Due to its small size the catchment descriptors for the burn were not available from the FEH web-service. To undertake the hydrological analysis the catchment descriptors for the closest catchment, a larger catchment including the catchment of the burn, was obtained. This catchment has an area of 0.6 km². Catchment descriptors are shown in Table 9.

Given the size of the catchment the design flows were estimated based on a selection of methods: the FEH Rainfall-Runoff method, the Institute of Hydrology (IH) Small Catchment Equation (IH124), and the Revitalised FEH Rainfall-Runoff method (ReFH2) for the "Summer" rainfall profile. The "Summer" profile was chosen over the "Winter" profile due to the steep catchment gradient.

Design flows were estimated for the IH124 method using the catchment AREA of 0.21 km², a SAAR of 1638 and a SOIL type of 5 (0.5).

Due to the unavailability of the catchment descriptors for the exact catchment of the burn design flows for the FEH Rainfall-Runoff and ReFH2 methods were estimated using two approaches. The first by adjusting the catchment descriptors of the larger catchment to better represent the smaller, burn catchment. The adjustments were undertaken in line with recommendations in the Flood Estimation Handbook. A second approach involved estimating the design flows for the larger catchment (0.6 km²) and then scaling them down based on catchment area. The results are shown in Table 10.

Burn 1 benefitted from the most detailed hydrological analysis due to the requirement for flows to guide the design of the proposed culvert.

**Table 9: Catchment descriptors from the FEH web-service (0.6km²) and for Burn 1 (0.21km²)**

| Parameter | FEH – Web-service 0.6km² | Burn 1 0.21km² |
|---|---|---|
| **EASTING (m)** | 260800 | 260654 |
| **NORTHING (m)** | 260169 | 580442 |
| **AREA (km²)** | 0.6 | 0.21 |
| **ALTBAR (m)** | 121 | 121 |
| **ASPBAR (°)** | 36 | 40 |
| **ASPVAR** | 0.76 | 0.76 |
| **BFIHOST** | 0.53 | 0.53 |
| **DPLBAR (km)** | 0.9 | 0.65 |
| **DPSBAR (m/km)** | 180.7 | 172 |
| **FARL** | 1 | 1 |
| **LDP** | 1.83 | 1.2 |
| **PROPWET** | 0.65 | 0.65 |
| **SAAR (mm)** | 1638 | 1638 |
| **SAAR4170 (mm)** | 1531 | 1531 |
| **SPRHOST** | 42.5 | 42.5 |
| **URBCONC1990** | -999999 | -999999 |
| **URBEXT1990** | 0 | 0 |
| **URBLOC1990** | -999999 | -999999 |
| **URBCONC2000** | 0.333 | 0.333 |
| **URBEXT2000** | 0.0125 | 0.0125 |
| **URBLOC2000** | 0.299 | 0.299 |





**Table 10: Design flows for Burn 1 (Return Period - $m^3/s$)**

| | Estimation Method | 1 in 200 | 1 in 500 | 1 in 1000 | 1 in 200 + 30% CC | 1 in 200 + 55% CC[1] | 1 in 1000 + 30% CC |
|---|---|---|---|---|---|---|---|
| 0.21 km² Catchment | FEH Rainfall-Runoff [a] | 1.4 | 1.7 | 2.0 | 1.8 | 2.3 | 2.6 |
| | ReFH2 Summer [b] | 0.65 | 0.7 | 0.8 | 0.9 | 1.0 | 1.1 |
| | IH124 | 1 | 1.1 | 1.3 | 1.3 | 1.6 | 1.7 |
| | | | | | | | |
| 0.6 km² Catchment Scaled | FEH Rainfall-Runoff [a] | 1.2 | 1.5 | 1.7 | 1.6 | 2.0 | 2.2 |
| | ReFH2 Summer [b] | 0.6 | 0.8 | 1.0 | 0.7 | 1.1 | 1.3 |
| | | | | | | | |
| | Previous estimate | 0.78 | 0.85 | 0.94 | 1.01 | NA | 1.22 |

*a Critical Storm Duration = 2.4 hours*
*b Critical Storm Duration = 2.45 hours*
*1 Uplift of 55% rainfall intensity for Rainfall-Runoff methods.*

It should be noted that the climate change estimates provided in this report make a 30% allowance for climate change in line with the previous Halcrow (2011) FRA.

The estimated design flows for the 1 in 200-year event vary between 0.6 $m^3/s$ and 1.4 $m^3/s$. The estimations made by Halcrow in their assessment are also provided for comparison purposes. The most conservative design flow was calculated using the FEH Rainfall-Runoff method, using the adjusted catchment descriptors.

A comparison of the design flows estimated here and as part of the previous Halcrow assessment suggests that the previous flows are generally in line with the estimates in this report. However, the previous assessment used a smaller catchment area in the calculation and the older IH124 method. The newer rainfall-runoff methods generally estimate higher flows than those estimated using the older IH124 method.

## 4.6 Estimation of Design Flows for Burn 2

Due to its small size the catchment descriptors for the burn were not available from the FEH web-service. To undertake the hydrological analysis the catchment descriptors for the closest catchment, a larger catchment was obtained. This catchment has an area of 0.6 km². Details of this catchment area provided in Section 4.1.5. The catchment descriptors for the adjusted catchment are shown in Table 11.

Given the size of the catchment the design flows were estimated based on a selection of methods: the FEH Rainfall-Runoff method, the Institute of Hydrology (IH) Small Catchment Equation (IH124), and the Revitalised FEH Rainfall-Runoff method (ReFH2) for the "Summer" rainfall profile. The "Summer" profile was chosen over the "Winter" profile due to the steep catchment gradient.

**Table 11: Catchment characteristics for Burn 2**

| Parameter | Value |
|---|---|
| EASTING (m) | 260654 |
| NORTHING (m) | 580442 |
| AREA (km²) | 0.44 |
| ALTBAR (m) | 121 |
| ASPBAR (°) | 40 |
| ASPVAR | 0.76 |
| BFIHOST | 0.53 |
| DPLBAR (km) | 0.65 |
| DPSBAR (m/km) | 172 |
| FARL | 1 |
| LDP | 1.2 |
| PROPWET | 0.65 |
| SAAR (mm) | 1638 |
| SAAR4170 (mm) | 1531 |
| SPRHOST | 42.5 |
| URBCONC1990 | -999999 |
| URBEXT1990 | 0 |
| URBLOC1990 | -999999 |
| URBCONC2000 | 0.333 |
| URBEXT2000 | 0.0125 |
| URBLOC2000 | 0.299 |

The estimated flows from the three methods are tabulated in Table 12.

**Table 12: Design flows for Burn 2 (Return Period - $m^3/s$)**

| Estimation Method | 1 in 200 | 1 in 500 | 1 in 1000 | 1 in 200 + 30% CC | 1 in 200 + 55% CC[1] | 1 in 1000 + 30% CC |
|---|---|---|---|---|---|---|
| FEH Rainfall-Runoff 2013 Rainfall [a] | 2.6 | 3.2 | 3.7 | 3.4 | 4.4 | 4.9 |
| ReFH2 [b] | 1.1 | 1.5 | 1.8 | 1.5 | 1.9 | 2.3 |
| IH124 | 2.2 | 2.4 | 2.8 | 2.9 | 3.4 | 3.6 |
| | | | | | | |
| Previous estimate | - | - | - | - | - | - |

*a Design Storm Duration = 2.5 hours*
*b Design Storm Duration = 2.45 hours*
*1 Uplift of 55% rainfall intensity for Rainfall-Runoff methods.*

The estimated design flows for the 1 in 200-year event vary between 1.1 $m^3/s$ and 2.6 $m^3/s$. In their previous Flood Risk Assessment Halcrow did not provide design flows for this watercourse.

The most conservative design flow was calculated using the FEH Rainfall-Runoff method. For this reason, this flow estimate was used in this assessment. The most conservative flows are shown in red.





## 4.7 Estimation of Design Flows for the Tailrace

The Tailrace is not a natural watercourse with flows conveyed through the structure controlled by the Glenlee Power Station. The previous Flood Risk Assessment reported that Scottish Power had confirmed that the Glenlee Power Station passes a maximum flow of 25.5 $m^3/s$, with normal operating peak flows averaging between 21 $m^3/s$ and 22 $m^3/s$.

The maximum flow of 25.5 $m^3/s$ has been used as part of this assessment.

# 5 Flood Risk Assessment

The flood risk assessment considers the risk from:

- Large Watercourses (Water of Ken, Coom Burn, Tailrace, Park Burn)
- Dickson's Strand and Burn 2
- Burn 1
- Surface water flooding;
- Groundwater flooding; and
- Infrastructure.

## 5.1 Large Watercourses (Water of Ken, Coom Burn, Tailrace, Park Burn)

A previous Flood Risk Assessment was undertaken by Halcrow to support the development of flood mitigation measures for the Glenlee Power Station. This report included a 1D ISIS (Flood Modeller Pro) mathematical model to represent the Water of Ken, Coom Burn and Tailrace.

To confirm the general accuracy of these results a 2D mathematical model was developed to represent the Water of Ken, Coom Burn and Tailrace in addition to the Park Burn. This model was based on high-resolution LiDAR DTM data. The model extents were set over 800m upstream and 1.5km downstream of the site.

The model was run using a 5m grid resolution for a steady 1 in 200-year flood event using the design flows assessed in Section 4. The model was run using global Manning's *n* roughness values of 0.08 to represent a mixture of terrain roughness. The downstream model boundary was set as a "normal depth" boundary with a shallow gradient of 1 in 400. All structures such as bridges were removed from the LiDAR DTM data.

The results of this assessment produce similar flood extents to those published in the previous Flood Risk Assessment. Additionally, the predicted peak water levels adjacent to the site are approximately 53.7m AOD. This is within the margin of error compared to the previous assessment which estimated peak water levels of between 53.76m AOD and 53.79m AOD adjacent to the site. It should be noted that a 1D mathematical model better represents the conveyance capacity within watercourses and could be considered more accurate in this instance.

Taking in consideration the results of this comparison it was considered acceptable to use the results from the previous Flood Risk Assessment to support this assessment.

The peak water levels within the vicinity of the site estimated in the previous assessment are provided in Table 13.





**Table 13: Peak water levels in the vicinity of the site (Halcrow, 2011)**

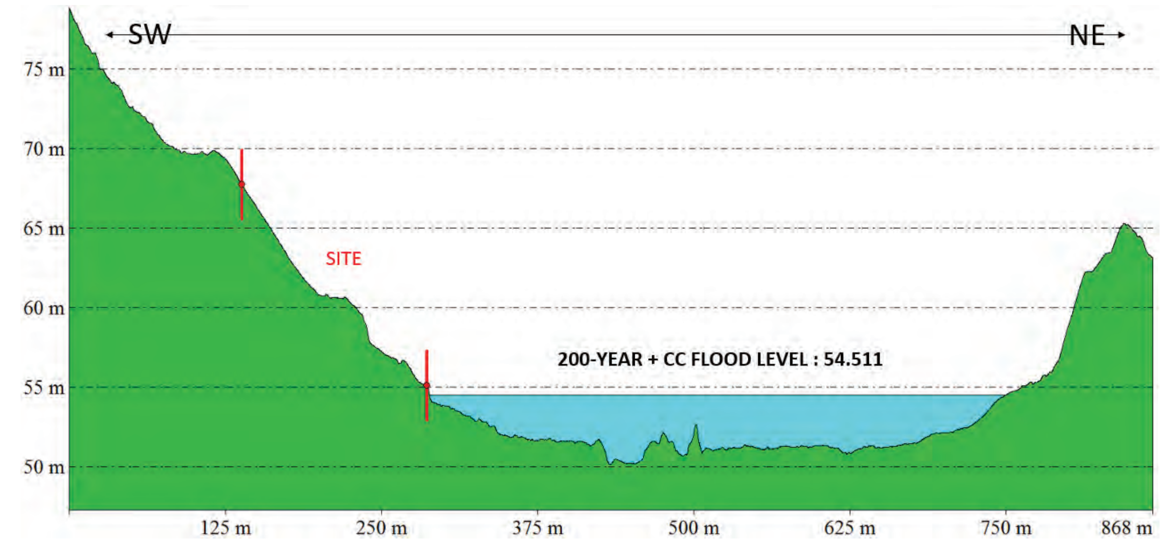
| Cross Section | Location | 1 in 200 | 1 in 500 | 1 in 1000 | 1 in 200 + 30% CC | 1 in 200 + 55% CC | 1 in 1000 + 30% CC |
|---|---|---|---|---|---|---|---|
| | | | | | | | |
| CoomB1 | Power Station | 53.786 | 54.111 | 54.377 | 54.504 | NA | 55.195 |
| CoomB2 | Substation Site | 53.785 | 54.114 | 54.382 | 54.509 | NA | 55.203 |
| CoomB3 | D/S of Substation | 53.760 | 54.089 | 54.358 | 54.486 | NA | 55.181 |
| | | | | | | | |
| TL1 | Power Station | 53.791 | 54.117 | 54.384 | 54.511 | NA | 55.203 |
| TL2 | Substation Site | 53.775 | 54.103 | 54.371 | 54.499 | NA | 55.193 |
| TL3 | D/S of Substation | 53.768 | 54.098 | 54.367 | 54.495 | NA | 55.190 |

Comparing the ground levels of the site with those provided in Table 13, the site is predominantly elevated above the peak modelled flood levels. For example, the vast majority of the site lies above the 1 in 200-year, 1 in 500-year, 1 in 1000-year and 1 in 200-year plus climate change peak water levels. This is demonstrated in Figure 13 which shows the highest water level of these events (54.511m AOD) against the levels of the site. Small areas of the site fronting the local road would be at risk of flooding.

In the event of a 1 in 1000-year plus climate change flood event additional flooding of the site that fronts the local road would also occur.

Halcrow did not undertake model runs to represent the 1 in 200-year + 44% climate change event, as this was not a requirement at the time. Peak flows for this event are lower than the 1 in 1000-year + 30% climate change event for all watercourses (see Section 4) meaning flooding and peak water levels would be expected to be less severe and lower than the 1 in 1000-year + 30% climate change event.

**Figure 13: Topography of the site and surrounding area within 200-year + CC flood level**



However, following discussions with Scottish Power it has been confirmed that the Glenlee Power Station and Substation are now protected against flooding by flood defences up to the 1 in 1000-year plus climate change flood levels estimated in the previous Flood Risk Assessment by Halcrow (Email from SPEN, 26 October 2017).

Considering the site is predominantly located further to the south, at a higher elevation than the existing development it is considered that these flood defences would continue to protect the proposed development so long as they are maintained as part of the new development.

The site is therefore considered to be at low risk of flooding from the Water of Ken, Coom Burn, Tailrace and Park Burn up to the 1 in 1000-year plus 30% climate change flood event.

## 5.2 Dickson's Strand & Burn 2

The flood risk to the site from the Dickson's Strand and Burn 2 was assessed by developing a 2D mathematical model of the watercourses.

This model was based on high-resolution LiDAR DTM data. The model extents were set over 600m upstream and 400m downstream of the site.

The model was run using a 3m grid resolution for unsteady, 1 in 1000-year plus 30% climate change flood events using the design flows assessed in Section 4. This is the most conservative event meaning that flooding would be anticipated to be less severe for the lesser events. The model was run using global Manning's *n* roughness values of 0.08 to represent a mixture of terrain roughness. All structures such as bridges were removed from the LiDAR DTM data.

Two scenarios were modelled:

1. 1 in 1000-year plus 30% climate change flood event with no flows within the Water of Ken, Coom Burn or Tailrace to represent a short duration event where flooding of the larger Water of Ken, Coom Burn and Tailrace had not yet occurred;
2. 1 in 1000-year plus 30% climate change flood event with a water level boundary of 55.2m AOD to represent a 1 in 1000-year plus climate change event in the Water of Ken, Coom Burn and Tailrace.

The results of this assessment show that in the event of the short duration event the site would not be impacted by flooding from the Dickson's Strand and Burn 2. In a flood event where the Water of Ken, Coom Burn and Tailrace were already flooded up to a 1 in 1000-year flood event, flooding from the Dickson's Strand and Burn 2 would still not impact on the site with the limited flooding of the site along the local access road being caused by the high water levels in the Water of Ken, Coom Burn and Tailrace and not the two smaller watercourses. Flood extent maps are shown in Figures 14 and 15.

It should be emphasised that the existing Glenlee Power Station has been protected up to the 1 in 1000-year plus climate change event through the implementation of flood mitigation measures.

The site is therefore considered to be at low risk of flooding from Dickson's Strand and Burn 2.

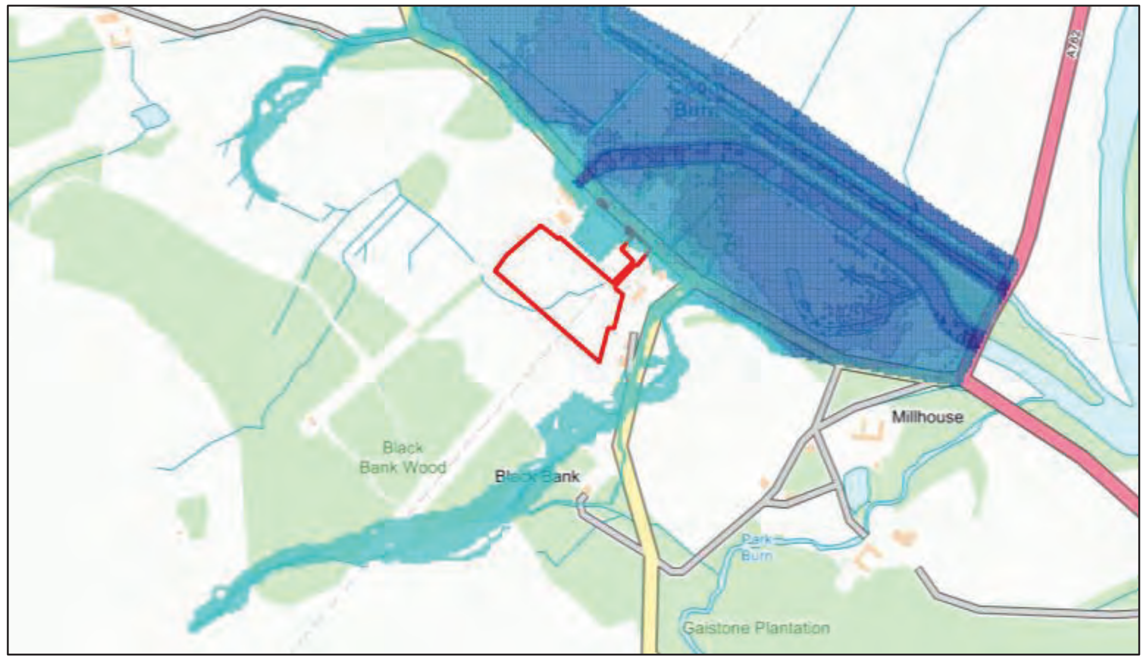




**Figure 14: 1000-year plus 30% climate change flood extent – Short duration**




**Figure 15: 1000-year plus 30% climate change flood extent – with flooding of Water of Ken, Coom Burn and Tailrace**



## 5.3 Burn 1

Burn 1 is an unnamed watercourse that drains from Glenlee Hill and enters the site from under the Penstock traversing the site in a north-easterly direction. The watercourse is culverted under the existing Glenlee substation.

### 5.3.1 Existing Culvert

Mathematical modelling undertaken by SPEN indicates that the existing culvert under the substation cannot convey the 1 in 200-year flow without flooding (SPEN, 2017).

A HEC-RAS 1D mathematical model was developed to provide further information on the culvert. Existing site plans and the topographic survey were used to represent Burn 1, just upstream of the culvert and the culvert itself. The parts of the culvert and how they were represented in the model are shown in Table 14.

**Table 14: Model details**

| Cross Section | Description | Diameter | Gradient |
|---|---|---|---|
| **90 - 84** | Head of the model – 2x cross-section to represent Burn 1 just upstream of the culvert | - | - |
| **83 CULV** | Steep section of culvert as it passes through the substation. 450mm Diameter | 0.45m | 1 in 10 |
| **34-33** | Manhole downstream of steep section of culvert | | |
| **31 CULV** | Flat section of culvert of 375mm Diameter. | 0.375m | 1 in 150 |
| **15-14** | Manhole downstream of flat section | | |
| **13 MO** | Culvert section under road with 3x culverts (multiple opening) | 0.3m x3 | 1 in 25 |
| **1-0** | Open section of Burn 1 downstream of site | - | - |

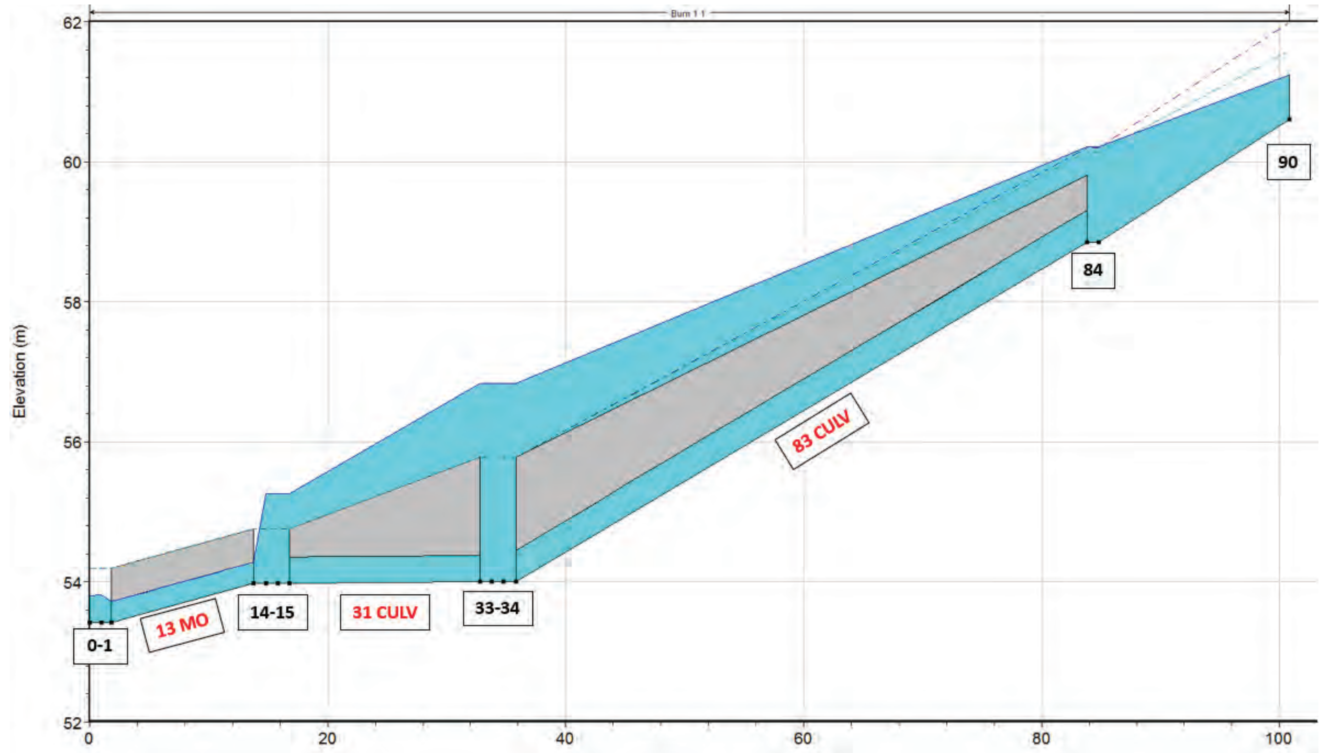
The modelled reach has a total length of approximately 100m and represents the culvert within the site and Burn 1 just upstream and downstream of the site. Manning's *n* roughness values were set to 0.02 for the culvert sections and manholes and 0.045 for the channel and overbank areas of the burn. Unsteady flows were input into the model using the estimated FEH Rainfall-Runoff hydrograph. The downstream model boundary was set as a "stage level" boundary with a level of 53.8m AOD to represent a 1 in 200-year flood level within the Tailrace (and other watercourses) downstream.

The model profile results for the 200-year event and the location of the cross-sections described in Table 14 are shown in Figure 16. Culverts are shown in red text.





**Figure 16: Model profile showing cross-sections and the 1 in 200-year water level**

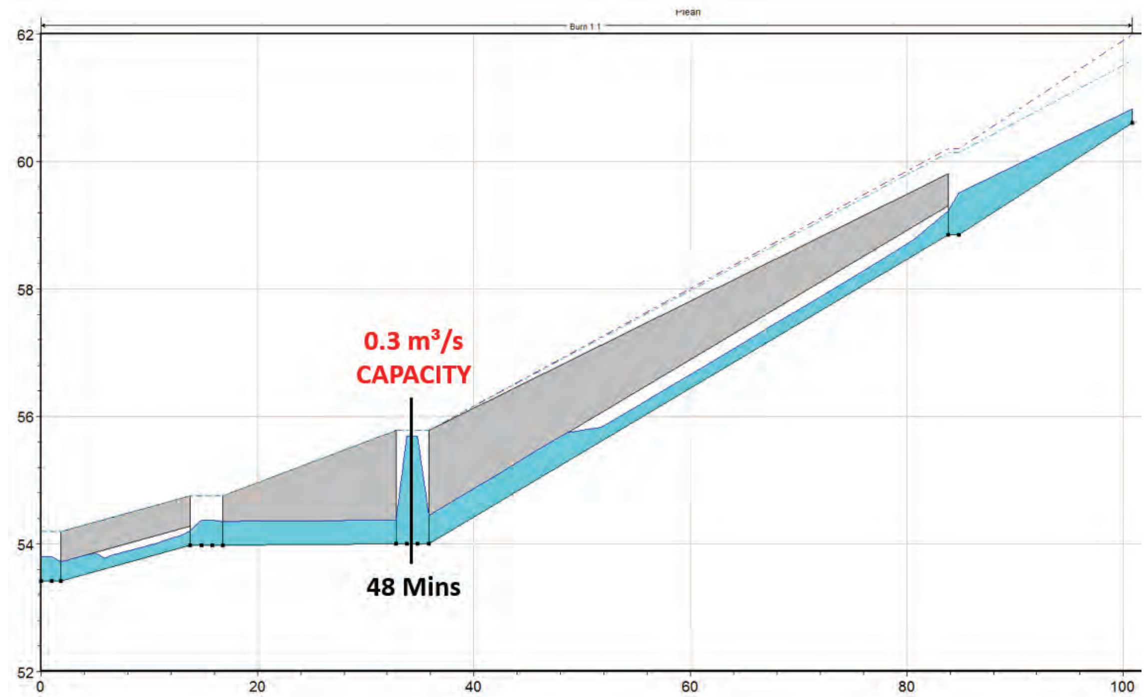


The results of the modelling suggest that the existing culvert cannot convey the 1 in 200-year flow of approximately 1.4 m$^3$/s.

Examining the model results throughout the flood event help provide additional information on when and where flooding may occur.

At approximately 48 minutes into the 4-hour 200-year flood event 31 CULV (flat section), begins to surcharge with flows backing up into the upstream manhole represented by cross-sections 33 and 34. Flows passing through the culvert at around this time are approximately 0.3 m$^3$/s suggesting that this is the maximum capacity of this section of the culvert. The manhole surcharges and flows overtop shortly after this time. The ability of the whole culvert to convey flows is limited by this section of the culvert, which surcharges first. This is shown in Figure 17.

**Figure 17: Model run at 48 minutes into the run showing 31 CULV surcharging**




At approximately 60 minutes into the 4-hour 200-year flood event 83 CULV, begins to surcharge with flows backing up just upstream of the culvert inlet. Flows passing through the culvert at around this time are approximately 0.5 m$^3$/s suggesting that this is the maximum capacity of this section of the culvert. Flows overtop the culvert inlet shortly after this time and flow into the existing substation. This is shown in Figure 15.

Allowing for the capacity of these sections of the culvert, and the peak flow of 1.4 m$^3$/s it can be estimated than in a 1 in 200-year event 0.9 m$^3$/s of flow would overtop the culvert at the inlet and flow through the existing substation with an additional 0.2 m$^3$/s of flow surcharging at 31 CULV and the manhole represented at cross-sections 33 and 34. This combines to a total overland flow of approximately 1.1 m$^3$/s in a 1 in 200-year event, with the remaining 0.3 m$^3$/s being conveyed by the culvert. This is represented in Figure 19.





**Figure 18: Model run at 60 minutes into the run showing 83 CULV surcharging**

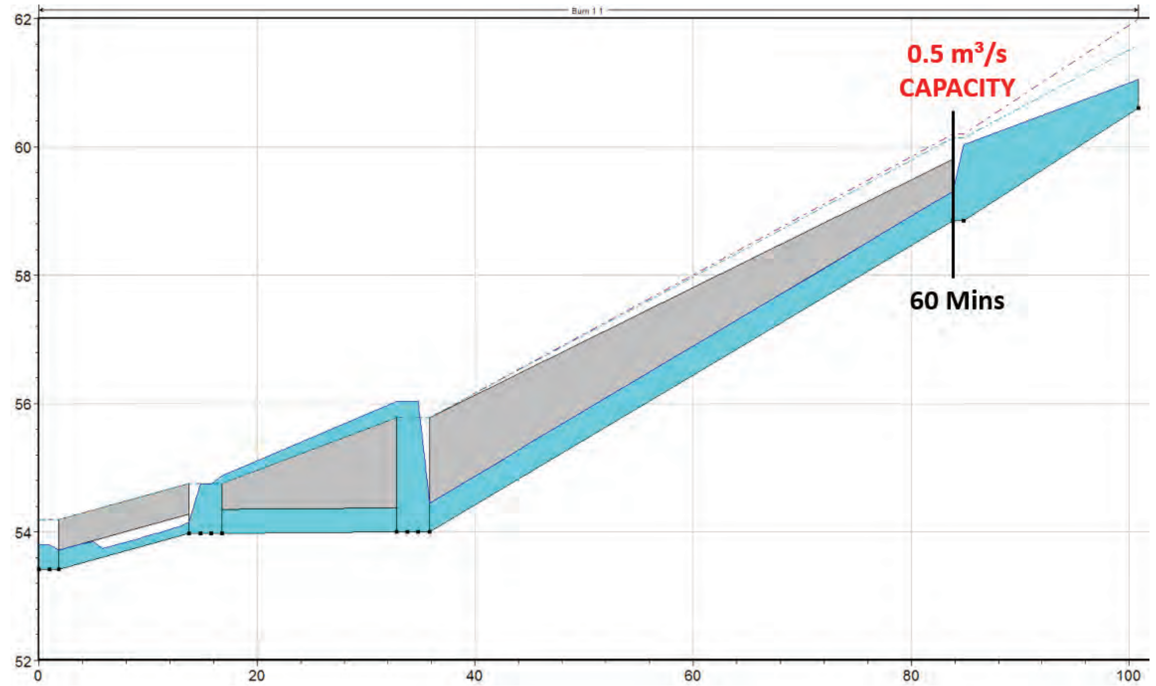
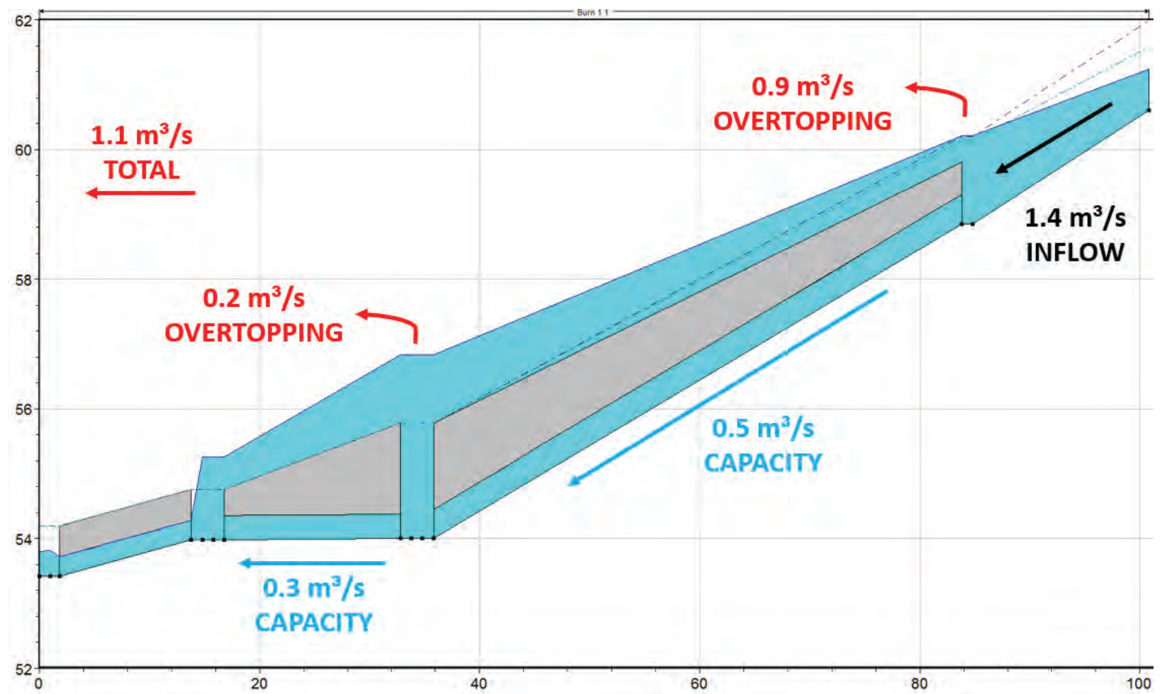



**Figure 19: Representation of Culvert capacity (0.3 m$^3$/s) and overland flow (1.1 m$^3$/s) in 200-year event (1.4 m$^3$/s)**



Therefore, in a 1 in 200-year flood event it is anticipated that overland flows would flow through the existing substation across the local road and back into Burn 1 and the low-lying, wetland area to the north-east of the site. Only 0.3 $m^3/s$ can be conveyed by the existing culvert.

A model run with only the 13 MO section of the culvert represented indicated that the triple (3x 0.3m) culverts have a conveyance capacity of approximately 0.45 $m^3/s$. This suggests that the existing triple culverts under the local road do not have sufficient capacity to convey the 1 in 200-year flood.

The existing culvert was not modelled for scenarios in excess of the 1 in 200-year event. The difference in flows between the 1 in 200-year event and any larger return period events are likely to flow overland through the site.

### 5.3.2 Proposed Culvert

The development proposals involve the extension of the Glenlee substation to the south, which will require the existing culvert to be realigned and lengthened. SEPA generally do not accept the culverting of open-channel watercourses. Additionally, it is likely that Burn 1 and the riparian corridor adjacent to the channel are at risk of flooding. SEPA generally do not accept development in areas at risk of flooding.

However, the proposed development can be considered “Essential Infrastructure” under the SEPA Land Use Vulnerability Classification as it would be impractical to locate the substation elsewhere. According to SEPA guidance, development of “Essential Infrastructure” can proceed so long as the development is designed and constructed to remain operational during 1 in 200-year floods and does not impede the flow of water. While not explicitly detailed in this guidance, development should not increase flood risk downstream or to others.

In order to permit the development of the substation extension it will be necessary to provide:

1. A culvert that can adequately convey the 1 in 200-year flow with no flooding; or
2. A culvert that can convey most flood events, with any overtopping flows conveyed through the site via a carefully designed overland flow pathway without posing a risk to the proposed development or increasing risk to others.

Discussions with SPEN have confirmed that, due to constraints with respect to cover and space for services within the proposed road and steep gradients it will not be possible to design a culvert that convey the 1 in 200-year flow.

For this reason, it was proposed to design a culvert that could convey the highest flows possible with any overtopping flows conveyed through the site via a carefully designed overland flow pathway. SPEN undertook modelling works in Microdrainage software to provide an indicative design of the proposed culvert. Kaya Consulting Limited subsequently developed a 1D HEC-RAS model to represent the culvert and provide confirmation of the suitability of the proposed design. This work was undertaken separately from the flood risk assessment and a separate report has been produced to support this work and is included as Appendix 1 of the FRA. This includes indicative proposals for the design of the spillway of which two suitable designs have been identified. A summary of the main findings is provided below.

The results show that the proposed culvert has a peak capacity of between 0.7 $m^3/s$ and 0.8 $m^3/s$. In a 1 in 200-year flood event flows are estimated to reach a peak flow of 1.4 $m^3/s$. Recommendations have





been made with respect to containing overland flows within an access road within the site and conveying them to a filter drain. This will contain flows within the road and make an attempt to drain them away to the filter drain. Excess flows that cannot be conveyed by the filter drain will need to be conveyed along the access road and across the local road and into the wetland area to the north-east of the site. The results of 2D overland flow modelling suggest that this will result in flood depths of no more than 0.3m and velocities of no more than 1m/s in the road. (see Figure 9, Appendix 1).

As the flood depths are less than 0.3m and velocities do not exceed 1 m/s the local road should be navigable by emergency vehicles in a 1 in 200-year flood event. Considering the restrictions on the site this may be acceptable. Additionally, the proposed culvert provides additional capacity over the existing culvert providing betterment and meaning that the flood risk to the road is reduced by the new proposals. There are no residential units located downstream of the site and flood risk downstream of the site is not increased.

The proposed culvert was not modelled for scenarios in excess of the 1 in 200-year event. The difference in flows between the 1 in 200-year event and any larger return period events are likely to flow overland through the site.

## 5.4 Surface Water Flooding

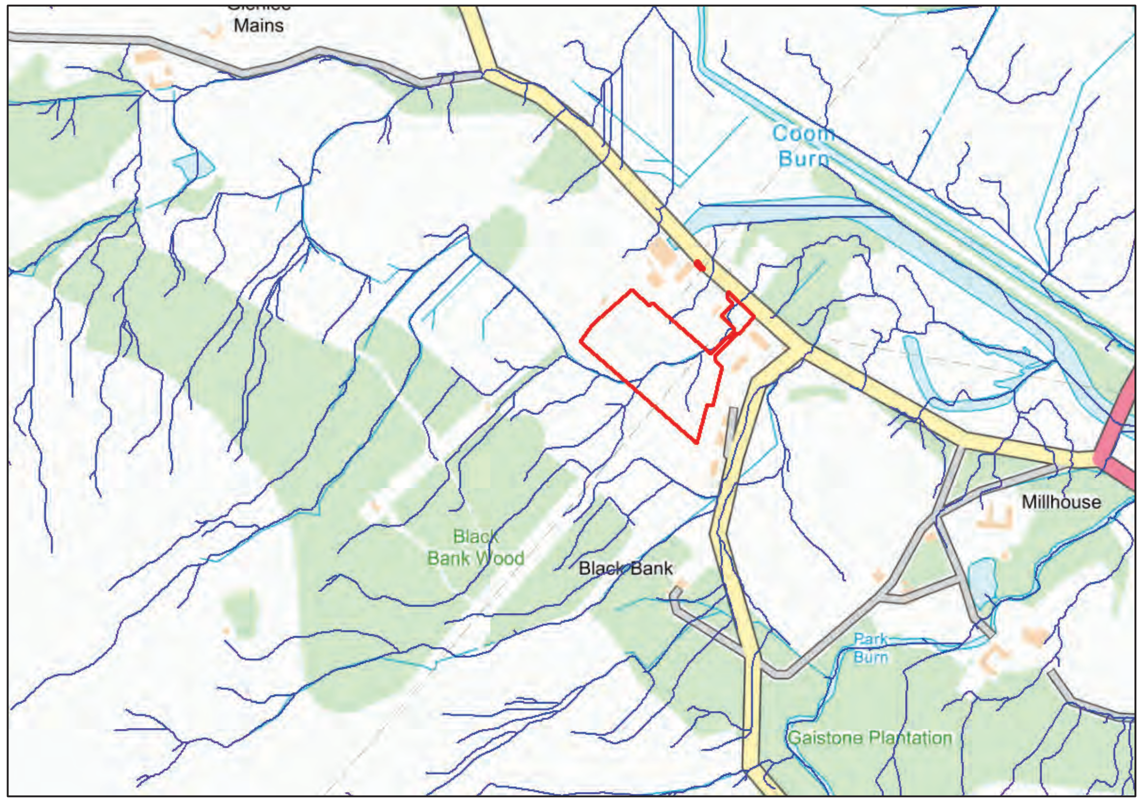
A detailed watershed analysis was undertaken using Global Mapper software to determine the route of any overland flows using the LiDAR DTM data. The results are shown in Figure 20.

The results of this analysis suggest that surface water from areas outwith the site will be predominantly intercepted by Burn 1. This indicates that the risk of the site flooding from surface water is strongly related to the flood risk from Burn 1. Surface water runoff from the areas shown in Figure 19 is included in the design flow for Burn 1. This is covered in Section 5.3 of this report.

Development proposals for the substation extension incorporate a cut-off ditch along the southern boundary of the site to intercept any surface water that is not drained by Burn 1.

It is further recommended that ground levels and overland flow pathways should be designed to convey any excess flows to landscaped areas and suitable discharge locations without impacting on the development or increasing flood risk to others.

Figure 20: Watershed analysis results






## 5.5 Groundwater Flooding

The SEPA flood map does not provide information on groundwater flooding for the site or surrounding area. Flooding from groundwater as a primary source is uncommon in Scotland.

The site lies close to numerous large watercourses including the Water of Ken and Coom Burn. It is likely that groundwater levels will be controlled by water levels within these watercourses. The site is generally located above the surrounding area and the levels of the watercourses meaning groundwater flooding is less likely.

Groundwater monitoring is normally undertaken as part of the Ground Investigation. If locally raised groundwater levels are identified during site investigations, suitable mitigation measures would need to be employed.

## 5.6 Flooding from Infrastructure

### 5.6.1 Drainage System

The design of the site drainage system is not part of this commission. However, as part of the development of the site a suitable drainage system employing SuDS should be designed to manage





surface water within the site. Ground levels and overland flow pathways should be designed to convey any excess flows to landscaped areas and suitable discharge locations.

Post development, surface water runoff from the site itself should be attenuated to existing or greenfield rates and discharged to a watercourse. Discharge rates should be discussed and agreed with Dumfries and Galloway Council.

Requirements for SuDS should be discussed and agreed with Dumfries and Galloway Council and SEPA.

### 5.6.2 Reservoirs & Flood Defences

Consultation of the SEPA reservoir map indicates that there are a number of reservoirs within the headwaters of the Water of Ken. These include Loch Doon, Loch Minnoch and Earlstoun Loch amongst others.

The SEPA reservoir map suggests that there are a possible 11 breach scenarios that could impact on water levels within the Water of Ken. In the majority of these scenarios flood waters would be pushed up the Tailrace and Coom Burn, flooding the low-lying wetland area to the north of the local road that runs adjacent to the Glenlee Power Station.

However, flood extents shown for the breach scenarios show that the majority of flows would be contained to the north of the local road. This suggests that the failure or breach of reservoirs in the headwaters of the Water of Ken is unlikely to lead to significantly worse flooding than that already modelled for a 1 in 1000-year plus climate change flood event. Considering that the existing site is protected up to this flood event by flood defences it is considered that flood risk to the site from reservoir breach or failure is low.

In addition, reservoirs and flood defences in Scotland are heavily regulated according to their risk of failure and are strictly maintained to mitigate against the risk of failure.

It has been confirmed that the existing development is protected by flood defences up to the 1 in 1000-year plus 30% climate change flood event. In the event of the failure of these defences, parts of the site closest to the local road may be at risk of flooding. However, considering that the flood defences were installed in 2013 according to the recommendations made in a report by Halcrow in 2011 the risk of these defences failing up to a 1 in 1000-year plus 30% climate change flood event is considered low.

## 5.7 Safe Access

Safe pedestrian access and egress should be provided to the site during extreme flood events so that visitors and workers can be safely evacuated without any undue risk to life.

Due to its proximity to the Water of Ken the section of the A762 close to the site is estimated to flood in all the events modelled by Halcrow (100 years and higher) as part of the previous Flood Risk Assessment. However, a comparison of peak water levels with the local topography suggests that the development could be evacuated up to the 1 in 200-year flood event via the local road to the east and then turning south along towards Black Bank. Both local roads are estimated to remain free from flooding up to a water level of 53.8m AOD, equivalent to a 1 in 200-year event in the Water of Ken, Coom Burn and Tailrace. In a 1 in 500-year flood event the water level would be approximately 54.1m AOD and the local road that fronts the power station may flood up to depths of approximately 0.3m.

Proposals for the realigned culvert to convey Burn 1 through the site will result in some flooding of the road within the site and within the local road in flood events (see Appendix 1). 2D modelling showed that flooding would spread out as it left the site and generally reach depths of no more than 0.3m. Velocities were estimated to be less than 1 m/s. While SEPA now request that "Flood-free" access is provided, wherever possible, it is considered that the local road should be navigable by emergency vehicles in most scenarios. SEPA may deem this level of flooding acceptable, considering the restrictions on the site. Site constraints mean that the culvert cannot be designed to convey the 1 in 200-year flow. The proposals provide betterment over the existing case (see Section 5.3 and Appendix 1)

Additionally, in the event of an emergency, the site could potentially be evacuated to the south-east via a farm track adjacent to an existing shelter. This farm track connects to the local road that runs to the south towards Black Bank.





# 6 Flood Risk Recommendations, Summary and Conclusions

Kaya Consulting Ltd was commissioned by Scottish Power Energy Networks (SPEN), through Land Use Consultants, to undertake a Flood Risk Assessment in support of the proposed Glenlee Substation Extension at Glenlee Power Station in Glenlee, close to St John's Town of Dalry.

A previous Flood Risk Assessment was undertaken by Halcrow (2011) to support the development of flood mitigation measures for the Glenlee Power Station. The Halcrow FRA report included a 1D mathematical model to represent the Water of Ken, Coom Burn and Tailrace. To confirm the general accuracy of the Halcrow results a new 2D mathematical model was developed for this assessment. The results of this assessment produce similar flood extents and flood levels to those published in the Halcrow (2011) FRA. It was therefore considered acceptable to use the results from the previous work to support this assessment.

The results indicate that the vast majority of the site lies above the 1 in 200-year, 1 in 500-year, 1 in 1000-year and 1 in 200-year plus climate change peak water levels. Small areas of the site fronting the local road would be at risk of flooding. In the event of a 1 in 1000-year plus 30% climate change flood event additional flooding of the site that fronts the local road would also occur. However, it has been confirmed that the Glenlee Power Station and Substation are now protected against flooding by flood defences up to the 1 in 1000-year plus climate change flood levels estimated in the previous Flood Risk Assessment. It is recommended these defences, or new ones, are maintained to protect the development up to this flood event. It should be emphasised that new climate change guidelines ask that flood risk be considered up to the 1 in 200-year + 44% climate change (or in some instances + 55% rainfall intensity) event. This event is estimated to be smaller than the 1 in 1000-year + 30% climate change event and so flood extents and water levels would sit somewhere between the 1 in 200-year + 30% climate change and 1 in 1000-year + 30% climate change scenarios that were previously modelled. 2D mathematical modelling undertaken in this study suggests that the site is not at risk of flooding from either the Dickson's Strand or Burn 2.

Burn 1 enters the site from under the Penstock traversing the site in a north-easterly direction. The watercourse is culverted under the existing Glenlee substation. The results of a 1D mathematical model suggest this culvert does not have sufficient capacity to convey the 1 in 200-year flood.

The development proposals involve the extension of the Glenlee substation to the south, which will require the culvert to be realigned and lengthened. SEPA generally do not accept the culverting of open-channel watercourses. Additionally, it is likely that Burn 1 and the riparian corridor adjacent to the channel are at risk of flooding. SEPA generally do not accept development in areas at risk of flooding. However, the proposed development can be considered "Essential Infrastructure" under the SEPA Land Use Vulnerability. "Essential Infrastructure" can proceed so long as the development is designed and constructed to remain operational during extreme floods and does not impede the flow of water. This report makes recommendations with respect to the proposed culvert and overland flow pathways to meet these requirements (see Appendix 1).

The results of the assessment suggest the site is at low risk of surface water flooding, groundwater flooding and flooding from infrastructure. Recommendations have been made to mitigate risk. Recommendations for safe access are provided in Section 5.7

It should be noted that the risk of flooding can be reduced, but not totally eliminated, given the potential for events exceeding design conditions and the inherent uncertainty associated with estimating hydrological parameters for any given site.





# Appendix 1: Assessment of Proposed Culvert and Overland Flow

Our Ref: 02/02/SS – KC1364 – V2.2
Your Ref:

16th August 2019

Iberdrola Engineering & Construction
Ochil House
10 Technology Avenue
Ground Floor
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Blantyre
G72 0HT

**For the attention of Stephen Jack and Sandy Boyd**

Dear Mr Jack,

**Glenlee Substation Extension FRA**
**Assessment of proposed Culvert Design and Overland Flow**

Kaya Consulting Limited were asked to carry out an assessment of the proposed design for the realigned culvert that passes through the Glenlee Substation Extension. This letter provides the requested information and forms an Appendix to the FRA report.

## 1. Background

SPEN have proposed a realigned culvert to replace the existing structure through the Glenlee Substation. The proposals include the realignment of the culvert to the east and provision of an additional spur of pipework to increase capacity within the relatively flat part of the network where flows are conveyed under the Local Road.

Due to local topographical factors it will be difficult to develop a design that can convey the 1 in 200-year flow. For this reason, it is proposed that flows in excess of the capacity of the network will be conveyed along the proposed substation road network, within the site, and intercepted by a “Road Verge Drain”. Indicative calculations have been undertaken to determine if there will be sufficient capacity in the road and Road Verge Drain to permit this.

The main aim is to minimise flooding within the site itself and downstream of the culvert.

## 2. Proposed Culvert

A HEC-RAS 1D mathematical model was developed to represent the proposed culvert. Existing site plans and topographical data were used to represent Burn 1, just upstream of the culvert, with the culvert itself represented based on the drawing provided named “Glenlee Extension – Enabling Works – Drainage Layout” (1CA-2-11HD-DO-SPENEC-4106). The parts of the culvert and how they were represented in the model are shown in Table 1.

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The model was divided into 3 reaches: Reach 1 represents Burn 1 upstream of the culvert and the first section of the culvert up until the flows are divided between two lengths of pipework. Reach 2 represents the part of the culvert that discharges downstream of the site via the proposed triple-barrelled culverts under the Local Road. Reach 3 represents part of the culvert that discharges downstream of the site via the existing triple-barrelled culverts under the Local Road. This is depicted visually in Figure 1. It is also proposed that in extreme events some flows at the culvert inlet to 83 CULV will be conveyed around the substation platform via a filter drain. The capacity of this structure has been assessed separately at the end of this section.

**Table 1: Model Details**

| | Cross Section | Description | Diameter | Gradient |
|---|---|---|---|---|
| Reach 1 | 195 - 125 | Head of the model – 2x cross-section to represent Burn 1 just upstream of the culvert with Interpolates to improve stability | - | - |
| | 83 CULV | Proposed steep section of culvert as it passes through the substation. 600mm Diameter. | 0.6m | ~ 1 in 20 |
| | 34 - 33 | Manhole downstream of steep section of culvert. Flows here are divided with some going to Reach 2 and others to Reach 3. | - | - |
| Reach 2 | 28 - 27 | Cross-sections that allow flows to be conveyed to Reach 3. | - | - |
| | 25 CULV | Proposed section of culvert of 450mm Diameter. | 0.45m | ~ 1 in 25 |
| | 10 - 9 | Manhole downstream of 25 CULV that conveys flows into proposed triple-barrelled culvert under the Local Road | - | - |
| | 5 CULV | Proposed triple-barrelled culvert under the Local Road | 0.3m x3 | ~ 1 in 30 |
| | 3-2 | Channel downstream of 5 CULV culvert. End of Reach. | - | - |
| Reach 3 | 30 - 29 | Cross-sections that allow flows to be conveyed to Reach 2. | - | - |
| | 27 CULV | Proposed section of culvert of 450mm Diameter. | 0.45m | ~ 1 in 70 |
| | 15-14 | Manhole downstream of 27 CULV that conveys flows into existing triple-barrelled culvert under the Local Road | - | - |
| | 13 CULV | Existing triple-barrelled culvert under the Local Road | 0.3m x3 | ~ 1 in 30 |
| | 1 - 0 | Channel downstream of 13 CULV culvert. End of Reach. | - | - |

CULV – culvert structure

The model has a total length of approximately 200m and represents the culvert within the site and Burn 1 just upstream and downstream of the site. Manning’s n roughness values were set to 0.02 for the culvert sections and manholes and 0.06 for the channel and overbank areas of the burn. Unsteady flows were input into the model using the estimated FEH (Flood Estimation Handbook) Rainfall-Runoff hydrograph. The downstream model boundary was set as a “stage level” boundary with a level of 53.8m AOD to represent a 1 in 200-year flood level within the Tailrace (and other relevant watercourses) downstream. A sensitivity analysis run with a normal depth downstream boundary of 0.01 provided very similar results as that using the “stage level” boundary. To maintain model stability a continuous base flow of 0.6 m³/s was added to the model.

The model profile results for the 200-year event and the location of the cross-sections described in Table 1 are shown in Figures 2 and 3. Culverts are shown in red text.

**Figure 1: Model Schematic**

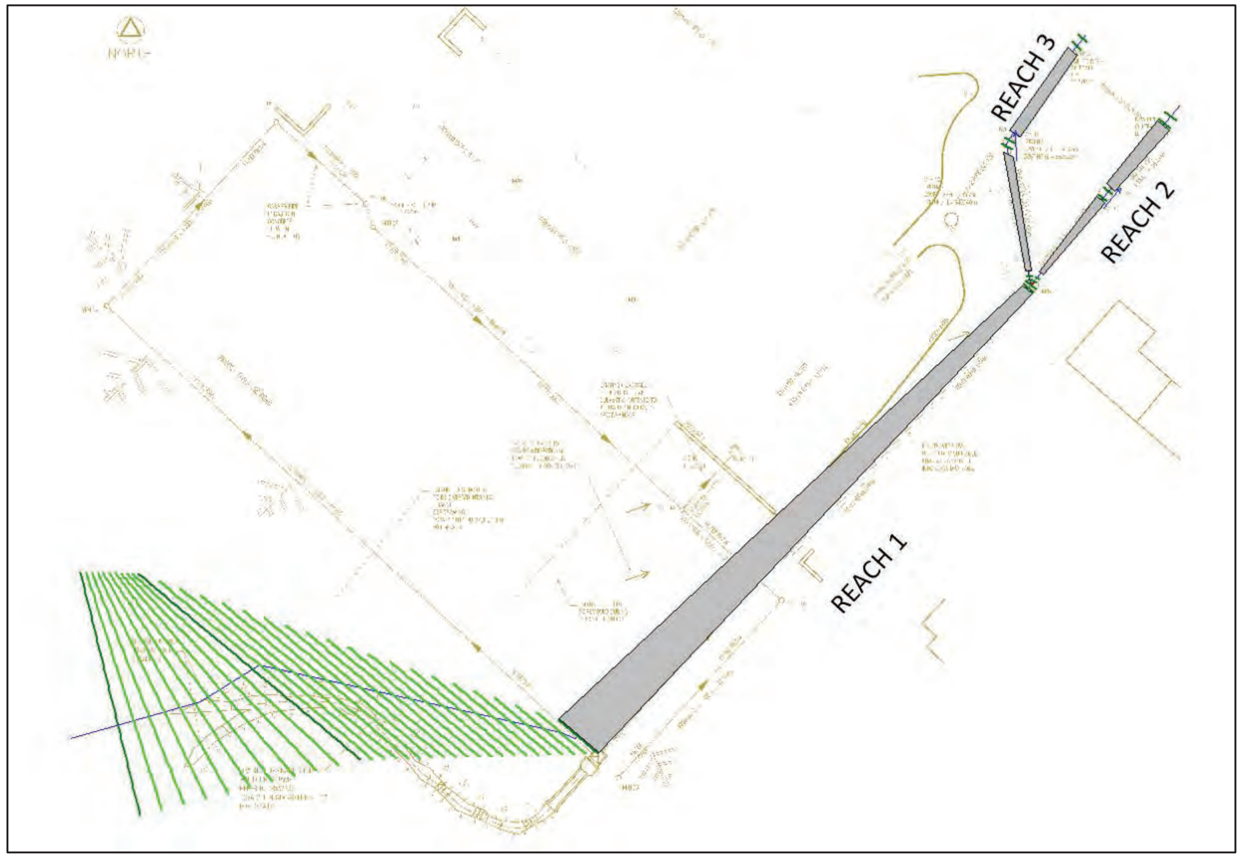



**Figure 2: Model Profile – Reach 1 & Reach 2 – 200-year flow**

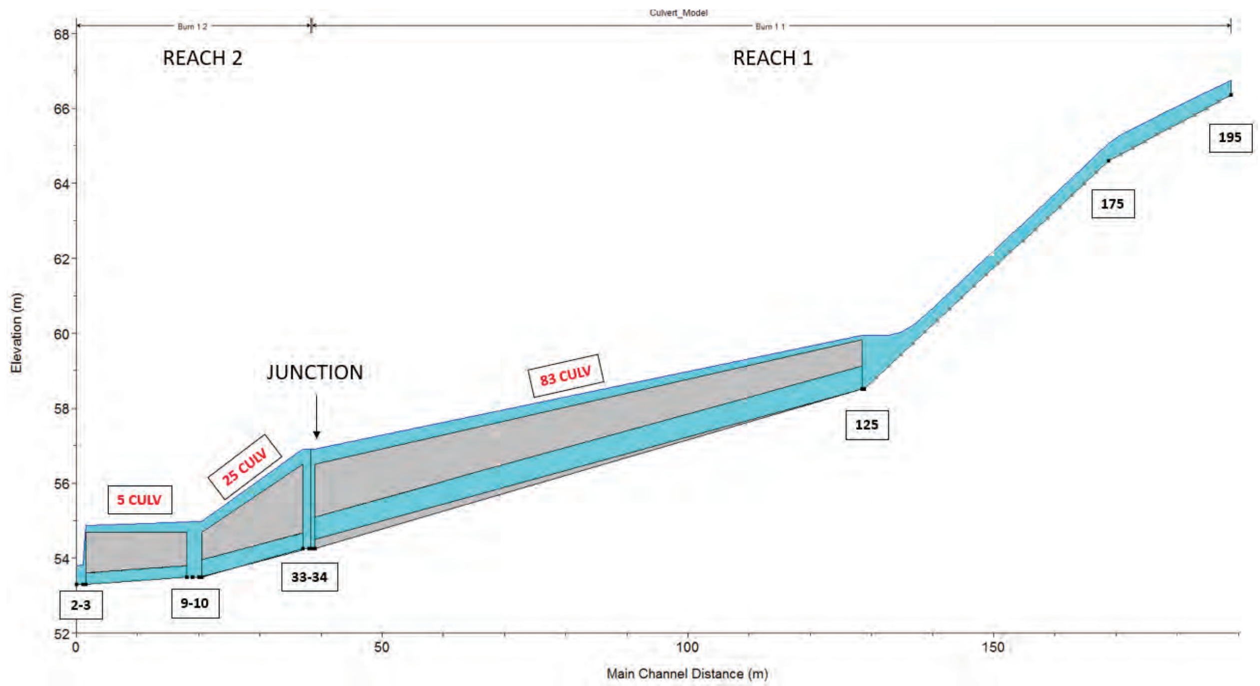
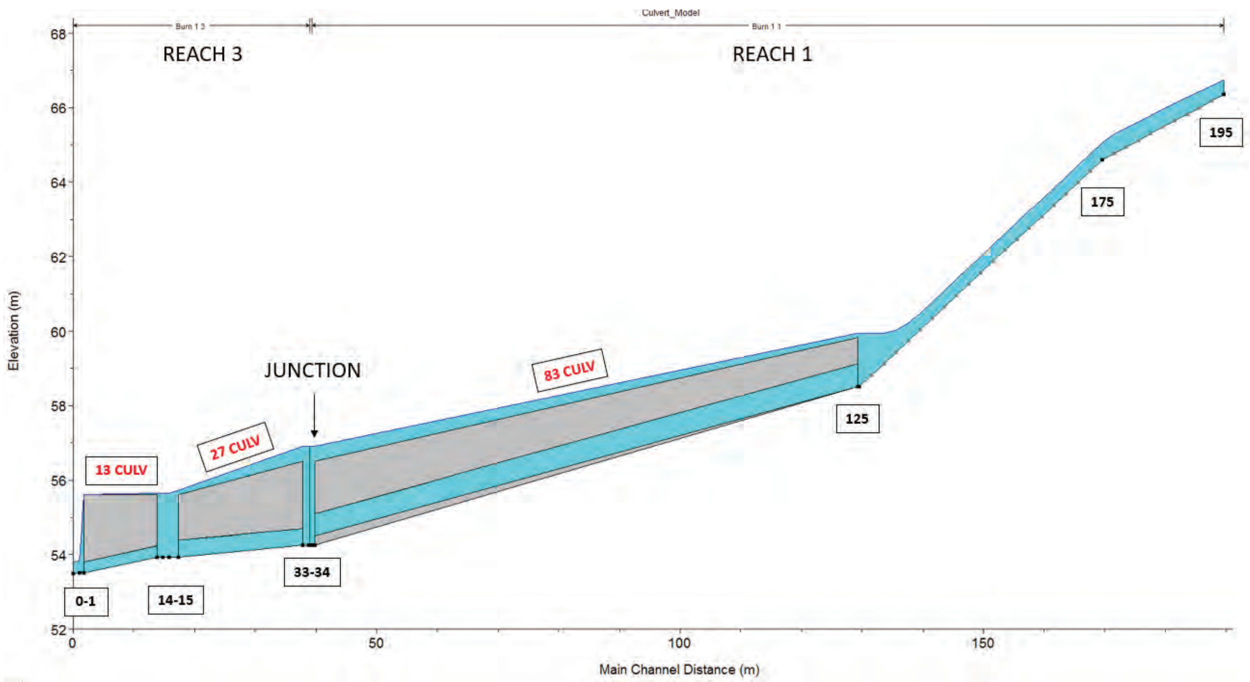


Figure 3: Model Profile – Reach 1 & Reach 3 – 200-year flow





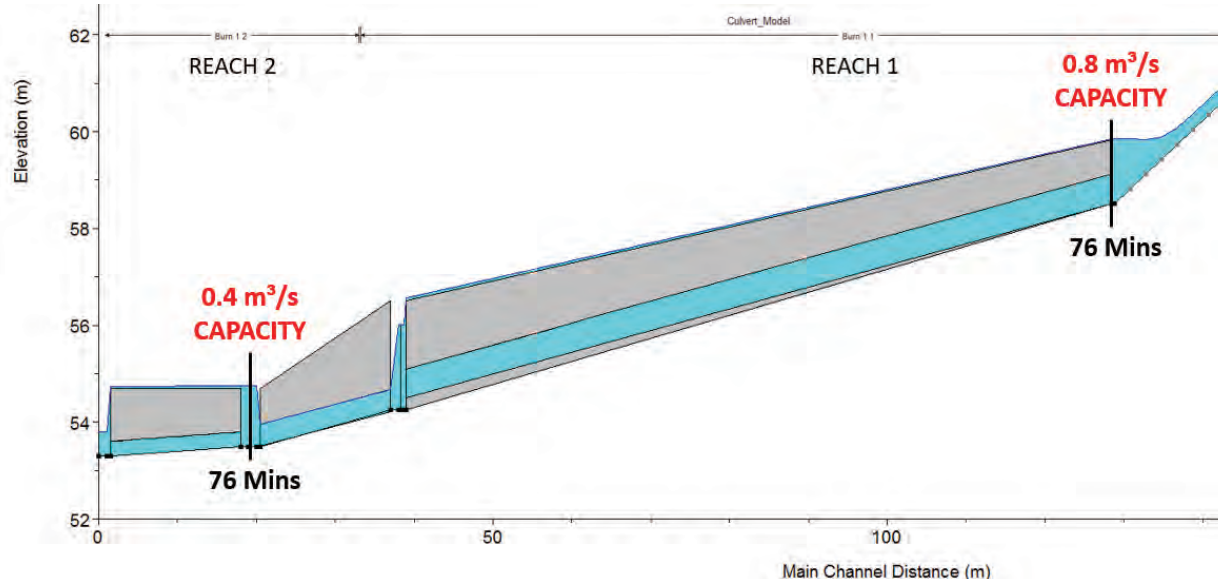

The results of the modelling suggest that, as anticipated, the proposed culvert cannot convey the full 1 in 200-year flow of approximately 1.4 m³/s.

Examining the model results throughout the flood event help provide additional information on when and where flooding may occur.

The results suggest that during the base flow of 0.6 m³/s all flows can be contained within the channel and culvert although some manholes are beginning to surcharge, predominantly those upstream of 83 CULV and 5 CULV, suggesting these culvert sections are close to capacity at this flow rate.

At approximately 76 minutes into the 4-hour 200-year flood event the 83 CULV section becomes overwhelmed with flows overtopping the manhole and flowing across road network within the site. This occurs at flows of approximately 0.8 m³/s. At approximately the same time 5 CULV becomes overwhelmed with flows overtopping the manhole. At this point the capacity of 5 CULV is approximately 0.4 m³/s, as approximately half the flows are also being conveyed through Reach 3. The ability of the whole culvert to convey flows is limited by these two sections of the culvert. This is shown in Figure 4.

Figure 4: Model run at approx. 76 minutes into the run showing 5 CULV and 83 CULV surcharging and overtopping




A short time later, at approximately 83 minutes into the flood event, both 27 CULV and 25 CULV become overwhelmed. At this point 25 CULV is conveying approximately 0.5 m³/s and 27 CULV 0.45 m³/s. This is shown in Figure 5. Flows do not surcharge and overtop at 13 CULV until approximately 1 hour 40 minutes into the flood event when this culvert section is conveying approximately 0.6 m³/s (Figure 6). By this time a significant rate of flow is being conveyed overland and is not within the culvert system.

The reason for 13 CULV continuing to convey flows for longer than the similar 5 CULV culvert section is likely due to the slightly higher downstream invert level, which will be marginally less impacted by the downstream water level boundary and, perhaps more importantly, the greater cover proposed for MH13 over MH03.

**Figure 5: Model run at approx. 83 minutes into the run showing 25 CULV and 27 CULV surcharging and overtopping**

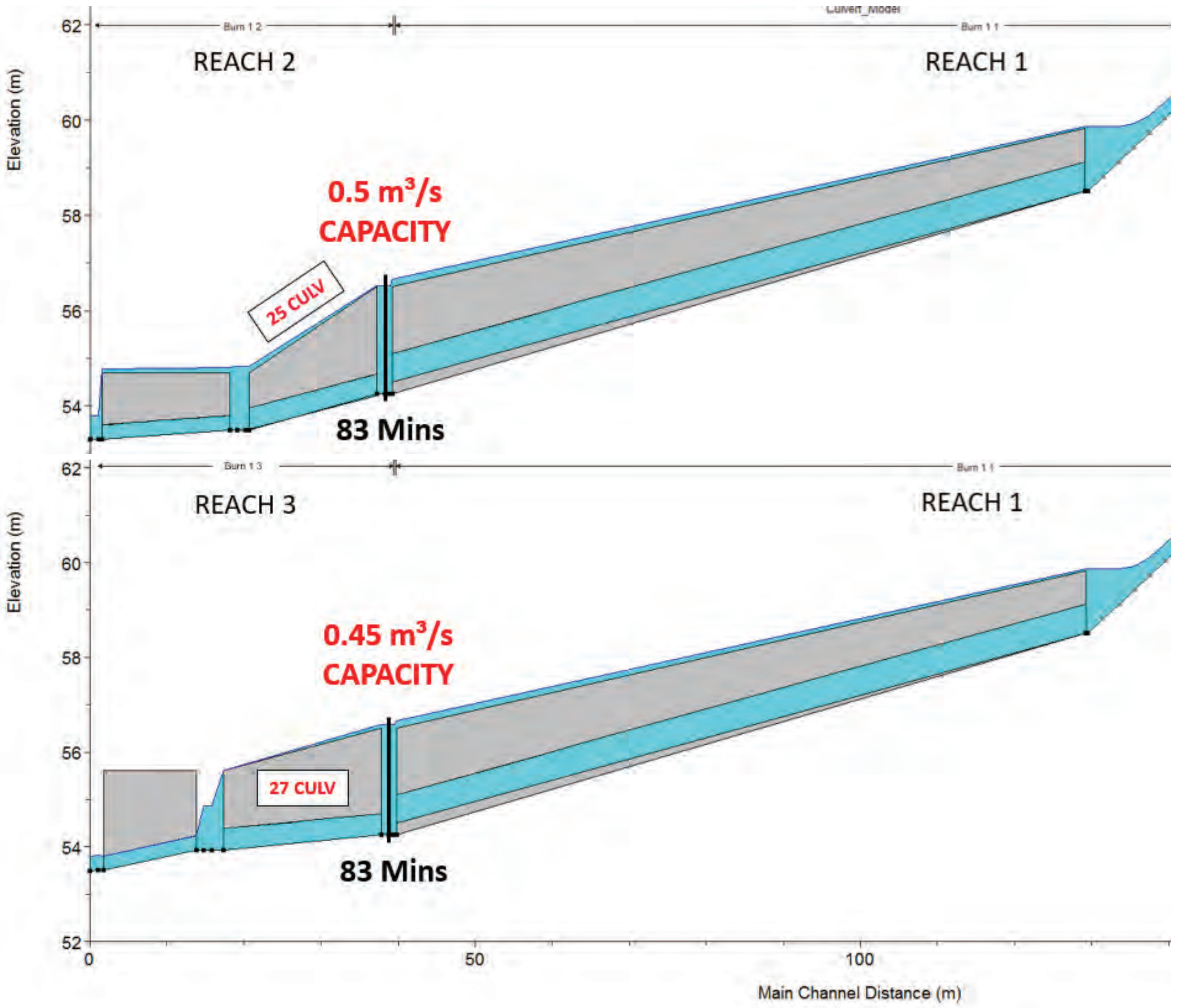
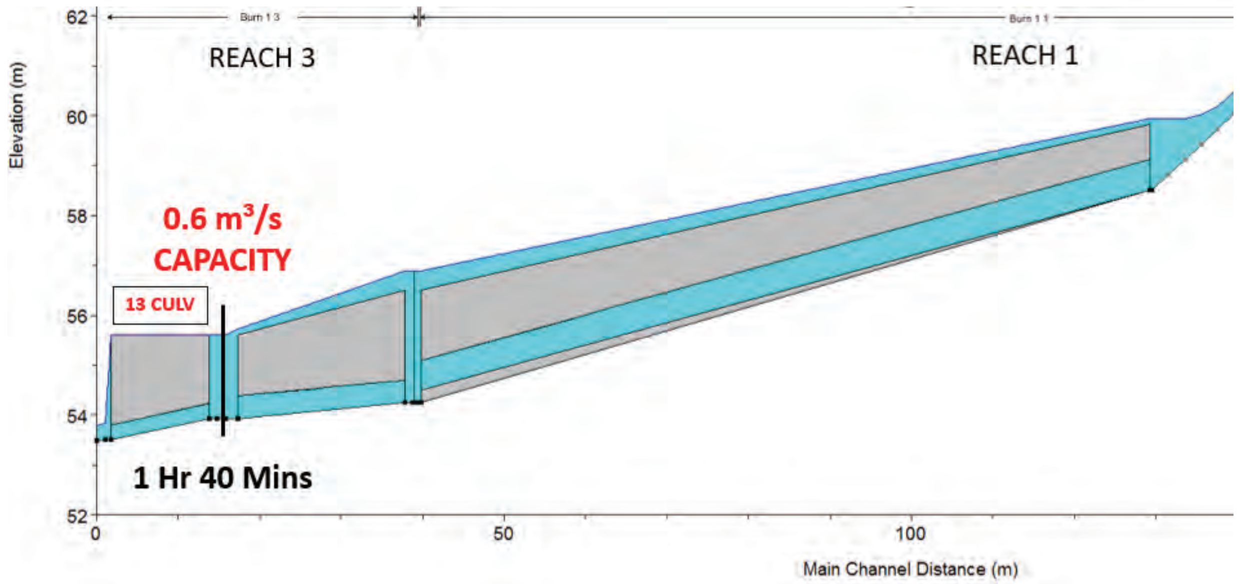





**Figure 6: Model run at approx. 1 hr 40 minutes into the run showing 13 CULV surcharging and overtopping**




The results of the assessment suggest that the culvert section 83 CULV is the limiting section of the culvert. While this culvert has a peak capacity of approximately 0.8 m³/s, the model results suggests that capacity during the peak of the flood event is lower, at around 0.7 m³/s, due to restrictions in the culvert sections downstream.

The model does not allow for the additional capacity of the filter drain that will convey flows around the substation extension and along the existing culvert route connecting back into the proposed culvert at MH13, upstream of 13 CULV. Indicative calculations using the Colebrook-White method suggest a capacity of no more than 0.1 m³/s.

Accounting for this capacity, the model suggests that the remaining flow of 0.6 m³/s would overtop at the inlet to the culvert. These flows would then be conveyed overland through the site and towards the Local Road and low-lying wetland area to the north-east of the site. In the 1 in 1000-year event flows overtopping the culvert at this location would be approximately 1.2 m³/s, assuming a peak flow of 2.0 m³/s.

It is proposed to design the site so that flows can be contained within the proposed road network within the site and conveyed to a Road Verge Drain, limiting flows that reach the Local Road.

It should be noted that this will not stop flooding of the Local Road as flows will surcharge from 5 CULV and 13 CULV both of which are located downstream of the proposed Road Verge Drain. Nevertheless, the addition of the second line of pipework (5 CULV and 25 CULV) provides additional capacity within the system which will reduce the flood risk to the Local Road compared to the pre-development situation.

## 3. Road Capacity

SPEN provided Kaya Consulting Limited with cross-sections representing the proposed ground levels for the site, including proposed dimensions of the road network within the site. This data was used to undertake indicative calculations to estimate the capacity of the road.

A Manning's Equation calculation was undertaken to estimate the capacity of the proposed road. The road was assumed to be a rectangular shape, allowing for a kerb on either side. In reality the road will need to fall towards the proposed road verge drain (Section 4). The road was noted to be 4.5m wide, at its narrowest, and have a slope of 1 in 85, at its shallowest. The road was assumed to be "gravel", with a Manning's *n* roughness value of 0.025, which is a higher friction value than asphalt or concrete. These parameters were chosen to represent the most "conservative" scenario. The results of the calculation suggest that flood depths within the road would reach approximately 0.13m, if 0.6 $m^3/s$ were to be conveyed along the road (1 in 200-year event). If the flows along the road were 1.2 $m^3/s$ (1 in 1000-year event) then flood depths would reach approximately 0.2m. Flood depths would be shallower in areas where the road is wider and where the gradient is steeper. The same is true if the road is surfaced with asphalt or concrete, which have lower friction values.

Considering the estimated flood depths of approximately 0.13m, in a 1 in 200-year event, it is recommended that parts of the road that are not conveyed into the Road Verge Drain be provided with a kerb (or verge) that is higher than this to maintain flows confined within the road. Additionally, Finished Floor Levels should be raised a minimum of 0.6m above the level of the kerb or verge to act as a freeboard.

It is generally recommended that Essential Infrastructure, such as substations, are protected against the 1 in 1000-year flood event. Considering that the 1 in 1000-year flood depth within the road is 0.2m, only 0.07m higher than the 1 in 200-year event the above recommendations would still provide a freeboard in excess of 0.5m.

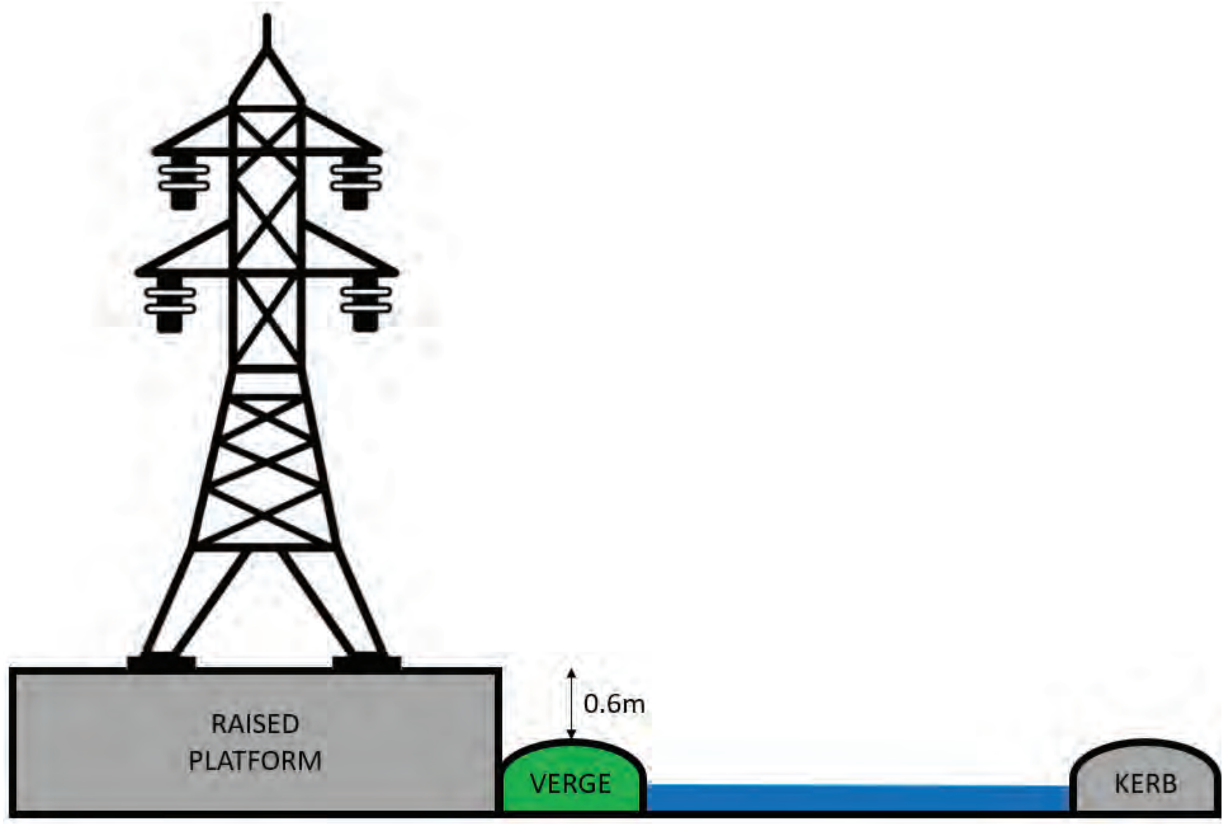
Details of the Manning's Equation are provided in Table 2. An example schematic showing road and Finished Floor Levels is provided in Figure 7.



**Table 2. Manning's Equation - Road**

| Manning's Equation – Road | |
|---|---|
| Road Width (m) | 4.5 |
| Kerb Height (m) | 0.3 |
| Gradient (1 in X), S | 85 |
| Manning's Roughness, n | 0.025 |
| | |
| Cross-sectional area of flow, A (m²) | 0.9 |
| Wetted Perimeter, P | 4.9 |
| Hydraulic Radius, R | 0.18 |
| | |
| **Flow Depth (m)** | **0.2** |
| **Flow Conveyance Capacity (m³/s)** | **1.2** |

**Figure 7: Example schematic of road and finished floor levels**




## 4. Road Verge Drain

It is proposed that the road through the site, via which excess flows will be conveyed, will be drained by a "Road Verge Drain" which will intercept some flows within the road and reduce flood depths. It is proposed that the drain be a "Filter Drain". A filter drain generally consists of a trench lined with a geotextile and filled with gravel or other suitable material with a perforated pipe in the bottom to aid conveyance. Conveyance capacity is generally equal to that of the pipe with a limited capacity provided by the trench itself.

The filter drain should ideally be able to convey the 1 in 200-year overland flow, although due to site constraints this will not be possible in this scenario. The proposed drain will serve to intercept some of the overland flows and, therefore, reduce flood depths within the road.

Guidance suggests it is only necessary for the Finished Floor Levels of the Essential Infrastructure to be raised sufficiently above the 1 in 1000-year event. There is no requirement to “manage” the 1 in 1000-year flow. The proposals for the road (Section 3) will maintain the 1 in 1000-year flows contained within the road network.

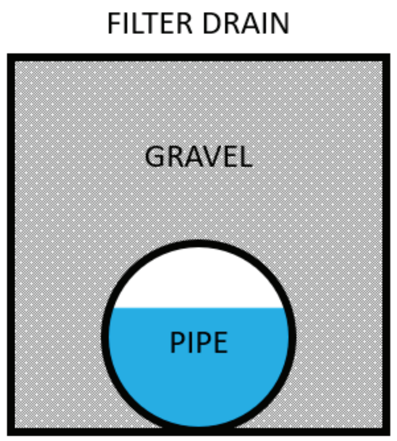
A Colebrook-White calculation suggests that a pipe with a diameter of 0.3m would be capable of conveying an overland flow of approximately 0.25 m³/s. The calculation assumes a gradient of 1 in 20 and an upstream water depth of 1m. The data provided to Kaya Consulting Limited suggests that the gradient is greater than 1 in 20.

**Table 3. Manning’s Equation – Filter Drain (Pipe)**

| Manning’s Equation – Filter Drain (Pipe) | |
|---|---|
| Pipe Diameter (m) | 0.3 |
| Gradient (1 in X), S | 20 |
| | |
| Cross-sectional area of flow, A (m²) | 0.07 |
| | |
| **Flow Conveyance Capacity (m³/s)** | **0.25** |

An allowance for the storage capacity of the trench has not been incorporated. It is likely that the storage capacity will be limited due to the steep gradient of the drain. Filter drains are generally designed with gradients of less than 1 in 100 to provide storage and water treatment benefits. Figure 8 shows an example diagram of a filter drain.

**Figure 8: Example schematic of a Filter Drain**




These calculations suggest that the proposed filter drain will be capable of conveying 0.25 m³/s with the remaining 0.35 m³/s being contained within the road network. Calculations suggest that this would reduce flood depths within the road to 0.09m in a 1 in 200-year flood event.

It should be noted that the proposed filter drain runs close to the site boundary within close proximity to existing residential properties. In events in excess of the capacity of the filter drain flows may back up out of the proposed filter drain. A barrier (kerb, verge, wall, bund, etc) should be provided between the drain and the existing



residential units. This barrier should be higher in elevation than the proposed verge or kerb on the other side of the road.

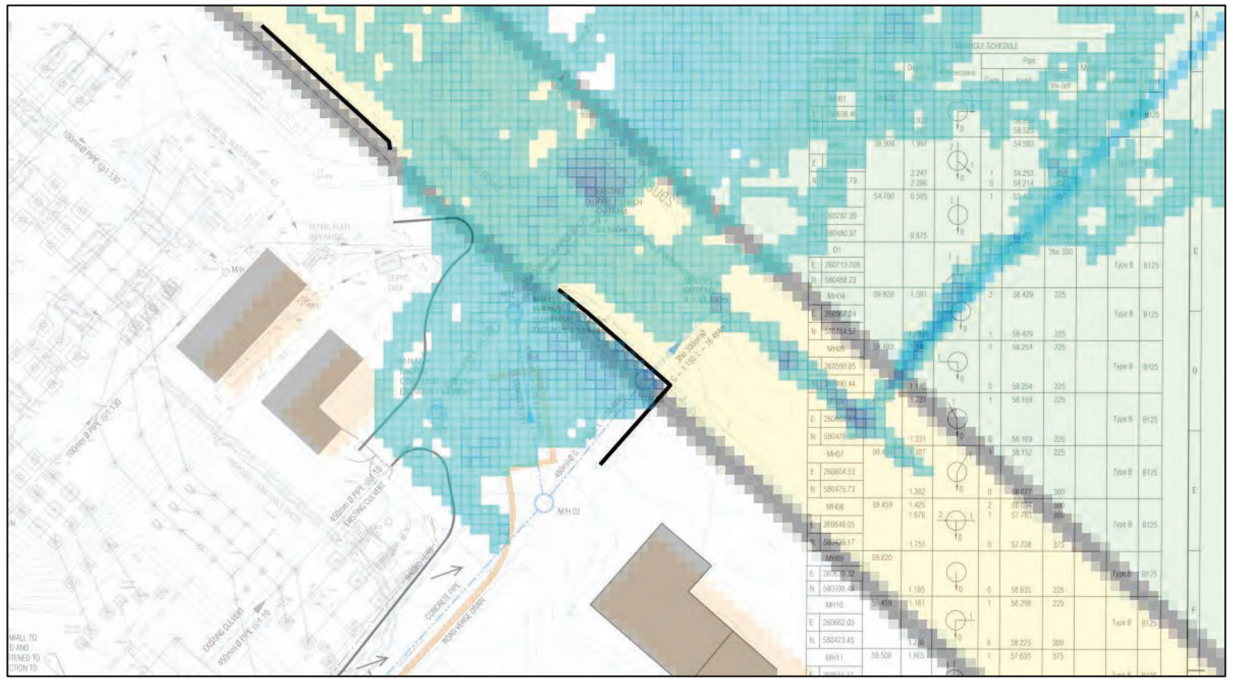
The filter drain will only convey flows to the lower part of the site close to the Local Road. Once flows reach this point they will spill out onto the road within the site.

At this point there are two options: Provide storage capacity within the site to mitigate against flooding. Alternatively, flows could be permitted to leave the site via the site access road and across the Local Road towards the wetland area, as they would do in the existing situation.

It has been calculated that it would be necessary to provide approximately 1700m³ storage within the site. It is not thought likely that this could be easily accommodated.

2D overland flow modelling was undertaken using Flood Modeller software to represent the flows that would leave the filter drain, and adjacent manholes, and flow down the site access road and across the Local Road. The existing stone walls were represented in the model. This showed that flooding would spread out as it left the site and generally reach depths of no more than 0.3m. Velocities were estimated to be less than 1 m/s. The flood extent is shown in Figure 9.

**Figure 9. Overland flow pathway after leaving the filter drain**





As the flood depths are less than 0.3m and velocities do not exceed 1 m/s the Local Road should be navigable by emergency vehicles in a 1 in 200-year flood event. SEPA may deem this level of flooding acceptable, considering the restrictions on the site.  Additionally, the proposed culvert provides additional capacity over the existing culvert meaning that the flood risk to the road is reduced by the proposals. Moreover, there are no residential units located downstream of the site.

## 5. Proposed Spillway

The proposed extension to the substation will require a realignment to be undertaken of Burn 1. To permit the watercourse to discharge into Culvert 1 a steeper channel, or spillway, is necessary. Two proposals for the spillway have been put forward:

- Spillway 1: A stepped concrete design;
- Spillway 2: A rock-cut design.

Spillway 1 is a concrete spillway with a stepped design with weirs provided at the end of each step. This design has a rectangular channel, similar to a flume of 1.2m width. Bank height is in excess of 2m. Each step is approximately 1.3m long, although the drop from each step to the next varies. The weir height at the end of each step if of 0.5m.

Spillway 2 is of a less man-made design using the excavated rockface to form the channel. This design includes a far larger number of steps of a smaller height, cascading down to the culvert inlet. This design uses a trapezoidal channel shape with side slopes cut back at a gradient of 1 in 1, cut into the rock. The base channel width would be 1.2m with a channel depth of approximately 2.25m.

Table 4 shows the advantages and disadvantages of each design.

**Table 4: Advantages and Disadvantages of the proposed spillway designs**

| | ADVANTAGES | DISADVANTAGES |
|---|---|---|
| **SPILLWAY 1** | **1.** Stepped design will reduce velocities and reduce the potential for erosion;<br>**2.** Steps will catch sediment and reduce the likelihood of blockage of Culvert 1. | **1.** Frequent maintenance will be required to maintain the effectiveness of the stepped design. Over time each step will fill with sediment and debris and reduce the benefits of the stepped design.<br>**2.** A more heavily-engineered less natural design. |
| **SPILLWAY 2** | **1.** A less heavily-engineered and more natural design;<br>**2.** Less likely to require maintenance. | **1.** Does not catch sediment meaning there is a greater risk of blockage to Culvert 1;<br>**2.** Design does not provide a reduction in flow velocities meaning there is a greater flow turbulence and a greater risk of erosion. |

Both Spillway 1 and Spillway 2 have been modelled using suitable hydraulic equations and have been confirmed to work hydraulically with predicted maximum water levels staying within bank. Both spillway designs provide a suitable freeboard.

As part of either design a suitable trash-screen should be provided designed in line with guidance to ensure the capacity of Culvert 1 is not restricted and that the peak flow can still be conveyed in the event of “blinding” (Blockage).



## 6. Additional Considerations

The proposed culvert has a peak capacity of between 0.7 $m^3/s$ and 0.8 $m^3/s$. This flow equates to a 1 in 17 to 1 in 25 year flood event, meaning that it has between a 6% and 4% chance of being exceeded in any one year. This means that there is between a 6% and 4% chance of the culvert being overwhelmed and flooding occurring within the site in any one year.

A maintenance regime should be put in place to maintain the proposed culvert including the inlet and the proposed channel to mitigate against blockages.

It has been noted that the culvert manholes are likely to surcharge in a 1 in 200-year flood event. Measures should be put in place to direct flows to suitable discharge locations. For example, a pathway should be provided to encourage flows surcharging from MH02 to be conveyed into the filter drain or access road. Flows surcharging from MH03 may be held within the site by the bounding site wall. It may be more appropriate to provide an escape route to allow flows to leave the site and be conveyed across the road, if they cannot be held on site. Either way flooding of the adjacent residential properties should be mitigated against.

## 7. Study Limitations

The modelling undertaken does not directly allow for surface water from within the site draining to the culvert. The catchment used to estimate the flows incorporates the proposed substation extension but not all of the existing site. Considering the conservative nature of the flow estimation undertaken in the assessment any additional flows from the existing substation site would be unlikely to have a significant impact on the results.

Modelling has only been undertaken until just downstream of the culvert outfalls under the Local Road. The model therefore does not account for the capacity of the channel downstream. As the modelling assumed a 200-year flood level in the Tailrace (and other watercourses) as the downstream boundary this does not matter.

Manholes cannot be explicitly modelled in HEC-RAS. They have been represented as rectangular cross-sections. The model makes the assumption that there are no manhole covers.

The channel upstream of the culvert has been represented based on the existing channel topography. Proposals with respect to the revised channel, check dams, etc. have not been incorporated at this stage.

A hydraulic model is only ever a “representation” of the proposals. Numerous industry-standard assumptions with respect to losses and other parameters are made. For example, for unsteady flow, HEC-RAS solves the dynamic, 1-D Saint Venant Equation using an implicit, finite difference method. In this equation, fluid motion is controlled by the three basic principles of conservation of mass, energy and momentum. The software simplifies very complicated scenarios using empirical approximations and numerical models based on these basic principles. For example, the software uses a simple, numerical model to calculate minor losses in meander bends which may not entirely represent real-life losses.

## 8. Summary and Recommendations

Kaya Consulting Limited were asked to carry out an assessment of the proposed design for the realigned culvert that passes through the Glenlee Substation Extension.

The results show that the culvert has a peak capacity of between 0.7 $m^3/s$ and 0.8 $m^3/s$. In a 1 in 200-year flood event flows are estimated to reach a peak flow of 1.4 $m^3/s$. Recommendations in this report have been made with respect to containing overland flows within an access road within the site and conveying them to a filter drain. This will contain flows within the road and make an attempt to drain them away to the filter drain. Excess flows that cannot be conveyed by the filter drain will need to be conveyed along the access road and across the Local Road and into the wetland area to the north-east of the site. The results of 2D overland flow modelling suggest that this will result in flood depths of no more than 0.3m and velocities of no more than 1m/s. This may be acceptable to SEPA. Alternatively, storage could be provided within the site but this would need to be in excess of 1700m³ to stop all flows leaving the site.

This assessment has been undertaken to inform SPEN of design constraints of the proposed culvert and discussions should be undertaken with them to progress the proposals. It is recommended more detailed modelling works are undertaken at a later date.

We trust the above satisfies your current requirements. If you have any queries regarding this please do not hesitate to contact the undersigned or Glen Perez-Livermore.

Yours faithfully,

[redacted]

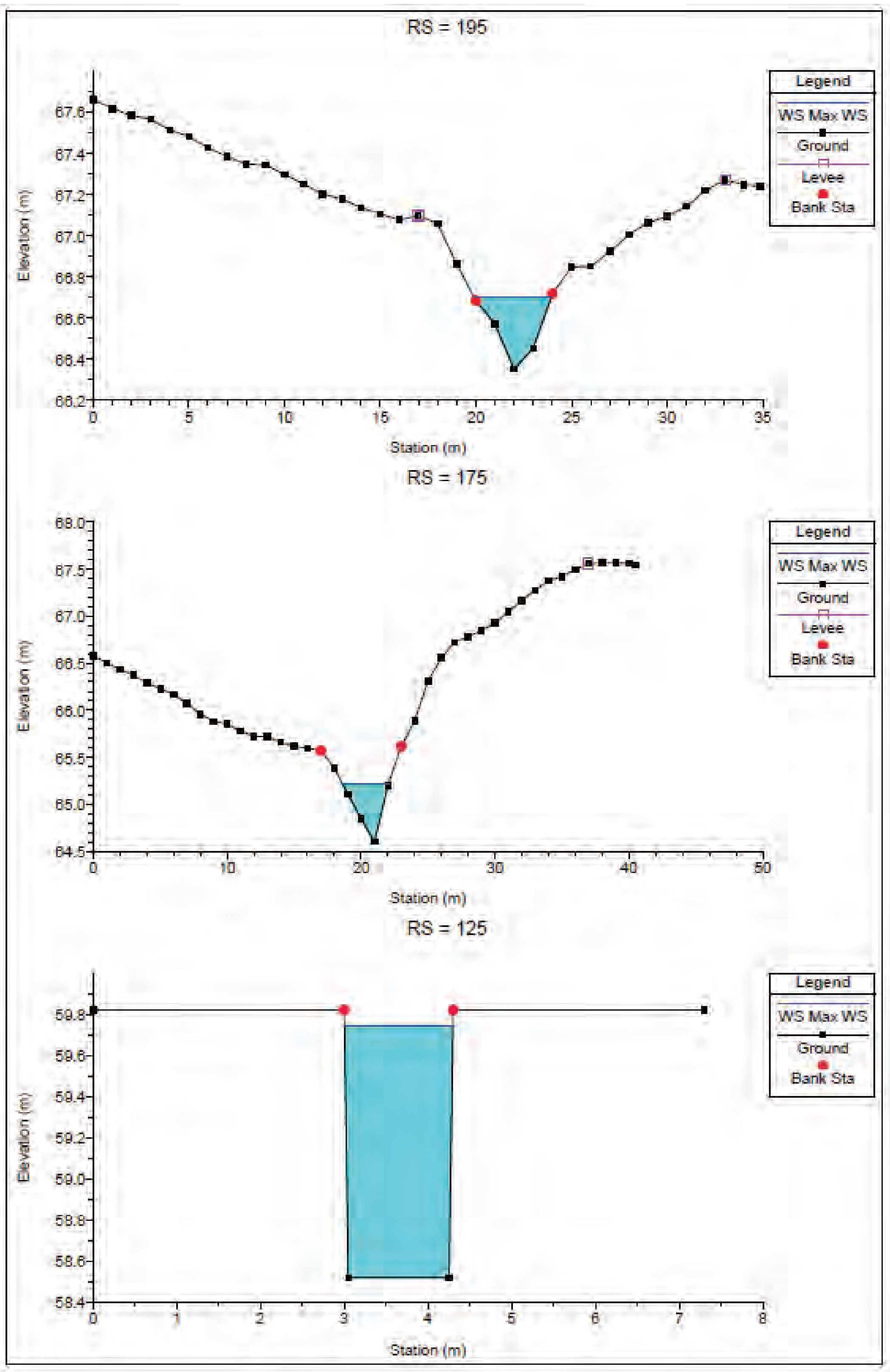
Dr Sally Stewart
Senior Environmental Consultant

**References**

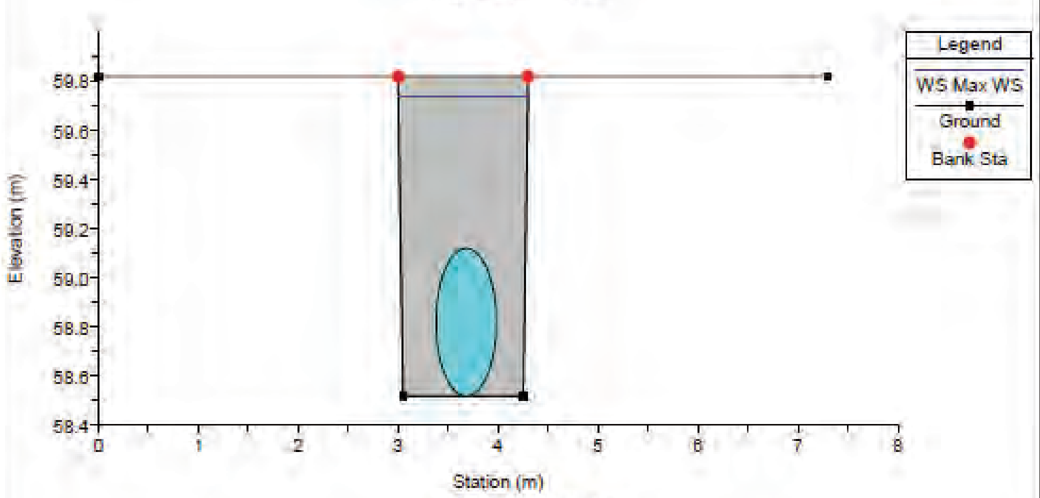

Kaya Consulting (2019) Glenlee Substation Extension Development, St John's Town of Dalry: Flood Risk Assessment



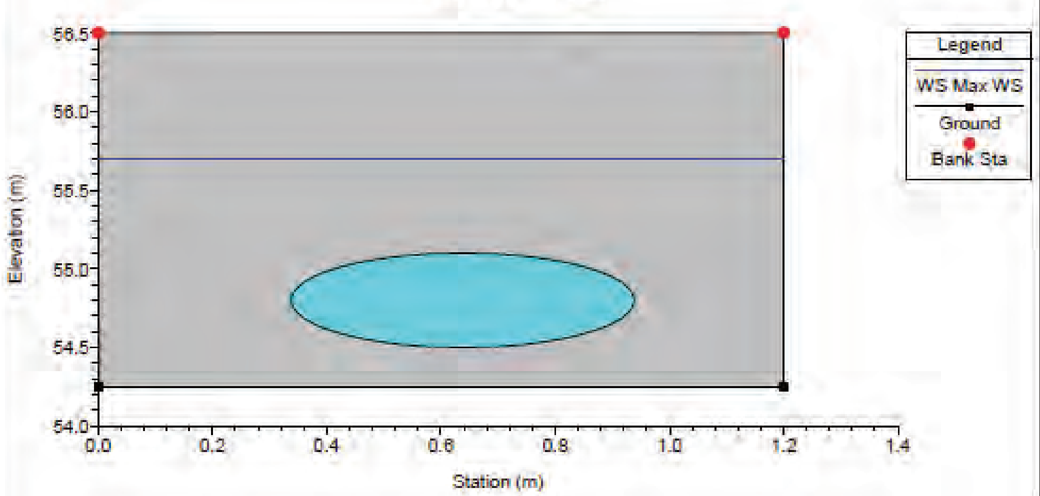

**APPENDIX – REPRESENTATIVE CROSS-SECTIONS**



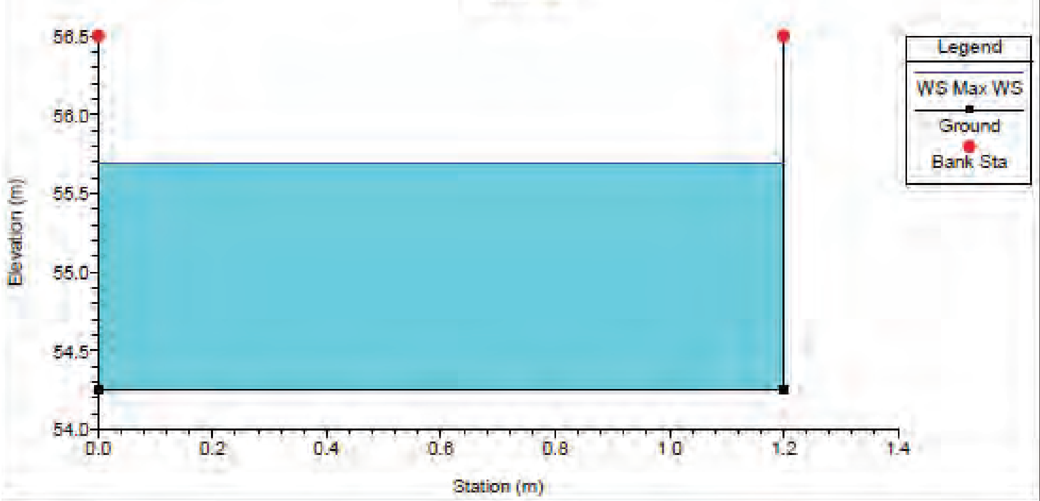
RS = 83 Culv




RS = 83 Culv

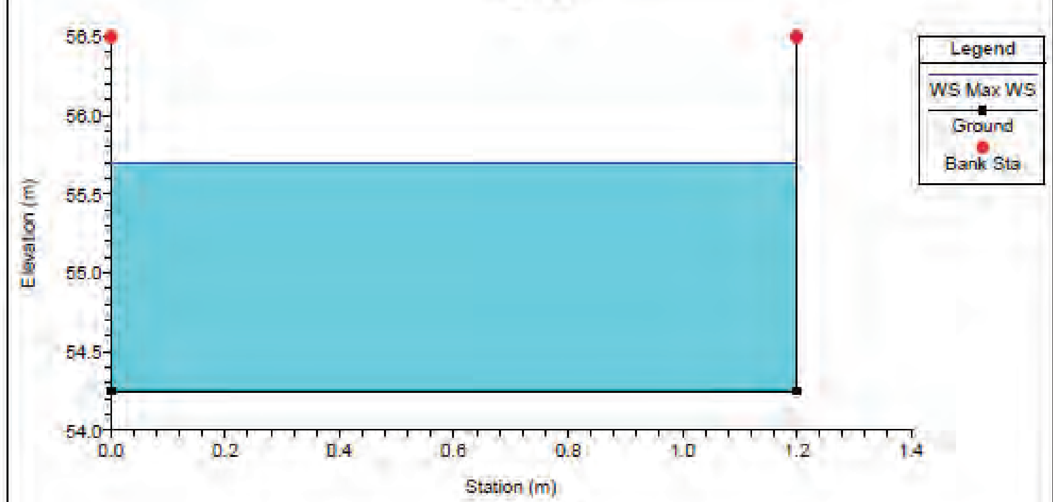



RS = 34

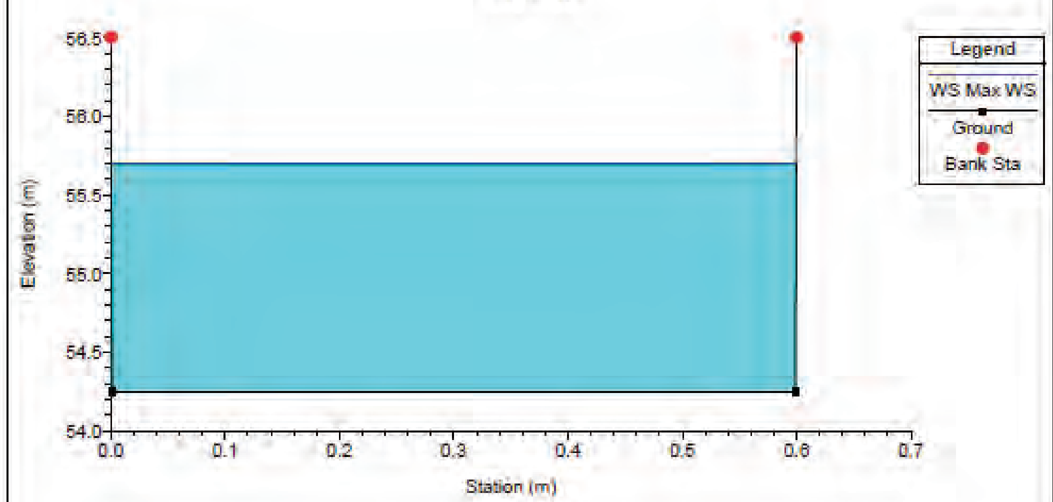




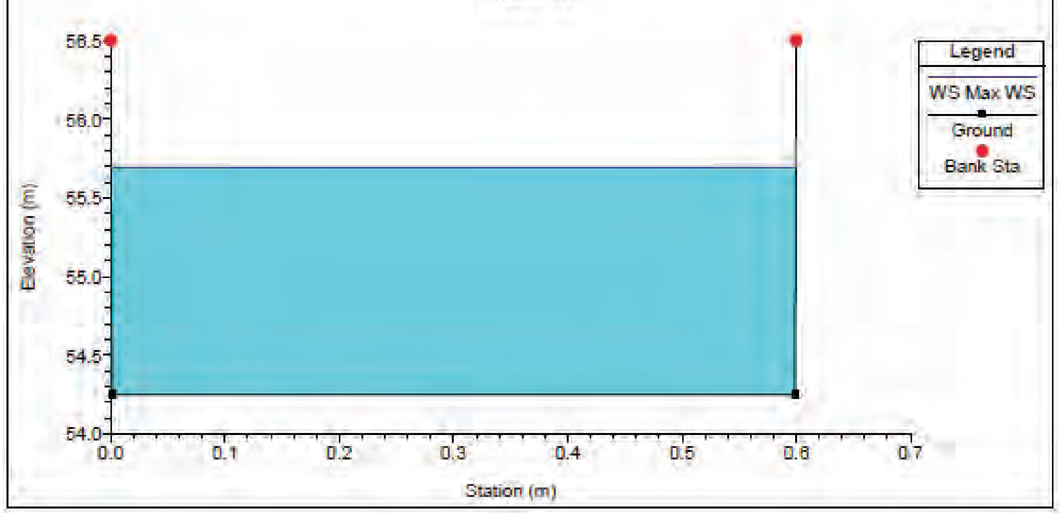

RS = 33

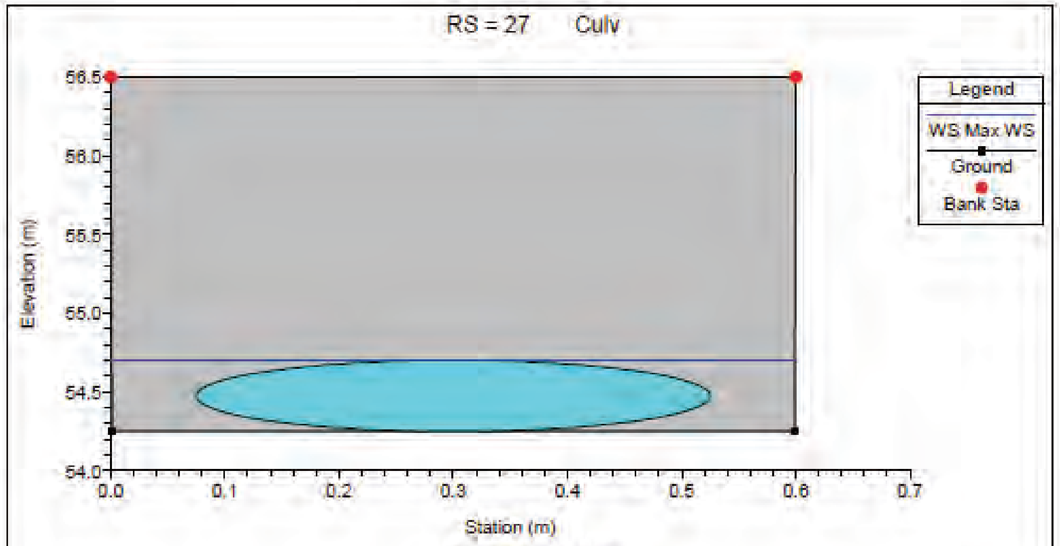


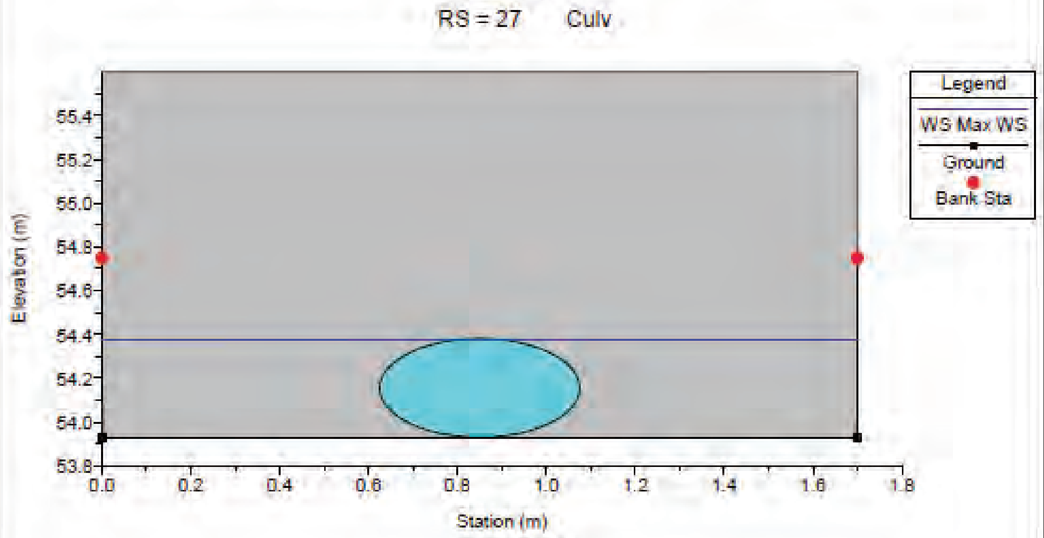

RS = 30

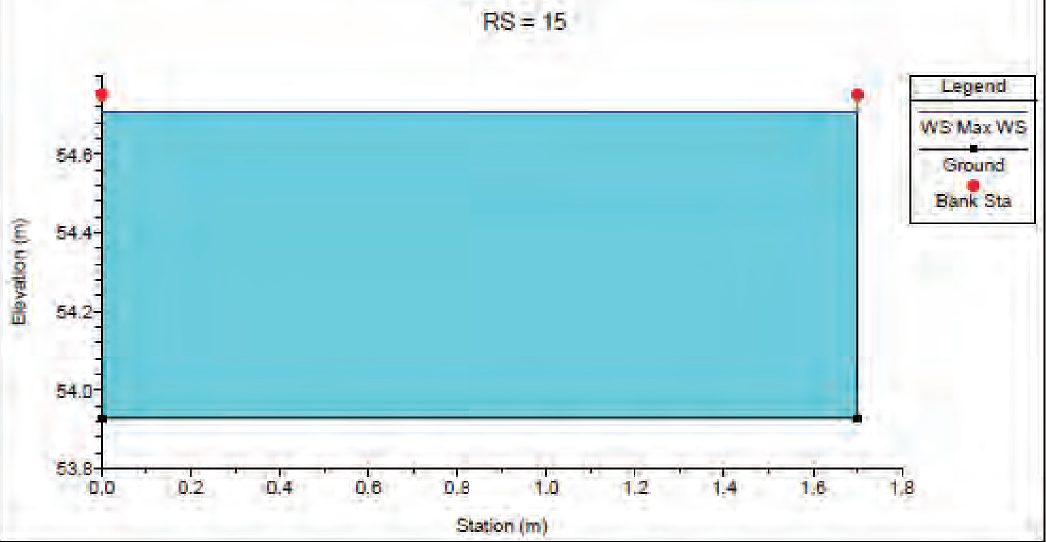



RS = 29

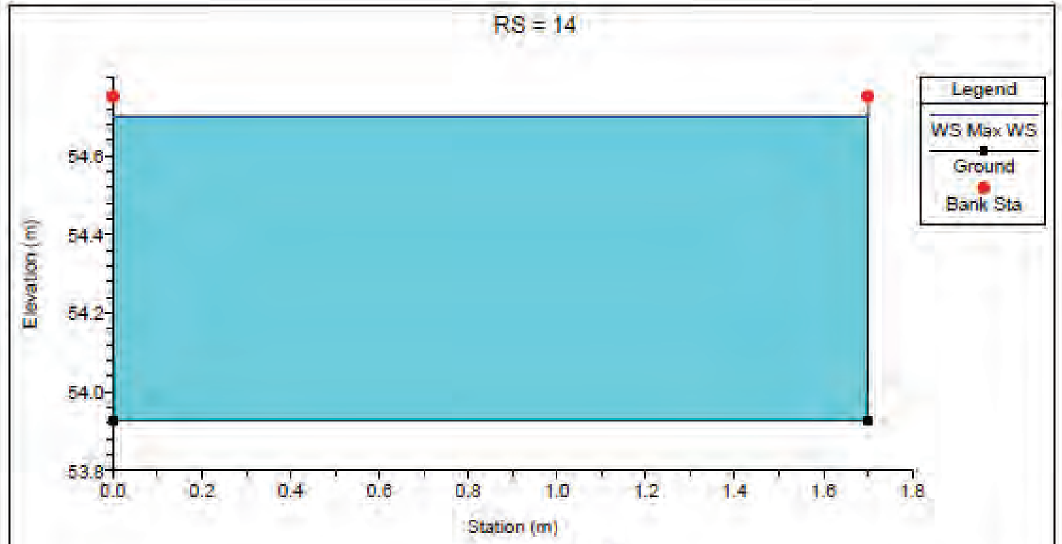


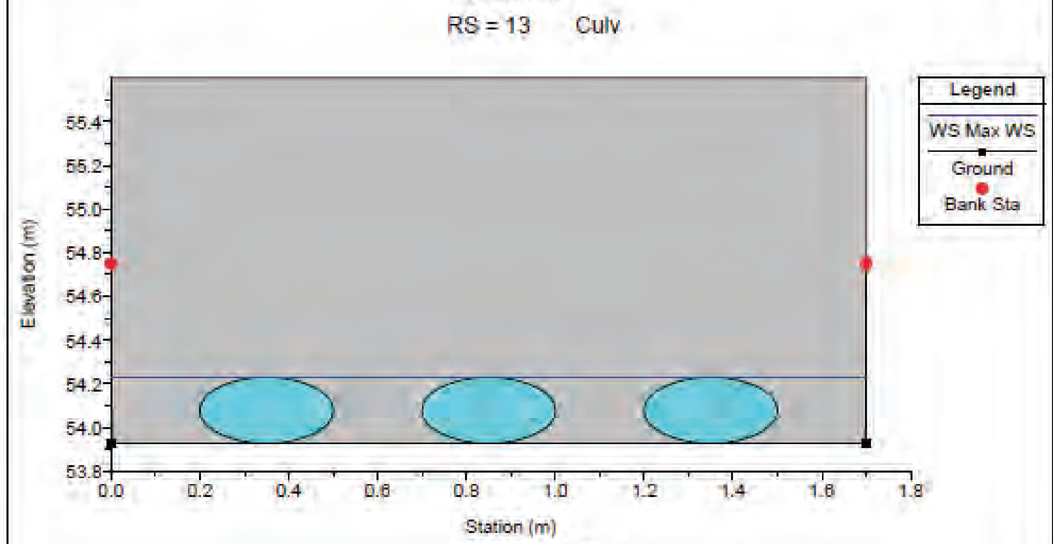

RS = 27 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

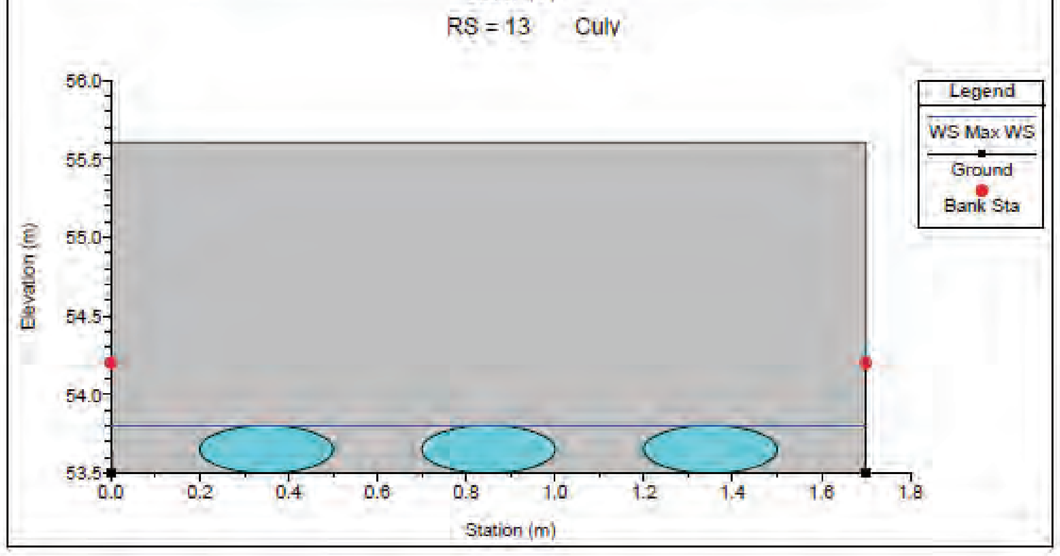

RS = 27 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

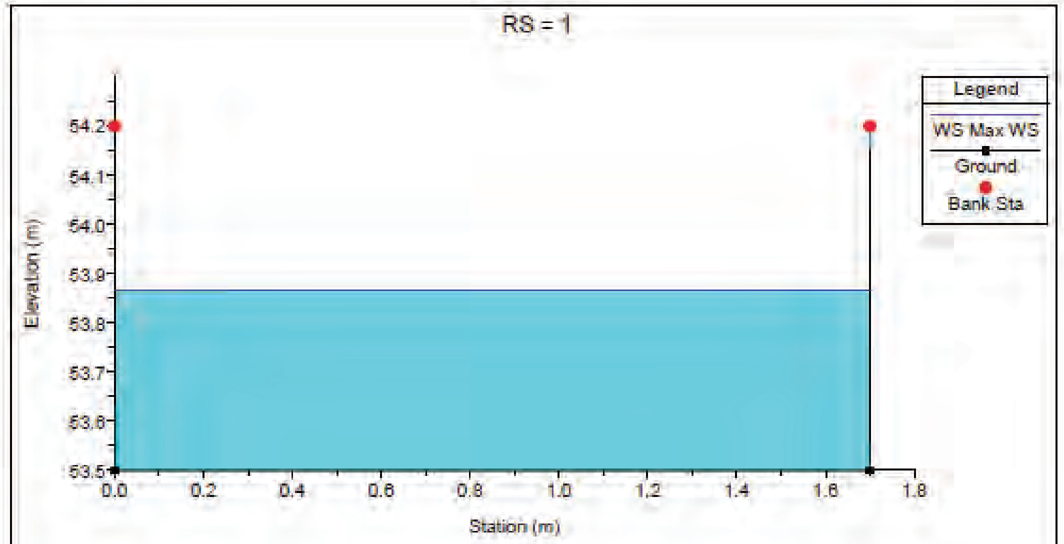

RS = 15
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

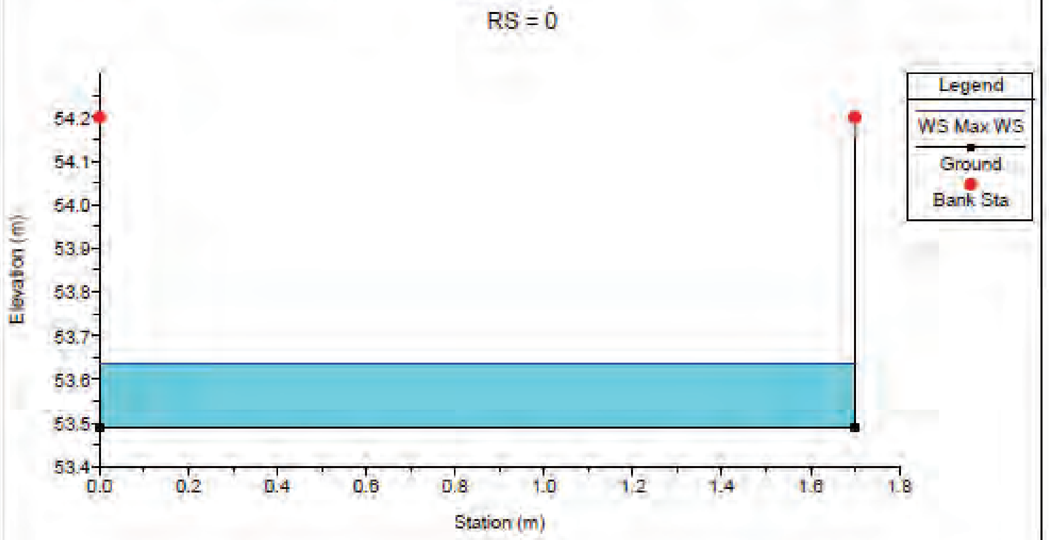


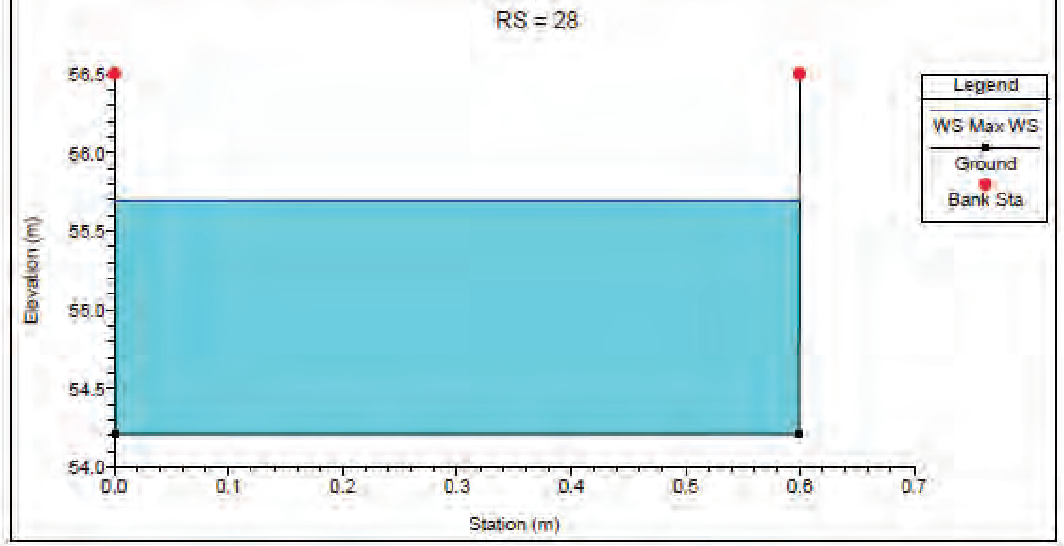

RS = 14
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)


RS = 13 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

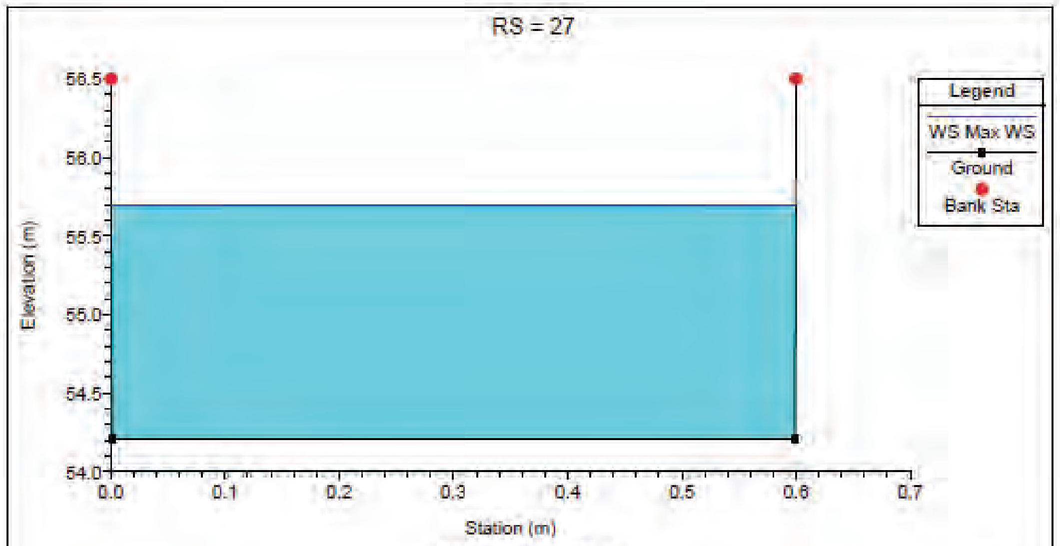

RS = 13 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

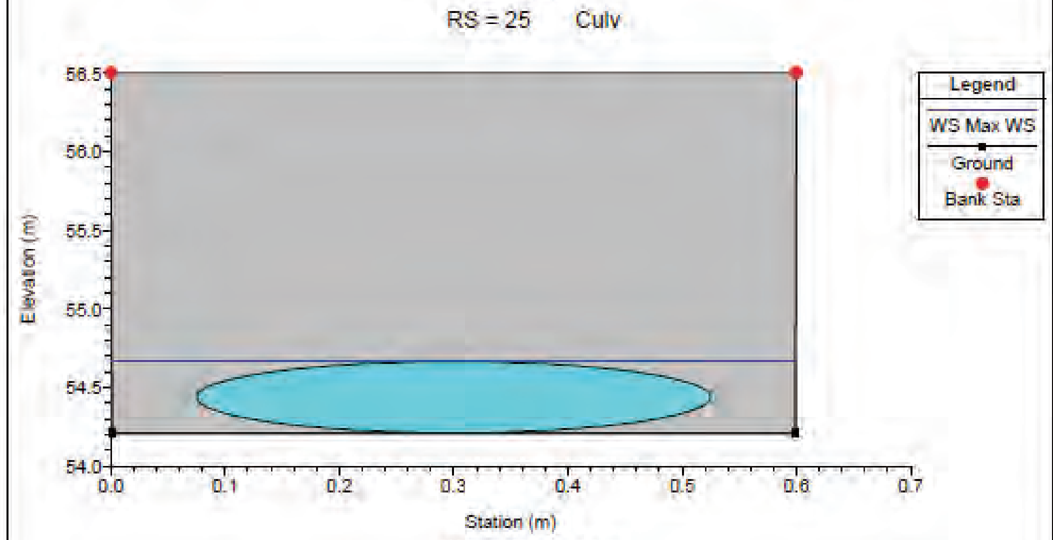

RS = 1
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

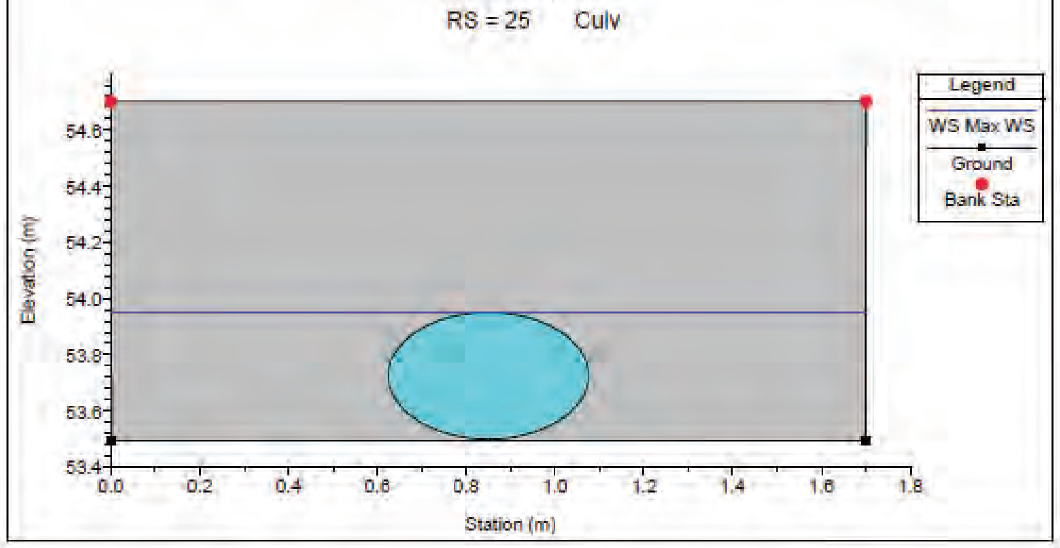

RS = 0
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

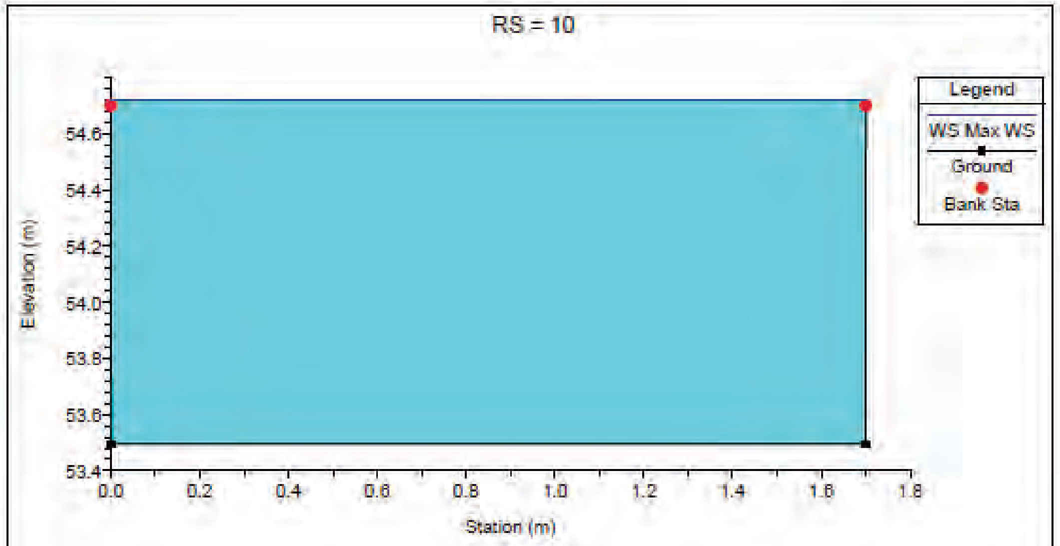

RS = 28
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

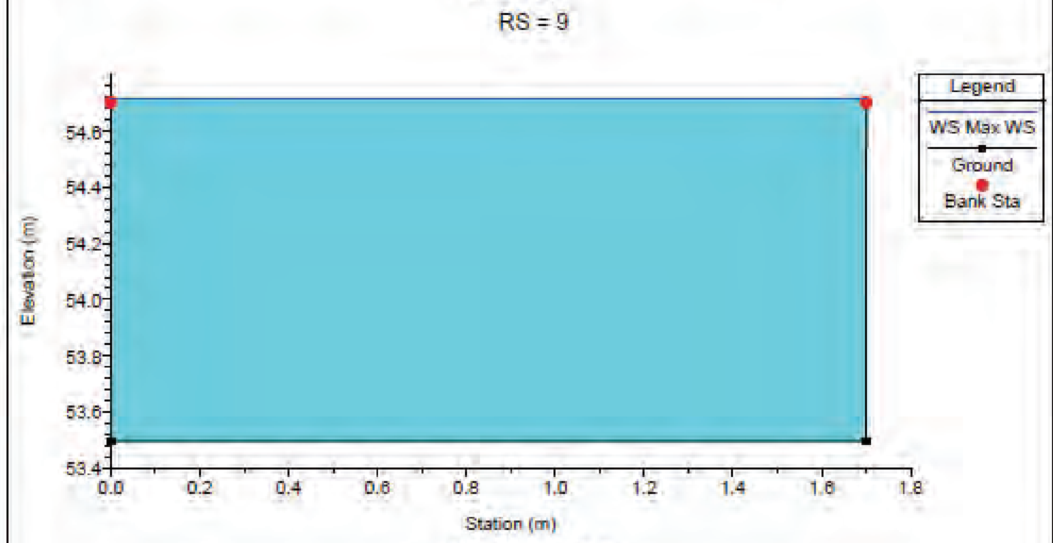


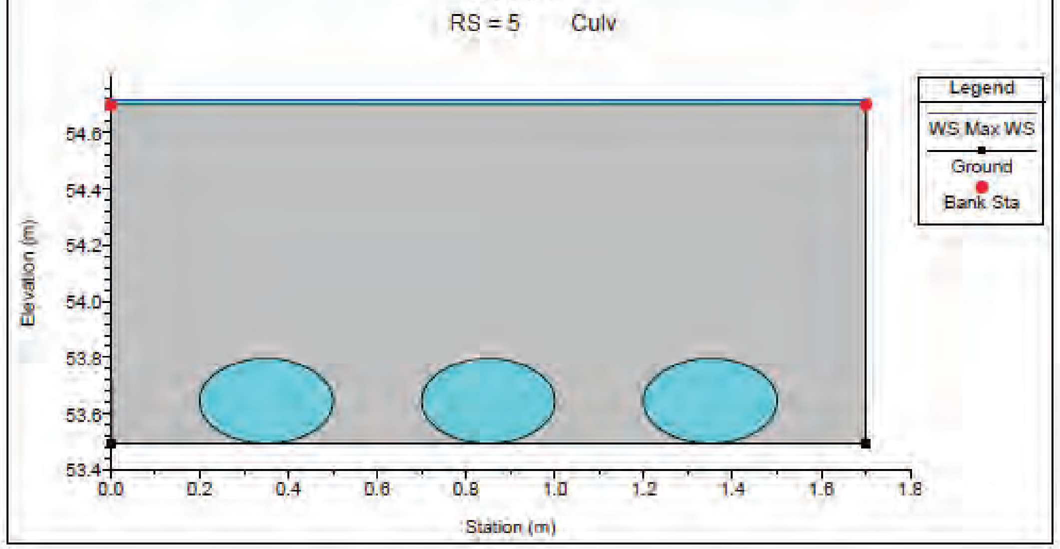

RS = 27
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)


RS = 25 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

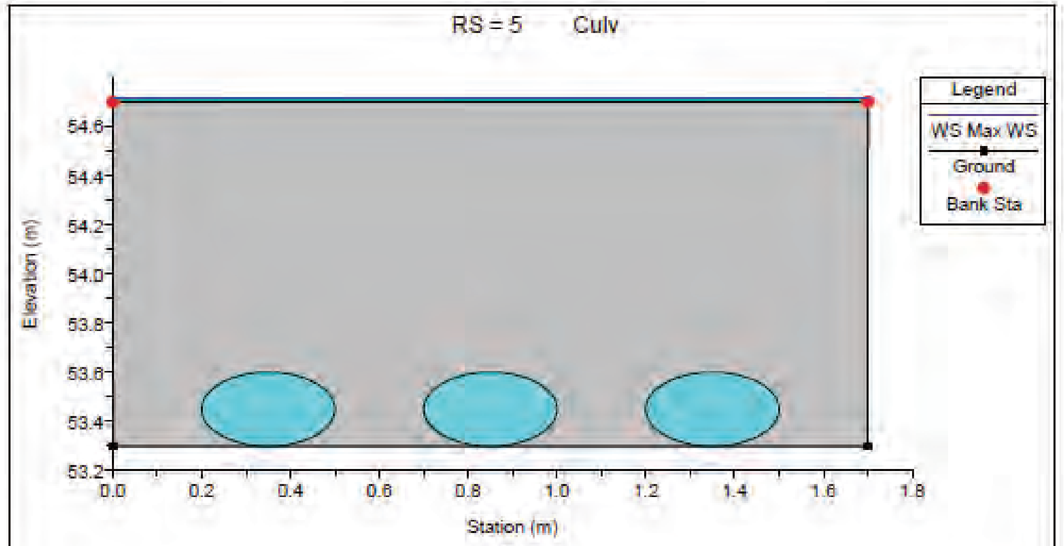

RS = 25 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

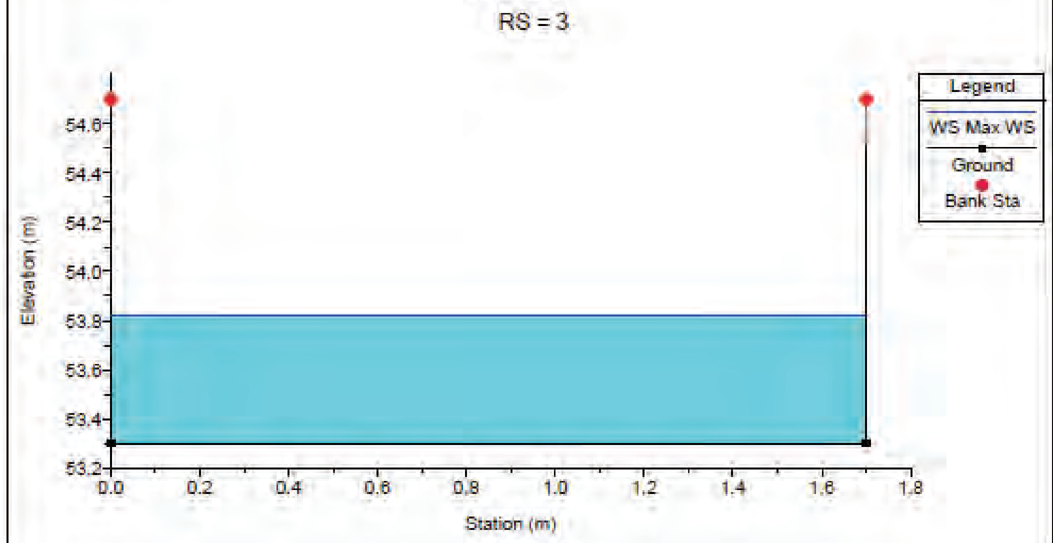

RS = 10
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

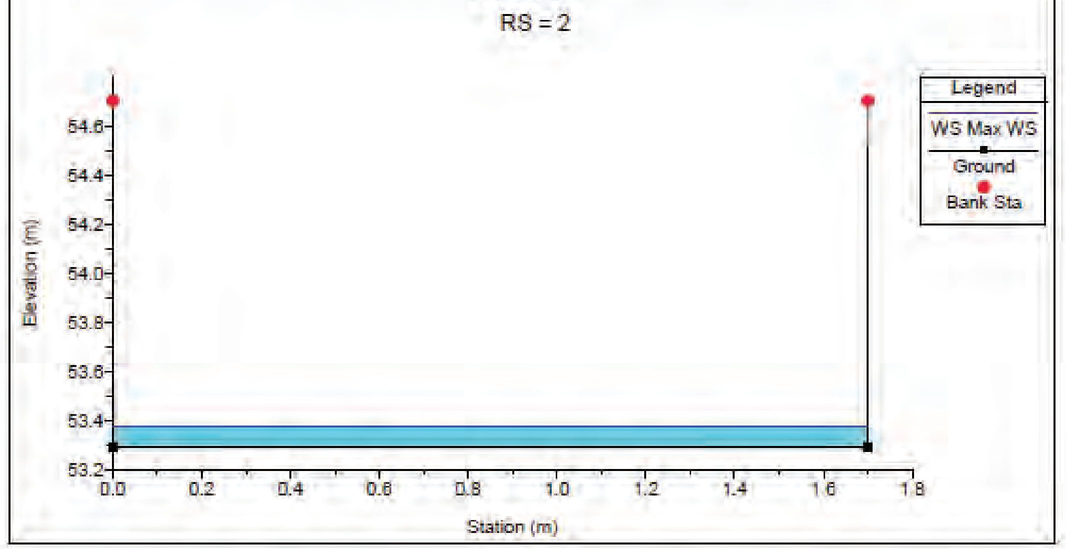

RS = 9
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)


RS = 5 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)




RS = 5 Culv
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)


RS = 3
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)


RS = 2
Legend
WS Max WS
Ground
Bank Sta
Elevation (m)
Station (m)

## REPRESENTATIVE – RATING CURVE