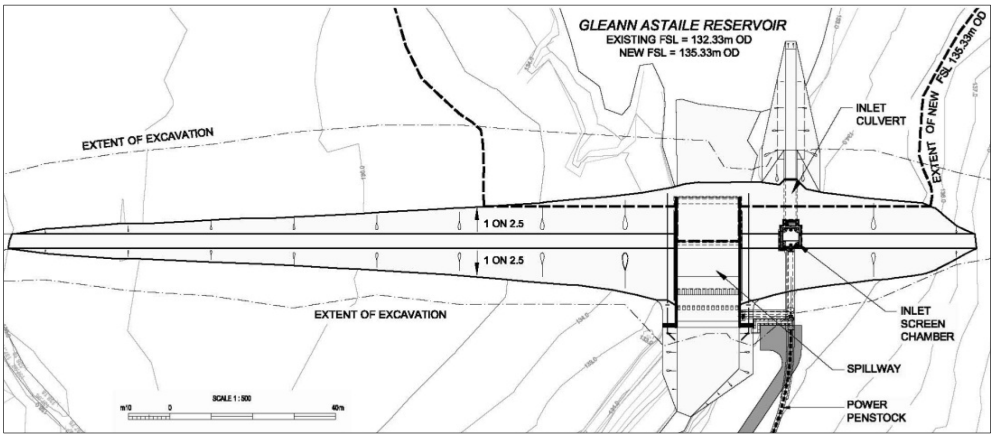


Figure 1. General Arrangement

The project site is located in an area of outstanding natural beauty. The visual impact of the dam and restoration of the surrounding area after construction were important factors in promotion of the scheme. Planning permission was obtained in June 2010 with ground investigation, outline design and energy yield studies completed by late 2010. A 'bankable' design was accepted by the project funders in early 2011 and construction commenced almost immediately thereafter.

A construction contract was awarded for the civil works to George McNaugton & Son with Gardner & Theobold as Project Manager and MWH as civil and structural designer. The early involvement with the Client's preferred contractor enabled construction works to commence almost immediately after financial closure. Impounding of the reservoir commenced in May 2012.

HYDROLOGY AND CATCHMENT

The reservoir is located on the southern part of the Isle of Jura and has a catchment area of 9km². The reservoir surface area is 278,000m² at the full supply level of 135.33mOD. The catchment rises up to a maximum of 733mOD at Beinn a Chaolais to the north of the reservoir and is characterised by open peat moorland, mountains and steep scree slopes.

Two further intakes on adjacent catchments and contour ditches increase the effective catchment area of the hydro scheme to over 21.4km².

The main stream passing through the reservoir is the Abhainn a’ Chnuic Bhric, which passes over a set of steep waterfalls just downstream of the dam and into a steeply sided river gorge, before fanning out further downstream at Cnoc Uachdarach.

DAM SITE GEOLOGY

The published 1:50,000 Geological Map of the area indicates that solid geology of the Jura Quartzite Formation, part of the Dalradian Supergroup. Igneous dykes are exposed at surface across Jura and are generally orientated in a northwest to southeast direction. Solid geology comprising Jura Slate, Scarba Conglomerate Formation and Port Ellen Phyllite Formation outcrop along the eastern coastline.

Drift deposits of Quaternary age overlie the solid geology across Jura and comprise Peat, Alluvium and River Terrace Deposits, Till and Hummocky Glacial Deposits. The general bedding dip and direction under the dam according to the geological map is approximately 15° to 25° striking north-northeast.

A ground investigation was carried out by BAM Richies Ltd in late 2010, which included rotary cored boreholes, trial pits, Mackintosh probes, packer permeability tests and laboratory testing. The investigation showed that the dam site was overlain by Peat deposits up to approximately 2.5m depth overlying Glacial Till and Quartzite bedrock. The strata encountered at the dam site are described as follows:

Glacial Till: Generally medium dense to very dense, greyish brown, slightly silty sand and gravel. Frequent cobbles and rare boulders were encountered throughout the material. The thickness of Glacial Till across the dam site ranged between 1.2m to 4.0m and passed down into shattered Quartzite at rock head.

Bedrock: Mainly Quartzite, with occasional Dolerite dykes passing across the site. The Quartzite was weak to very strong, grey angular highly fractured rock with orange/ yellow staining on some joint surfaces. Dolerite was encountered as a very strong, dark grey / black rock with widely spaced discontinuities.

Packer permeability tests were carried out in bedrock in all five boreholes on the dam axis in approximately 5m ascending stages. The Lugeon results generally indicated lower permeability with depth, as expected, since the upper zone near rock head level was highly fractured and shattered in places, presumably due to glacial action. Despite low core recovery and the apparent fractured nature of the bedrock, permeability was low (generally less than 5 Lu), indicating that fractures were tight. Inspection of the cores revealed that joint surfaces were generally fresh with little weathering and staining.

The ground water table in the superficial deposits was encountered just below ground level across the valley.

SELECTION OF DAM TYPE

The range of construction materials available from sources within the site was limited. There was an abundance of Glacial Till (mainly gravels, sands and silt) that could be borrowed from areas close to the dam. There was no cohesive material available at the site and therefore a clay core type dam was not feasible. A concrete dam would have been technically feasible, although not economically viable as the remote site location meant that the cost of producing large quantities of concrete was very high.

The remote location and local climatic conditions were expected to make earthworks construction with the Glacial Till difficult during wet weather and the earthworks season was also expected to be relatively short, being confined to the drier months. Therefore, embankment designs requiring complex zoning or sequential work were viewed less favourably and the owner's implementation programme was also a factor in selection of dam type.

A detailed parametric study for three embankment type dams was carried out as part of the outline design stage and costs estimated to determine the relative cost of each form of construction. These costs were then used as part of the economic analysis for the scheme. The three dam types studied in detail included:

i) Earthfill embankment with 'structural/water-tight' reinforced concrete core wall founded on bedrock;
ii) Earthfill embankment with 'non-structural' concrete core wall founded on bedrock with a waterproof geomembrane lining fixed to the upstream face;
iii) Earthfill embankment with upstream waterproof geomembrane;

A 'structural' reinforced concrete core wall would require substantial concrete section and reinforcement to meet water retaining design codes and compaction loads. Therefore, a lighter 'non-structural' concrete section was also considered with a waterproof geomembrane attached to the upstream face.

Unit rates were obtained from the prospective contractor and used to generate a cost model for incremental storage volumes for each of the three dam types shown above. The cost curves for the three options are shown on

Figure 2, normalised against the cost of the geomembrane option were used in the economic analysis for the scheme.

The energy yield analysis indicated that a 3m raising of the current lochan would provide sufficient storage to meet generation requirements. From Figure 2, the relative cost difference between an upstream geomembrane faced embankment and a non-structural concrete core / geomembrane embankment was marginal. However, the concrete core / membrane would require longer construction period and cause disruption to the earthworks operations. Therefore, based on a number of factors, including cost, schedule and buildability, the final dam type incorporating an earthfill embankment with an upstream geomembrane was selected.

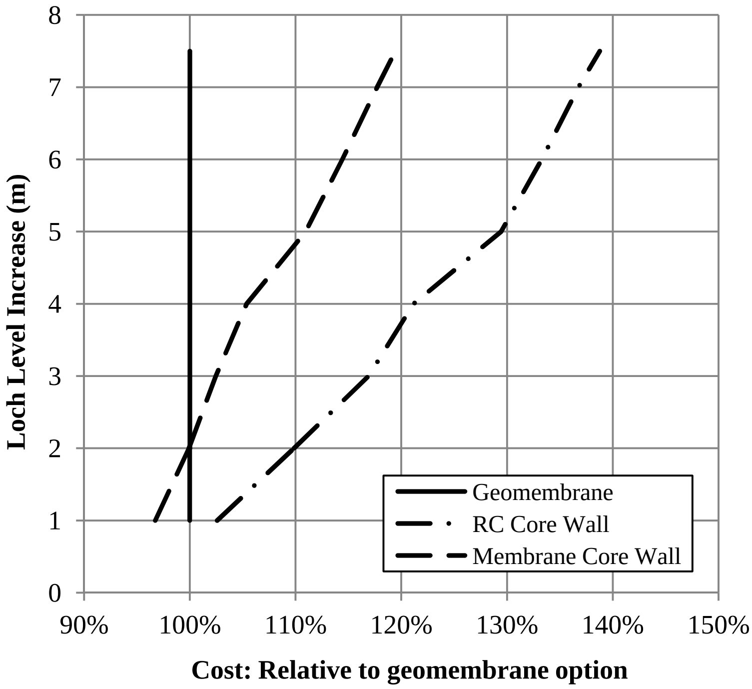


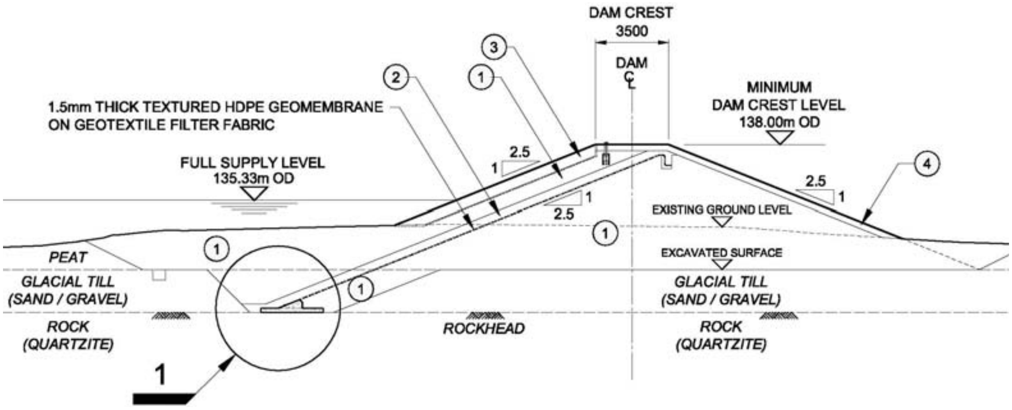
Figure 2. Cost Curves for Dam and Ancillary Works

EMBANKMENT DAM DESIGN

The embankment has a crest width of 3.5m set at a level of 138.0mOD with upstream and downstream slopes at 1 on 2.5. Landscaping fill was placed mainly on the right low flank to reduce the visual impact of the embankment. The waterproof element was formed by an upstream sloping geomembrane that was covered over with graded granular fill and rip rap. A typical cross section of the dam is shown on Figure 3.

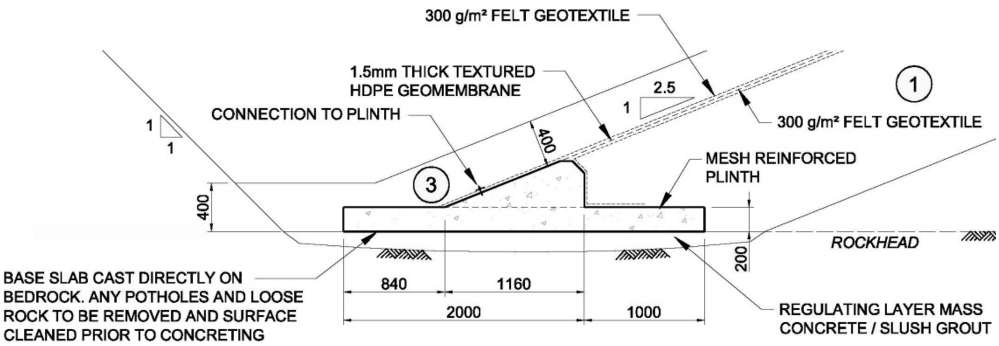
All peat and organic material was excavated from the footprint of the dam and temporarily disposed of at a nearby location by end tipping prior to restoration and landscaping works. The peat thickness varied across the dam site to a maximum of about 2.5m with temporary cut slopes in peat generally stable throughout the construction period at about 1 on 1. No special de-watering works were required other than surface water discharge and filtration through straw bales/geotextile before returning back to the burn. The Glacial Till formation was trimmed and proof rolled prior to filling works.

The geomembrane was connected at the upstream toe to a concrete toe plinth cast on a prepared rock surface with a sloping concrete wedge (similar to a CFRD toe plinth) to provide a planar watertight surface from which to connect the geomembrane. The geomembrane was secured at the crest in an anchor trench.



(1) General Fill (Glacial Till) (2) Selected Fill (3) Rip rap (4) Peat/Till

(a) Typical Cross Section



(b) Upstream Toe Plinth Detail

Figure 3. Typical Cross Section and Plinth Detail

A 3m wide toe plinth was cast directly onto a prepared rock surface with the height of the concrete wedge limited to 0.4m to allow compaction of the embankment material immediately behind the wedge to reduce the risk of differential settlement and damage to the geomembrane. All concrete edges were chamfered to eliminate any sharp edges in contact with the geomembrane.

A concrete cut-off trench was discounted due to the perceived difficulty in forming a trench in the Quartzite formation without causing extensive disturbance of the upper rock zone, which might result in increased leakage losses, preferential leakage paths and a significant increase in concrete quantities.

The upstream toe plinth trench was advanced by excavator through the Glacial Till to expose sound rock head. Final clean-up was carried out by high pressure water/air jets, brushing and hand picking of loose debris. The micro-fissured nature of the uppermost part of the foundation meant that grouting would have stood little chance of success even using very low pressures and closely spaced holes. The need to reduce the permeability was questionable as the future water head is very modest and the hydraulic gradients small. Furthermore, a 2D seepage analysis showed that little benefit would be gained from grouting the foundation, even if it was possible.

The rock surface was geologically mapped and then either slush grouted or built up with mass concrete to regulate the irregular rock profile to facilitate construction of the toe plinth. In the event, following foundation clean-up, the bedrock longitudinal profile produced a series of strong ridges with completely shattered quartzite gravel in between, which resulted in additional mass concrete required to regulate the rock profile. The plinth foundation was classified, by fracture spacing, into one of three grades on site for record purposes.

Grade I Rock surface generally intact, definable joint structure with fewer than 5 fractures per metre in each direction;

Grade II Rock surface generally intact, definable joint structure with more than 5 fractures per metre in each direction;

Grade III Rock surface highly fractured with no definable joint structure.

The composition of the toe plinth foundation included 31% (Grade I), 54% (Grade II) and 15% (Grade III). In general, the Dolerite proved relatively straight forward to clean up since the orthogonal bedding planes provided regular planes of weakness that could be readily trimmed with a 3T excavator. The Quartzite formations proved more difficult to clean up, especially in the valley bottom, where a fault zone was exposed, requiring more jetting and hand cleaning.

An extensive search for suitable embankment materials was carried out as part of the ground investigation. The southern side of the lochan identified suitable granular materials (Glacial Till), although it was perceived at design stage that these would be difficult to win, due to the limited zone between the existing lochan and the future full supply level. The northern shoreline comprised wet silts and sands, which were deemed unsuitable. Therefore efforts were concentrated on an extension to the existing access track to the north of the dam site, where Quartzite outcrops were visible. This borrow area would yield sufficient quantities of Glacial Till (silty sandy gravel) although the silt fraction was up to 30%. Grading, classification and shear box tests (re-compacted at 9% maximum dry density) were carried out on these materials which yielded design parameters of $\phi' = 35°$, $c' = 0$ and $\gamma = 1.8 Mg/m^3$ for the Glacial Till.

Compaction trials were carried out early in the construction stage and the Glacial Till was proved to be a satisfactory material, although the high fines content rendered the material unworkable in wet weather. Initial fill placement with this material provided satisfactory results, although as delays in completion of the toe plinth and appurtenant works mounted during late autumn 2011, it became clear that this source of material could not be worked over the winter months. Therefore, an alternative borrow pit was opened up close to the left abutment of the dam, which yielded shattered Quartzite that could be readily obtained by ripping and excavation. This material was effectively well-graded, fine to medium grained rockfill and could be placed and compacted in almost all weather conditions to produce an excellent dense fill (average D_{100} = 100mm; D_{50} = 10mm and non-plastic fines <8%). Compaction was achieved with 6 No. passes of a 10T smooth drum vibrating compactor on 0.4m thick layers.

Stability analysis carried out for the embankment was based on limit equilibrium methods using effective stress parameters. Long term steady state pore pressures were estimated by 2D finite element seepage analysis. Construction stage pore pressures were not considered to be significant, although pessimistic assumptions were used in the analysis.

The stability analysis followed normal practice with target factors of safety of 1.5, 1.3, 1.2 and 1.1 for normal operation, end of construction, rapid draw down and seismic analysis, respectively. Circular slip circles were derived using Slope/W for deep seated slips in the upstream / downstream shoulders. Non-circular slip surfaces were also investigated for the upstream slope due to the presence of the buried geomembrane, which could create a preferential surface for sliding.

The HDPE geomembrane was textured on both sides to increase its frictional resistance and a heavy ‘felt’ type geotextile was placed below and above the geomembrane to protect against puncture / damage during backfilling. Design parameters for the geomembrane were assigned based on a paper by Vaid & Rinne (1995), with design parameters of $\phi' = 20°$, $c' = 0$ and $\gamma = 0.94 Mg/m^3$ used in the analysis. The critical design case was rapid drawdown of the reservoir. However, since the fill material above the geomembrane was permeable gravel fill and the rip rap is very permeable, excess pore pressures during rapid drawdown are not expected to be significant.

GEOMEMBRANE DETAILS

A 1.5mm thick double textured HDPE geomembrane liner was chosen for the waterproof element and installed by specialist contractor Landline Ltd. The total coverage area was approximately 5,000m² with mechanical seals at the upstream toe plinth and to all the appurtenant structures. The toe plinth connection detail is shown on Figure 4. Special details were required to fix the geomembrane to the spillway and intake works to accommodate differential settlement of the embankment fill and also rigid structures founded on bedrock. Anchorage at the crest was provided by lapping the geomembrane into an anchor trench.

The upstream embankment face was prepared by trimming and compacting the slope with a heavy excavator-mounted vibrating plate. This produced a regular surface, free from pot holes and angular quartzite fragments. The surface was locally blinded with sand where any local irregularities had formed and a 300g/m² 'felt' type geotextile was placed on the surface and the geomembrane rolled out and fixed at the toe and crest. Temporary sand bags were used to weigh the geomembrane down until it was securely fastened and welded.

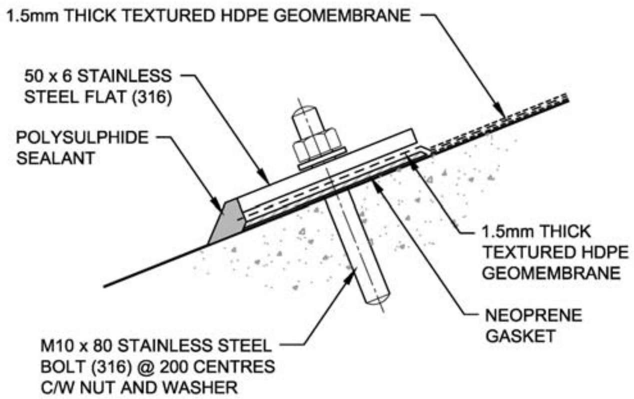


Figure 4. Geomembrane / Toe Plinth Connection

A further geotextile was placed on top of the completed geomembrane to provide protection from the transition fill placed above. The transition fill consisted of selected Glacial Till (Average D_{100}<40mm; D_{50}=8mm and D_{10}=0.1mm) to provide a free draining buffer between the geomembrane and the general fill. The geomembrane was buried to provide additional protection against the effects of sunlight and temperature stresses. Rip rap was abundant close to the dam site and was used as wave protection to the upstream face. The principal properties for the geomembrane are summarized in Table 1.

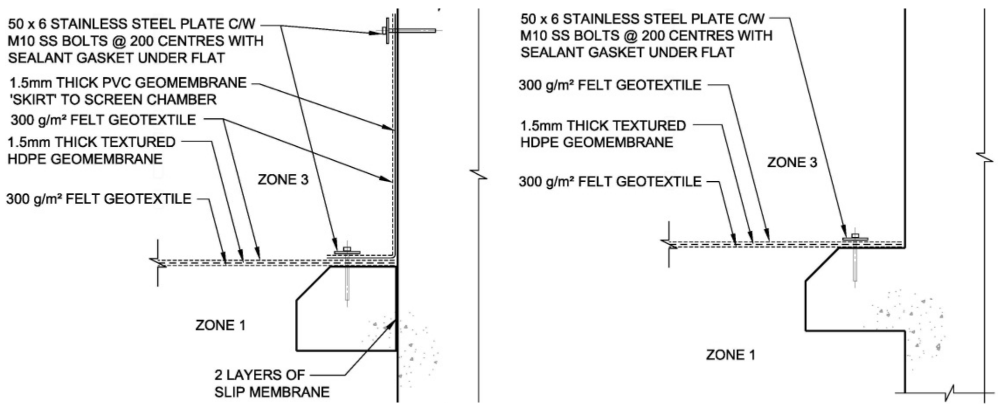
Table 1. Geomembrane Properties

Description	**Parameter**	**Test Method**
Thickness	1.5mm	ASTM D5199
Tensile strength @ break	49N/mm	ASTM D638
Elongation @ break	800%	ASTM D638
Density	940kg/m³	ASTM D1505
Puncture resistance	560N	ASTM D4833

The geomembrane was supplied in 7.5 m wide rolls and all site welds were carried out using an automatic wide wedge welding machine which formed double seam welds that were all air tested to 2 bar pressure.

Before installing the geomembrane, a site trial was carried out to demonstrate that the contractor's backfill and compaction methods would have no detrimental effects on the integrity of the geomembrane. A section of geomembrane was backfilled and later exhumed to assess the condition of the geomembrane. The trial showed no signs of puncture or distress and the contractor's method of working was accepted. The geomembrane was installed in two visits to suit the contractor's river diversion sequence.

The installation and welding was extremely quick due to the relatively short lengths of membrane, although work was temporarily suspended during high winds and heavy rain on safety grounds.



(a) Connection with Intake Shaft (b) Connection with Spillway

Figure 5. Intake/Spillway Connection Details

As the geomembrane was to be connected to the intake shaft, which was founded on bedrock, a slip membrane was used between the structure and the concrete connection nib, with a PVC 'skirt' connection between the nib and the shaft to provide additional elongation against any differential settlement of the rockfill. This detail was not required on the spillway, since the spillway structure would settle in concert with the fill material. The connection details for the intake shaft and spillway are shown in Figure 5.

SPILLWAY DESIGN

The spillway was designed to cater for the routed PMF discharge of 85m³/s. Alternative spillway configurations that best suited the site topography were studied, including side channel/chute spillway, auxiliary spillways, morning glory spillway and an articulated spillway over the central section of the embankment. Ordinarily, spillways located remote from the embankment would be preferred, although in this case, there was a clear economic advantage for the 'over the top' spillway and was deemed suitable at this low rockfill embankment.

Overflow weirs of different length were considered and the effect on the freeboard required to contain the design flood and wave surcharge were evaluated. Since the cost of concrete works was high and earthworks / geomembrane were relatively less expensive, it was decided to proceed with a 15m long overflow weir, chute and USBR Type III stilling basin design. This produced a gross freeboard requirement of 2.67m, which extended the length of the embankment across the relatively flat right abutment.

In order to mitigate the problems of fill settlement under the structure, all the superficial materials were removed from the footprint of the spillway down to bedrock and re-filled with compacted rockfill.

The stilling basin was formed on rock and expansion joints were provided in the structure at the stilling basin and crest to provide articulation. A cross section through the spillway structure is shown on Figure 6.

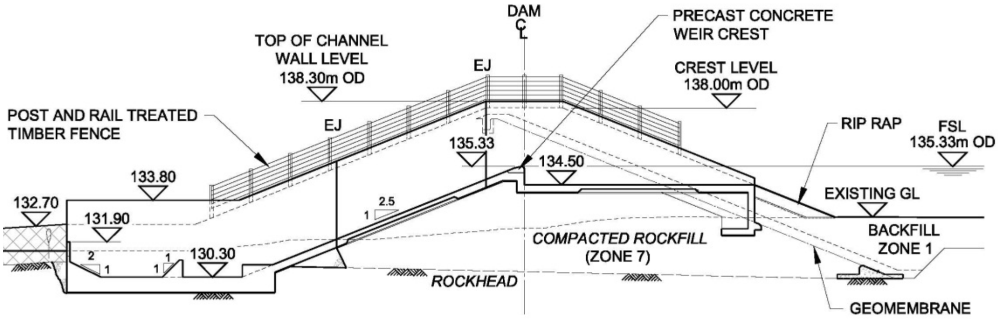


Figure 6. Cross Section through Spillway

INTAKE WORKS

The power intake structure was formed by a square shaped shaft within the dam embankment located just to the left of the spillway structure. The shaft housed a 10mm bar screen, power intake and floating arm compensation inlet. Water enters the shaft via a 2.6m square box culvert leading to the reservoir, with penstock gates providing isolation to the shaft and stop log channels provided upstream of the bulkhead gate. The inlet culvert was sized to provide an approach velocity to the screen of 0.3m/s to mitigate the risk of fish becoming trapped in the approach channel to satisfy environmental requirements.

The 1.2m diameter polyethylene power penstock was founded within a rock trench and encased in concrete as it passed under the downstream shoulder of the embankment to provide dual containment. The compensation main was also carried in the same trench and concrete surrounded. A ‘pigging’ facility was provided for the pipeline by means of a 45° vertical steel Y-branch located on the penstock pipeline just downstream of the dam, from where a ‘soft pig’ could be launched for periodic cleaning of the inside of the penstock to improve efficiency. A branch off the pig launcher also provided an outlet for Freshet environmental releases and emergency drawdown at the dam.

SUMMARY

Gleann Astaile Reservoir is one of only a handful of new impounding reservoirs built in the UK in recent years. The scheme was constructed in a relatively short period under arduous conditions on a remote island.

The choice of dam type was influenced by the site geology, cost and programme. After detailed parametric studies for various dam types, an earthfill embankment with an upstream geomembrane offered an economic and practical solution which also minimized environmental and visual impact in the sensitive location.

The foundation preparation for the upstream toe plinth was relatively straightforward in the Dolerite dykes that passed across the site, but more difficult in the shattered Quartzite formation. Slush concrete and mass concrete was used to regulate the prepared rock surfaces prior to construction of the toe plinth, which provided a planar surface from which to connect the geomembrane. Various mechanical connection details were used to connect the geomembrane to the toe plinth, spillway and intake structure.

Figure 7.   Photograph of Upstream Toe Plinth and Geomembrane

ACKNOWLEDGEMENTS
The authors would like to thank Inver LLP for permission to publish this paper and in particular, John Lithgow. Others involved include Ed Smith of Gardner & Theobold, George McNaughton & Sons and colleagues at MWH.

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