

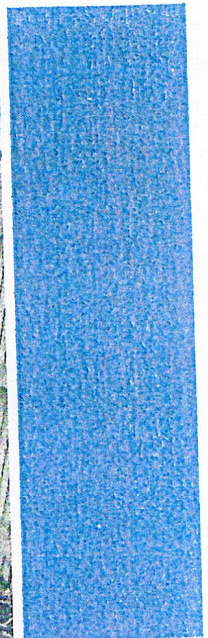


Kingdom Housing Association

Freuchie Mill Culvert

Assessment of Culvert Remedial Options

April 2009





Revision Schedule

Freuchie Mill Culvert – Assessment of Proposed Remediation Options

April 2009

S106008

Rev	Date	Details	Prepared by	Reviewed by	Approved by
01	April 2009	Draft	E. Wingate Senior Engineer	D. Wright Technical Director	
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Front Cover : Culvert Inlet and Gabion Headwall, South of the Site

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Figure 1 – Site Survey Plan and Modelled Cross Sections

Figure 2 – Catchment Plan

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- Selected Hydraulic output

1.0 Introduction

Scott Wilson Scotland Ltd was engaged by Kingdom Housing Association to undertake an analysis of catchment hydrology and investigation into the effectiveness of culvert remediation measures at the Freuchie Mill housing development, Freuchie, Fife.

Freuchie Mill Burn is culverted through the site via a 1200mm diameter corrugated steel culvert, which outlets at a point just south of Freuchie Mill Road. The watercourse then passes through a stone arch bridge, at Freuchie Mill Road.

Flooding of the Freuchie Mill site occurred on the 13th August 2008 and resulted in damage to residential properties, road and drainage infrastructure. Details are discussed in the report *Damage to Culvert Outlet Structure and Suggested Remedial Options for Kingdom Housing Association at Freuchie Mill, Freuchie, Fife (September 2008)*, which attributed the flooding to insufficient capacity of the culvert, and a rare rainfall event.

Residual damage to the culvert and outlet structure were reported as potentially increasing the likelihood of the housing estate experiencing further flooding. To reduce this risk, repairs to the existing culvert and outlet structure were proposed to reduce the flood risk to that which existed prior to the flood and damage.

An alternative approach is to undertake culvert remedial works that could further reduce the flood risk to the site, and the above-mentioned report (September 2008) identified four potential culvert remediation measures. Their effectiveness is assessed in this investigation against a series of design flood events. As a comparison, the capacity of the re-instated 'existing' culvert has also been investigated.

This report has been prepared for the purposes of assessing the proposed remedial options based on a simplified assessment methodology, and does not provide statements of fact on the existing culvert and site drainage, or on the August 13 event.

2.0 Background

2.1 Site Description

The site is described as Freuchie Mill, Freuchie, and is located within Fife Council area. The subject site has an area of approximately 0.6 hectares, and is bounded by Freuchie Mill Road to the north, Freuchie Mill to the East, rural-residential housing to the west, and undeveloped land to the south. Further details are provided in the 2008 report. Freuchie Mill Burn is culverted north through the site.

The subject site consists of residential development. The land beyond the east and western boundaries rises steeply away from the site. Ground levels around the extremities of the site typically vary between approximately 52 mAOD in the northwest corner near Freuchie Mill Road, to 48.7 mAOD on the southern boundary near the start of the culvert, and 49.6m AOD at the northern site entrance at Freuchie Mill Road.

The site has a depression at its centre, with a number of low-points at around 48 mAOD near the centre of the site, and along the eastern boundary. A site survey plan is provided in Figure 1.

2.2 Existing Culverts

Freuchie Mill Burn is culverted through the site via a 1200mm diameter corrugated steel culvert, which outlets at a point just south of Freuchie Mill Road. The watercourse then passes through a stone arch bridge beneath Freuchie Mill Road, with a span of 2.27m and a rise of 2.0m.

At the upstream end of the main culvert, a 300mm culvert discharges through the gabion headwall on its west side. Site design drawings indicate that this may have previously been fitted with a flap valve. It is not known whether this is the outlet to a small watercourse that drains the catchment west of the site.

A 530mm diameter pipe is located near the east side of the main culvert upstream headwall. Site design drawings indicate that this may be a bypass pipe installed during the original construction of the culvert. Near its downstream end, the bypass pipe changes to 800mm diameter pipe.

CCTV of the culverts was undertaken to confirm pipe origin, condition, and connectivity. The 1200mm main culvert was found to be damaged at a number of locations, with two locations where the pipe was deformed and may constrict the flow; however the extent of deformation on culvert conveyance could not be quantitatively determined. The bypass pipe was found to have no significant deformation that could restrict flow.

A number of other minor pipes (smaller than 150mm) were surveyed around the main culvert outlet and the stone arch bridge beneath Freuchie Mill Road. These pipes have not been considered further in this investigation.

2.3 Proposed Culvert Remediation Options

Four culvert remediation options are presented in the 2008 report. These are broadly summarised as follows, with further details provided in the earlier report:

- Option 1** Inlet and outlet improvement works, with internal bend reforming;
- Option 2** Inlet and outlet improvement works, with culvert duplication;
- Option 3** By-pass channel to east of site with new culvert;
- Option 4** Alterations to road levels and outlet improvement works to improve overland conveyance.

2.4 Data and Terminology

This assessment is based on information collected from the following sources:

- The report *Damage to Culvert Outlet Structure and Suggested Remedial Options for Kingdom Housing Association at Freuchie Mill, Freuchie, Fife (September 2008)*, including design drawings of the culvert;
- Ordnance Survey 1:25,000 Explorer map;
- Detailed topographic survey of the site, including the Freuchie Mill Burn;
- CCTV footage and report of main culverts;
- Advice from SEPA and Fife Council officers;
- SEPA rain gauge data;
- Photos of flooding and flood damage, provided by SEPA and residents;
- Site visit;
- The Flood Estimation Handbook (Institute of Hydrology); and
- The SUDS Manual, CIRIA C697 (2007).

Within this report, flood events are referred to in terms of their return period. A return period does not predict the period of time that will elapse between such events; rather it is an expression of the probability of that event occurring in any given year.

Table 1 summarises the return periods discussed in this report, and the probability of these flood events occurring in any given year. It should be noted that two or more events of any given probability can occur within any given year.

Table 1: Probability of Flood Events of Given Return Periods

Return Period	Probability of Occurrence In any Year
1 in 2 year (Q_2)	50 %
1 in 5 year (Q_5)	20 %
1 in 30 year (Q_{20})	3.3 %
1 in 100 year (Q_{100})	1 %
1 in 200 year (Q_{200})	0.5 %

Where the left and the right of the river are referred to in this report, these are in relation to an observer facing downstream.

2.5 Consultations

SEPA

The site is shown to be flood affected on SEPA's 1 in 200 year recurrence event indicative flood maps. It is noted that the SEPA Flood Map only shows an approximate 200-year flood envelope and does not necessarily include the impact of factors such as culverts and bridges on flood conveyance and storage.

SEPA provided daily, hourly and 15 minute intensity rainfall data from the SEPA gauge at Newton of Falkland, which is located only 2km west of the subject site, for a period over several days prior to the flood event at Freuchie Mill. SEPA advised that the data had not been fully quality controlled. A number of photos were provided, taken by SEPA as well as local residents.

Fife Council

Discussions were held with Council (Jim Moodie) regarding the event. A summary of discussions is provided below:

- Council do not have any relevant flood studies or flood risk assessments in the area that may be utilised for comparison.
- The culvert was designed and constructed in the early 1990's. It is likely that the culvert would have been designed for a less frequent event (such as the 30 year return period event) since this was the required standard at the time. There is no hydrology / hydraulic report for the original development that may be reviewed.
- The antecedent catchment conditions would likely have contributed significantly to flooding; prolonged rainfall over many days resulted in saturated soils, so when the heavier rain occurred that resulted in the flood event, almost all the rainfall resulted in run-off (rather than infiltrating into the ground).
- Remedial options should consider 'design events' for determining catchment hydrology.
- The Lomond Hills to the south are some of the highest in Fife and may have contributed to the rainfall characteristics.

Discussions were also held into repair options, culvert and site ownership, statutory responsibilities and future flood prevention schemes; however these issues have not been investigated further in this report.

Met Office

The Met Office was consulted to determine whether they hold rain gauge data useful for this investigation. The Met Office advised that they do not have any rain gauge located closer to the site than the SEPA rain gauge, and were only able to provide rainfall intensities estimated from radar imagery. This data is liable to error due to technical and meteorological causes, and was considered not to offer better data than the SEPA gauge. Then Met Office data was not obtained as part of this investigation.

3.0 Flood Flow Estimation

3.1 Catchment

3.1.1 Watershed Boundary and Catchment Description

Freuchie Mill Burn drains a predominantly agricultural catchment upstream of the subject site, with large areas of agricultural land and some small areas of plantation forest in the upper reaches. There are no significant storages or lochs in the catchment.

The approximate catchment boundary was provided in the 2008 report, shown in Appendix A of that report, and was stated to be approximately 10km² (as determined from 1: 50,000 OS mapping). The catchment boundary was shown to extend as far south as Balfarg and east to Star Moss. The approximate catchment boundary as previously reported has been reproduced in Figure 2.

The watershed of Freuchie Mill Burn was also determined using the FEH software. The FEH software indicates that the area of the catchment to the subject site is 5.7 km², and omits a portion of the catchment identified in the 2008 report that was centred on the Lochmuir wood region. This boundary is also shown in Figure 2.

For this investigation, the catchment boundary was also estimated based on the 1:25,000 OS map, as well as a site visit to ground-truth connectivity of drainage lines. It is noted that not all drainage lines were inspected. The catchment boundary is difficult to define due to the following:

- Large numbers of field drains in rural land, often against the natural fall of the topography.
- Culverts through the main residential area of Freuchie with unclear discharge locations.
- The presence of the railway within the catchment, including significant cuttings and embankments. A large number of drainage lines (anecdotally used for old steam trains) run within the rail land, and there are a number of aqueducts to convey watercourses across the railway when in cutting.
- Flat marshy areas in the south east, with undefined catchment boundaries. 1:25,000 OS mapping indicates that drainage lines between the railway and Lochmuir wood area appear to go in both directions, and as such may not represent the natural fall of these drainage channels.

For the purposes of this assessment, the catchment has been separated into a number of sub-catchments, each with distinct hydrologic features that would impact on peak flows during extreme events. The total area of the total catchment (including all sub-catchments) was estimated to be 10.1 km², and is shown on Figure 2. However, it is unlikely that total area would equally contribute to flows within Freuchie Mill Burn at the subject site during extreme events. The sub-catchments are described in further detail below.

3.1.2 Sub-catchments

- West Sub-catchment: The catchment that drains to the Site's west culvert is likely to originate in fields directly south of Freuchie, where a number of field drains were observed that flow into a culvert at Muir Road, Freuchie. The route of this culvert is not known. One possibility is that this culvert enters Freuchie Mill Burn downstream of the subject site; another possibility is that this culvert discharges beneath the A92 and into a series of open drains that lead to the western boundary of the subject site. The area of this sub-catchment was estimated as 0.44 km², as shown on Figure 2.
- South-West Sub-catchment: This sub-catchment originates to the south-west of the subject site, on the eastern slopes of the East Lomond hills. A number of field drains intersect runoff which is then conveyed beneath the A912 and the A92 near Forther Mill Farm. From here, runoff enters the main Freuchie Mill Burn (downstream of the aqueduct). The area of this sub-catchment was estimated as 1.59 km², as shown on Figure 2. This area would likely experience runoff similar to "greenfield" conditions, and has the least number of hydraulic controls.
- East Railway Sub-catchment: A drainage channel in the farm east of the subject site is thought to be culverted to a location on Freuchie Mill Burn just upstream of the subject site. Beneath the railway embankment, water is conveyed by a 500mm diameter culvert, with two smaller culverts of 100mm and 300mm (with no outlets surveyed). Runoff from the Nottingham area discharges to these culverts, with no other flow route possible. The area of this sub-catchment is 1.03 km², as shown on Figure 2. This area may be considered as a separate sub-catchment, and the maximum possible outflow limited by this combination of culverts.
- Wooded Storage Area: The watershed of the Starr Moss and Lochmuir Wood area is poorly defined. This flat, marshy area would likely behave in a similar way to that of a loch – providing considerable storage and attenuation of peak flows, with a relatively constant outflow. This area has a large number of drainage channels, or areas which drain directly to the rail embankment. The area of this sub-catchment is estimated to be around 4.39 km², as shown on Figure 2, and is approximately defined by the difference in catchment boundaries determined from the FEH software and that in the 2008 report. This area may be considered as a separate sub-catchment, with a constant low-outflow that combines with runoff from the remainder of the catchment.
- Aqueduct Sub-catchments: A number of aqueducts connect drainage paths across the railway cutting. The 2008 report states that these aqueducts have been recently upgraded. Each aqueduct has a maximum hydraulic capacity, which when exceeded, would be overtopped resulting in water spilling into the railway cutting. The most downstream aqueduct, south of Forthar Mill Farm, may therefore limit runoff at this location. It was estimated during the site visit that any aqueduct overflows would travel east towards the East Railway Sub-catchment (and be limited by the railway culvert capacity as discussed above). The area draining to this aqueduct, excluding the wooded storage area defined above, is estimated to be around 2.66 km² ha, as shown on Figure 2. This area may be considered as a separate sub-catchment, with peak out-flows limited by the aqueduct capacity, estimated to be 2.2 m³/s (based on an average slope).

It is noted that culverts, road crossings, and other control structures may further impact on the catchment hydrology, however these have not been considered further in this exercise.

Furthermore, the field drains within each sub-catchment may easily have their capacity exceeded, resulting in altered catchment hydrology site during extreme events.

Runoff from various parts of the total catchment may drain directly to the railway cutting, and may not then be discharged to the main Freuchie Mill Burn, rather, be held within the cutting. It is possible that these areas are significant, since a large portion of the catchment west of the A92 drains east towards the railway. Since these areas are difficult to define, and the impact of the railway on peak flows is now known, then these areas have been included in the sub-catchments identified above.

3.2 Hydrologic Model – Options and Development

The industry standard for the assessment of flood flows in the UK is the "Flood Estimation Handbook" (FEH), published by the Institute of Hydrology in 1999. Current methods of flood flow estimation recommended in the FEH include either / or a combination of the following methods:

- Statistical analysis procedures using gauged data. The FEH recommends this is the preferred method in most instances, however is generally only applicable to large catchments exceeding 1,000 km². Furthermore, no substantive gauged flow data is available for Freuchie Mill Burn;
- Rainfall-runoff based approaches using unit hydrograph models for smaller catchments that are more likely to experience a catchment-wide design storm. The FEH rainfall-runoff (R-R) procedures are relevant to catchments of a size larger than 200 ha (2 km²), and as such are likely to be appropriate for this present study.

Other methods are also available for estimating catchment hydrology, and may be used as a comparison to the FEH-Rainfall-runoff method. The IH 124 method with regional growth curves as described in CIRIA C697 (2007) is used for determining runoff from 'greenfield' catchments. UK growth curves were selected for Hydrometric Area 1 however the site is on the boundary of Area's 1 and 2. For 'design purposes' (that is, determining the peak flow for the catchment under natural catchment conditions), a SOIL index of 0.40 was adopted based on moderate infiltration (type S3) representing either very fine sands, silts and sedimentary clays; permeable soils (e.g. gravel, sand) with shallow groundwater in low lying areas; or mixed areas of permeable and impermeable soils in similar proportions.

Other rainfall-runoff models (e.g. SWMM, RORB, etc) may also be used but have not been considered in this present study.

As outlined in Section 3.1, the catchment may be considered as having a number of sub-catchments with independent runoff characteristics that contribute to the total flow at the subject site during extreme rainfall events. The following approaches have been considered:

- **Method 1:** The total main catchment contributes as a single catchment, with no consideration of hydraulic controls or storage areas. This represents a total area of 10.1km². This is considered to provide an over-estimate of the catchment area, and therefore an over-estimate of the peak runoff. This is included for comparative purposes only;

- **Method 2:** Catchment as defined by the FEH software. This is approximately equal to the total main catchment area, less the area that may provide storage in the Starr Moss and Lochmuir Wood sub-catchment, and is poorly defined. The catchment area is 5.7km^2 and is likely to provide a good approximation of the contributing catchment. This method does not consider possible hydraulic controls due to the railway culvert (East Railway Subcatchment) or the aqueducts, as discussed below.
- **Method 3:** Assessment of each sub-catchment separately and combination of peak flows as though they occur concurrently. This is conservative and does not allow for a reduction in peak flows through each catchment. This is essentially a 'revised' Option 2:
 - West Sub-catchment: with an area of 0.44 km^2 , the peak flow was estimated using the IH 124 method
 - South-West Sub-catchment: with an area of 1.59 km^2 , the peak flow was estimated using the IH 124 method. As a check, the West Sub-catchment and South-West Sub-catchment, with a combined area of 2.03 km^2 , was modelled using the FEH R-R (with area adjusted to equal sub-catchment area);
 - East Railway Sub-catchment: with an area of 1.03 km^2 . The peak flow would vary from a Q2 of $0.4\text{ m}^3/\text{s}$ to a Q200 of $1.1\text{ m}^3/\text{s}$, however would be limited by the capacity of culverts beneath the railway, estimated to vary up to $0.8\text{ m}^3/\text{s}$ with 1.5m surcharge. As such, the peak flow was therefore assumed to vary up to this limiting flow;
 - The total area of the Wooded Storage Area and Aqueduct Sub-catchments is 7.1 km^2 . The peak flow from this total catchment area would vary from $1.9\text{ m}^3/\text{s}$ in the 2 year event to $7.9\text{ m}^3/\text{s}$ in the 200 year plus climate change event. Such flows are considered unlikely to be generated due to influence of storages and the aqueducts. The following approach was used:
 - Aqueduct Sub-catchment: with an area of 2.66 km^2 . The peak flow would vary from a Q2 of $0.8\text{ m}^3/\text{s}$ to a Q200 of $2.5\text{ m}^3/\text{s}$.
 - Wooded Storage Area Sub-catchment: with an area of 4.39 km^2 . Any contributions from the Wooded Storage Area are likely to be in the form of low base-flows, arbitrarily estimated to add 20% to the flows from the Aqueduct Sub-catchment.
 - The combined flow was therefore varied from $1.0\text{ m}^3/\text{s}$ up to the limiting capacity of the aqueduct, estimated to be $2.2\text{ m}^3/\text{s}$.

3.3 Hydrological Model Results

Table 2 shows the results of the hydrological analyses. Table 2 defines the peak flow for 'design purposes' for assessing runoff using current practices.

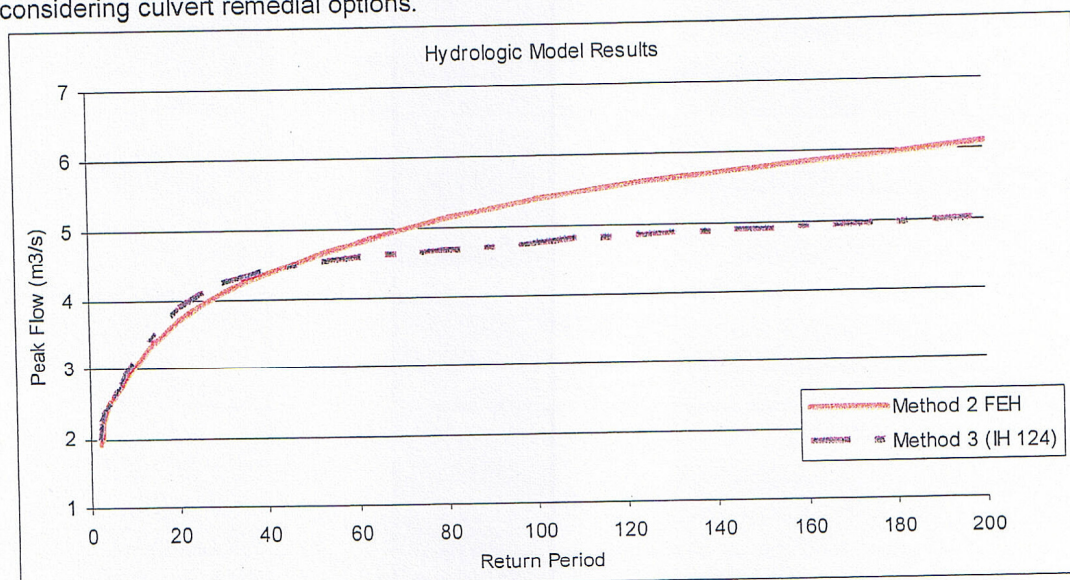
Table 2: Freuchie Mill Burn – Peak Flow Estimation (m^3/s)

Return Period	Method 1		Method 2		Method 3
	FEH	IH 124	FEH	IH 124	(IH 124)
Q2	3.3	2.7	1.9	1.6	2.0
Q5	4.6	3.5	2.6	2.2	2.6
Q30	7.3	5.8	4.1	3.5	4.2
Q100	9.5	7.3	5.4	4.4	4.8
Q200	10.9	8.3	6.1	5.0	5.0

Under current best-practice arrangements, an additional allowance for climate change of 20% would be included in the above flows.

The IH124 approach for Method 2, which was used for comparative purposes only, produces a lower peak runoff than the FEH R-R technique, and is not considered further.

Table 2 is summarised in the figure below, for Methods 2 and 3. Method 3 results in a flatter 'growth' curve, with peak flows limited by the aqueduct and rail culvert, and the apparent maximum flow of Method 3 is sensitive to catchment parameters. For Method 3, the hydraulic controls reduce peak flows by an estimated $0.6 \text{ m}^3/\text{s}$ in the 200 year event. It is likely that the peak flow at the sight would rarely exceed $6 \text{ m}^3/\text{s}$, and could be used when considering culvert remedial options.



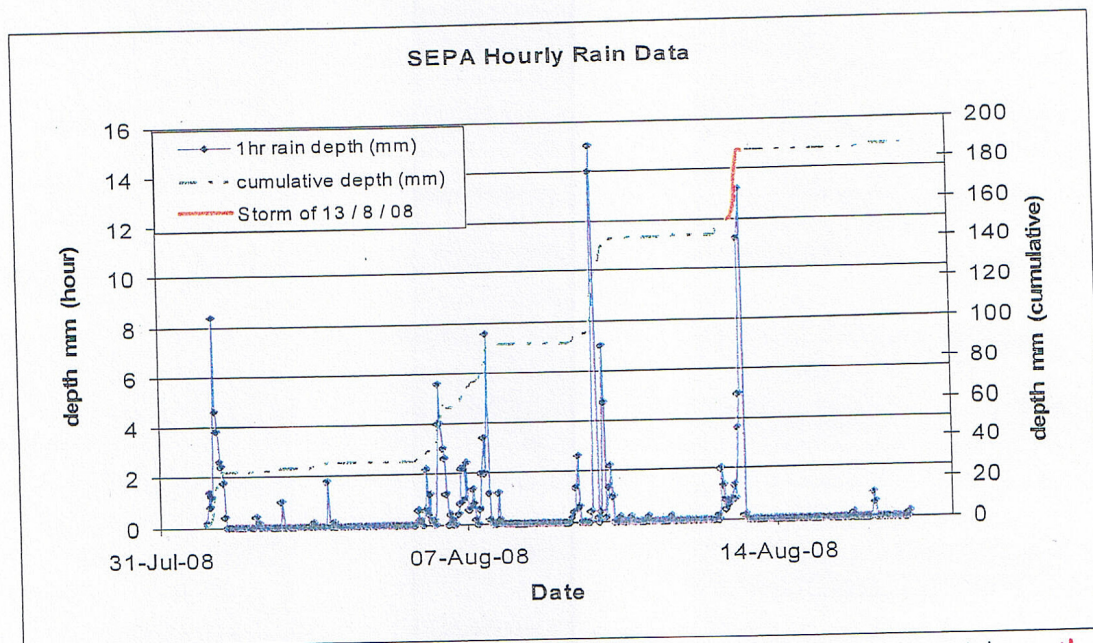
Flows in the catchment to the western culvert are included in the total catchment flows entering the site. Flows from the West Sub-catchment vary from a Q2 of $0.6 \text{ m}^3/\text{s}$ to a Q200 of $2.0 \text{ m}^3/\text{s}$, and as such represent a substantial portion of the total catchment flows. Further details on the western culvert are provided in Section 5.3.

3.4 Hydrologic Assessment of August 13 Event

3.4.1 Gauged Rain Data

The 2008 report stated that the rainfall recorded at Leuchars, Fife was 31mm on the 13 August 2008, and that Fife Council had stated that a gauge west of Cupar noted 60-70mm of rainfall for that day. For this present investigation, SEPA provided daily and hourly rainfall data (with some 15 minute rainfall data) from the SEPA gauge at Newton of Falkland (around 2 km west of the subject site), for the first 16 days in August, as well as daily rainfall totals over July and August.

The rainfall data for the first half of August is shown in the graph below. Note that the hourly rainfall data starts mid-way through a large storm centred on the 1st of August. Further discussion of this is provided below.



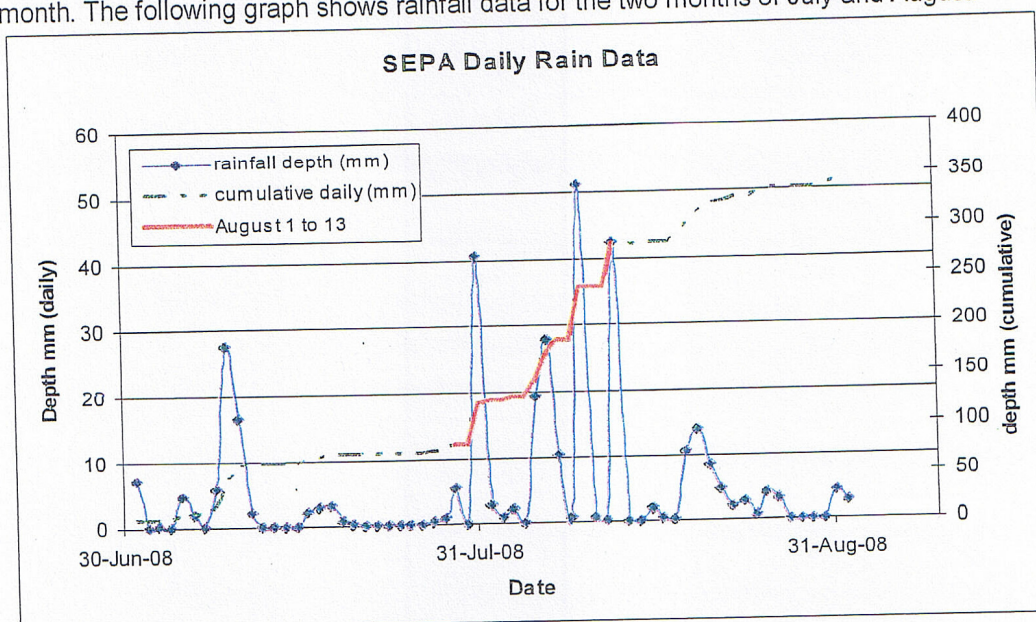
The cumulative rainfall over 13th August event is shown in red, and equates to a total rainfall depth of 33mm over a 4 hour period (preceded by 7mm over 8 hours). Within that 4 hour intense rainfall period, 24.4mm was recorded over a 2 hour period, and 14.2 mm over a 1 hour period. The greatest 15 minute rainfall intensity in this period was 4mm.

The Flood Estimation Handbook (FEH) software allows the estimation of the return period of a rainfall event that has been observed at a particular location based on depth, duration and grid reference. Rainfall frequency estimates are calculated using a model of rainfall depth-duration-frequency (DDF model) within the FEH software for a point within a catchment. A catchment-average design rainfall is required for flood design. The rainfall at a point location (the gauge) is required to be reduced by an areal reduction factor (ARF) to estimate the catchment rainfall. The ARF varies depending on the storm duration, however for a 2 hour storm, using the catchment area determined from the FEH CD software, the ARF was 0.944.

Design rainfall was generated using FEH software for the total catchment and compared to rainfall depths recorded on the 13th August, factored by the ARF:

- A point rainfall depth of 33mm over a 4 hour period is approximately equal to a 23 year catchment return period,
- A point rainfall depth of 24.4mm over a 2 hour period is approximately equal to a 19 year catchment return period,
- A point rainfall depth of 14.2 mm over a 1 hour period is approximately equal to a 8 year catchment return period.

The preceding graph demonstrates that the rainfall event of the 13th August was not an isolated incident during August, nor was it the most significant event to occur during that month. The following graph shows rainfall data for the two months of July and August.



Highlighted in red are four intense storm events that occurred in close succession: 41mm falling in 24 hours around the 1st August, 58mm falling over the 6th – 7th August, 51mm falling in 24 hours around the 9th August, and 33mm over the 13th August. Together, the total rainfall over this 14 day period was 199.6 mm. If the rainfall over this 14 day period is considered as one long event, comprising a number of intense periods, then comparison to design rainfall generated using FEH software for Freuchie, allows the following to be concluded:

- A point rainfall depth of 199mm over a 14 day period is approximately equal to a 190 – 200 year catchment return period event. The August 13 event then formed the last portion of this event. The impact of this on runoff is further discussed below.

The 2008 report noted that there had been significant rainfall in the time before the August 13 event, resulting in a loss of storage within the soils, and the graph above supports this. It is likely that the preceding rainfall would have resulted in gradual reduction in the infiltration capacity of the soil, and as such potentially a higher proportion of runoff during the August

13 event than if the 'design-approach' hydrologic techniques were applied (which consider standard antecedent conditions).

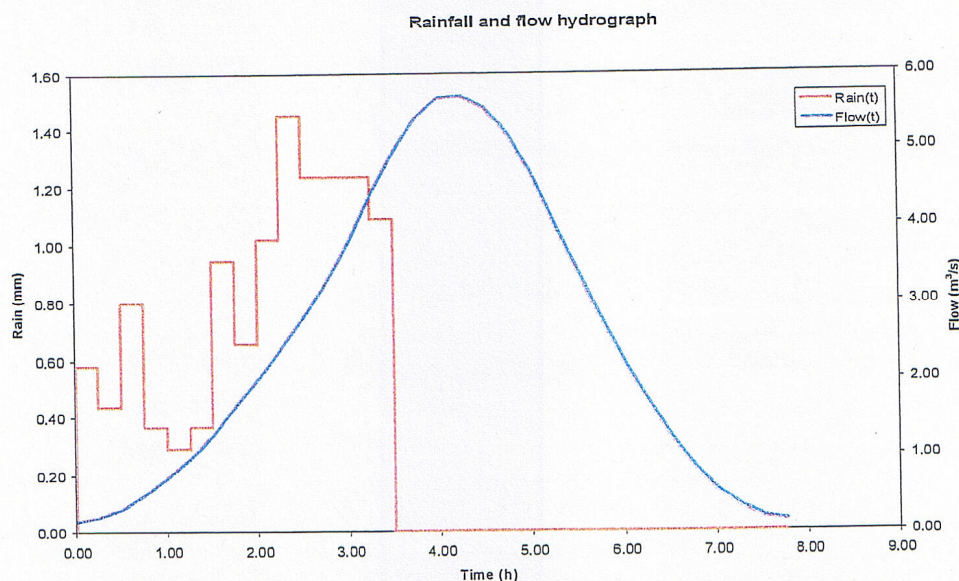
3.4.2 Hydrologic Assessment

Whilst the previous assessment indicates that rainfall considered over 14 days may be a very rare event, assessment of peak flows (runoff) needs to consider the individual August 13 rain event, since it is this rainfall that resulted in the peak flow and flooding.

The FEH Handbook (*Book 4, Fig 3.2*) provides a quick method for estimating the return period of a flood, based on the return period of the storm (rainfall) depth. Generally, the peak flood return period is around two thirds of the storm depth return period. However this method is considered inadequate due to the influence of antecedent soil conditions, and more accurate measures are recommended. If an assumption is made that the soils are saturated, and that the catchment acted as an "impervious (urban) catchment", then the peak flood return period would be equal to the storm depth return period (ratio of 1:1), providing an upper bound on the flood magnitude. Therefore, as above, the 33mm over a 4 hour storm period represents approximately a 30 year return period flood event.

Both the FEH and IH 124 techniques may be used to determine the hydrology (peak flows) that might be associated with the August 13 rain event. The following adjustments were made to account for the antecedent catchment conditions, rainfall and percentage runoff:

- **FEH:** This method is only relevant to Method 2 outlined earlier. Using the FEH techniques the following were determined, and using the rainfall data for the Newton of Falkland rain gauge: Catchment Wetness Index (CWI) of 130; Percentage Runoff (PR) of 35%; and Baseflow (BF) of 0.145. The rapid response hydrograph was then developed using the FEH techniques and spreadsheet, applied to the net rainfall. The total runoff hydrograph determined is shown below for Method 2.



The peak runoff for Method 2 was estimated to be $5.7 \text{ m}^3/\text{s}$. When compared to the design event peak flows reported in Table 2, the August 13 event would have a return period approximately equal to a 130 year event.

When developing the net total runoff hydrograph, the Time to Peak (TP) has a major influence on the peak flow determined. It was estimated that a time to peak of 1.5 hours occurred during the August 13 event (as opposed to a TP of 3.5 hours calculated using the FEH R-R method). This is considered reasonable due to the antecedent catchment conditions.

IH 124: This approach is relevant to Method 3 as outlined earlier. A SOIL index of 0.50, representing very high runoff, was adopted, and is usually used to describe soils of the wet uplands: bare rocks or cliffs; shallow, permeable rocky soils on steep slopes; or peats with impermeable layers at shallow depth based on moderate infiltration.

The antecedent conditions in the catchment were adjusted to reflect soil saturation. Table 3 shows the results of the hydrological analysis for a range of return intervals.

Table 3: Saturated Catchment – Peak Flow Estimation: Method 3

Method 3 (IH124)			
"Design" Return Period (Table 2)	"Design" Flow (Table 2)	Saturated Catchment Flow (m^3/s) (Soil = 0.50)	Estimated Equivalent "Design" Return Period
Q2	2.0	3.1	10
Q5	2.6	4.2	30
Q30	4.2	5.2	> 200
Q100	4.8	5.8	>> 200
Q200	5.0	6.2	>>> 200

Table 3 suggests that peak flows are around 20 – 60 % greater due to the antecedent soil conditions, attributable mainly to runoff from the West Sub-catchment.

The culverts and aqueducts control (limit) the peak flow, therefore the saturated catchment flows determined above exceed the range of design return periods reported in Table 2. This highlights the complexity of determining the return period and catchment hydrology using Method 3. However Table 2 and Table 3 indicate that peak flows would rarely exceed $6 \text{ m}^3/\text{s}$ due to the aqueduct and rail culvert hydraulic controls.

The saturated catchment peak flows may be input into the hydraulic model in order to assess the August 13 event. Further details are provided below.

4.0 Hydraulic Model Development

A hydraulic model of the Freuchie Mill Burn and culvert was developed to determine capacity of the culvert as well as flood levels across the site. Both the Culvert Master and HEC-RAS software packages were chosen to develop the model, and to check their results for the complex flow behaviour arising from split overland and culvert flows. The HEC-RAS model was set-up to allow assessment of remedial options, as discussed in Section 5.

In general, the flow regime in the burn at the subject site is one-dimensional with little off-line flood storage. Therefore, the model was run as a steady state analysis, providing conservative flood levels over the site.

The hydraulic assessment has focused on the main culvert. Consideration has been given to the smaller western catchment and culvert, however details of culvert outlet were not available, therefore the western culvert could not be assessed in detailed.

Culvert deformations were not explicitly modelled since their dimensions could not be established from CCTV footage. More detailed modelling may be carried out at a later stage to ascertain the full impact of these minor pipe constrictions.

4.1 Hydraulic Parameters

4.1.1 HEC-RAS Parameters

The Manning's Equation is used in HEC-RAS to estimate flow depths in river reaches assuming normal flow conditions. Manning's 'n' values are used in this equation to reflect the roughness of the flow channel. Large Manning's 'n' values imply higher degrees of roughness and consequently higher flood levels. Manning's 'n' values selected for this hydraulic study varied as follows:

- 0.065 for the defined watercourse channel upstream and downstream of the site, that was observed to be relatively straight, with some trees and shrubs along the banks of the watercourse;
- 0.05 for floodplain regions with generally long grass cover with some low shrubs;
- 0.024 for the culvert with corrugations and bends, and allowing for some additional losses due to minor culvert deformations;
- 0.03 for overland flow through the subject site, for paved surface, short grass and some minor obstructions including cars and fences.

These values are within the range recommended in the HEC-RAS technical reference manual as well as in other standard references. A sensitivity analysis of parameters was not undertaken as part of this assessment.

The boundary of the model was extended approximately 120m upstream and 120m downstream of the site boundaries using surveyed cross sections. Model sections are shown on Figure 1. Both the upstream and downstream boundary conditions were set to normal depth using a conservative energy slope of 0.5%, compared to typical slope of Freuchie Mill Burn upstream and downstream of the site of 1.3 %. A mixed subcritical and

supercritical flow regime was adopted for hydraulic modelling purposes to account for both backwater effects as well as possible supercritical conditions through the hydraulic controls and down the steeper portions of the watercourse.

Contraction and expansion coefficients of 0.1 and 0.3 were used for gradual transitions and 0.3 and 0.5 for rapid transitions either side of the culvert. Ineffective flow areas were included in the model to represent inactive flow on either side of culvert embankments. The energy method was adopted for the road embankments and culvert, and the standard (default) weir coefficient of 1.44 was adopted for overtopping of embankments.

In large flows beyond the capacity of the culvert, flood water overtops the upstream headwall gabions and flows into the site. Photos of the August 13 event show this flood behaviour. Slow-moving / ponded water is observed across the site, with 'weir flow' as water is discharged towards the stone-arch bridge beneath Freuchie Mill Road. Critical to determining the flood event magnitude is ensuring that the weir levels are correctly modelled, since these control water levels over the entire site. Additional cross sections were included in the model using data extracted from site topographic survey.

A lateral weir was modelled upstream of the site to accommodate overland flow entering the site. Adjustments to the weir coefficient were found only to impact on flood levels in the burn upstream, and not on flood levels through the site, with little or no impact on culvert capacity. This is to be expected with a backwater model. The model stability was, however, sensitive to this parameter.

4.1.2 Culvert Master – Parameters

A check of the HEC-RAS model was performed using Culvertmaster software. The culvert details were entered as surveyed with a Manning's 'n' for the culvert of 0.024, and a maximum allowable headwater depth equal to top of the gabions, approximately 48.7mAOD. The headwater required to convey a range of flows was also determined.

The culvert length and friction losses mean that the culvert is subject to outlet control. That is, increasing the water level on the upstream side will generally not significantly increase discharge through the culvert, as flows are limited by pipe-flow losses. Since the culvert is outlet controlled, tailwater levels may impact on the ability of the culvert to convey flows. The tailwater was varied from zero (freely discharging) to a water level equal to the culvert obvert.

4.2 Hydraulic Modelling

4.2.1 Culvert Assessment

The HEC-RAS model calculations indicate that the existing culvert capacity is approximately 2.8 m³/s (under normal tailwater conditions). The culvert capacity may be lower in large flood events when the stone arch bridge influences outflows and high tailwater conditions prevent efficient culvert operation.

Culvertmaster was used as a check of the HEC-RAS model. The capacity was found to vary from 2.7 – 2.9 m³/s depending on tailwater conditions. This analysis indicates that the HEC-RAS model culvert hydraulic assessment is suitable.

The capacity of the culvert may be expressed as a function of the design return period. Based on the above flows, and comparison to Table 2, the capacity of the culvert is approximately equal to a 7 year return period “design” flood event for Methods 2 and 3.

4.2.2 HEC-RAS Model Results

The peak ‘design event’ flows determined from Methods 2 and 3 (Table 2) were modelled. The flood behaviour over the site, as indicated in the photos of the August 13 event, was adequately replicated in the HEC-RAS model. Flood levels for the design event are summarised in Table 4 below, and are reported as a water level over the “entire site”, since it experiences an essentially flat water level before discharging over the ‘weir’ to the stone-arch under Freuchie Mill Road. Selected model output is attached.

Table 4: Design Event – Hydraulic Model Results: Site Water Level (mAOD)

Return Period (Design Event)	Method 2 * (Design Event)	Method 3 * (Design Event)
Q2	n/a (culvert flow only)	n/a (culvert flow only)
Q5	n/a (culvert flow only)	n/a (culvert flow only)
Q30	49.13	49.16
Q100	49.36	49.27
Q200	49.42	49.31

The additional “by-pass culvert” capacity of 0.4 m³/s not included, and has an impact of around 0.05m in the 200 year event.

Table 4 indicates that water levels would be around 49.3 – 49.4 mAOD in the 200 year event (excluding any allowance for climate change).

To achieve a water level of 49.4 mAOD requires a flow of around 6 m³/s. This indicates a flood equivalent to a ‘design event’ with a 200 year return period (Method 2) or greater (Method 3). However at this depth, and with slow-moving flow over the site, variations in flow (such as using the 100 year flow rather than the 200 year flow) results in only minor changes in flood water levels (of around 0.05 m). Since the modelled water level is relatively insensitive to the flow, it is difficult to determine the return period for a particular flood level.

It is noted that modelled flood levels may be higher or lower depending on model parameters (such as Manning’s ‘n’) and definition of “weir level” cross section data. Hydraulic modelling did not consider culvert blockage. Water levels through the site could increase significantly depending on the degree of blockage and its location.

'High tide' marks during the August 13 flood event were not surveyed, however an estimate of these levels can be made using photographs. It is estimated that the maximum water level was approximately 300 mm above the floor level of the houses in the south east of the site, indicating a water level of approximately 49.4 mAOD. This would appear to be consistent with photographs of the car-park with water depths estimated to be around 1m above ground levels (48.5 mAOD), as well as photos near the culvert outlet in the north of the site. The culvert would have been conveying just under half this flow, with the remainder passing through the site as overland flow and trapped by the higher ground between the northern buildings and access track and car-park, with minimum weir point levels at around 48.6 m AOD.

As detailed above, a water level of 49.4 mAOD requires a flow of around $6 \text{ m}^3/\text{s}$, equivalent to a 'design event' flood with a 200 year return period (Method 2) or greater (Method 3). Blockage of the culvert was not considered. Photos of the August 13 event suggest some blockage of the stone-arch at Freuchie Mill Road may have occurred, however the photos indicated that "weir-flow" occurred towards the stone-arch – as such, partial blockage at this location would generally have not impacted on water levels through the site. Blockage of the main culvert at its upstream end may have occurred but was not reported in the 2008 report, and as such as was not modelled. If this had occurred, the observed water level of around 49.4mAOD would have resulted from smaller flows and a more frequent flood event.

The hydraulic model was also used with the August 13 event estimated peak flow of $5.7 \text{ m}^3/\text{s}$ for Method 2. The model indicated a water level of 49.38 mAOD. This is broadly consistent with observed water level, estimated to be 49.4 mAOD. This suggests that the water level during the August 13 event (excluding blockages) would have approximately been the result of a 200 year return period flood event.

Assessing the August 13 event using Method 3 suggests that to achieve a water level of 49.4 mAOD would require a flow much rarer than the 200 year design event (estimated to be closer to the 500 year event). Method 3 results in a 'flat' growth curve (refer Section 3.3), with peak flows limited by the aqueduct and rail culvert controls – however the graphed of flow and return period is sensitive to the selection of catchment parameters. It is unlikely that the return period determined using Method 3 represents the August 13 event, since this flood return period is not consistent with rainfall observations.

4.3 Other Culverts

4.3.1 Western Culvert

The flow arriving at the western culvert (from the West Sub-catchment) would vary from a Q2 of $0.6 \text{ m}^3/\text{s}$ to a Q200 of $2.0 \text{ m}^3/\text{s}$. The 120mm pipe with grate would not be able to provide significant flow capacity – a pipe of such dimensions would typically be able to convey around 50 L/s ($0.05 \text{ m}^3/\text{s}$) or less. Furthermore, the small grated inlet could block.

During most storm events, runoff from the western catchment would likely be conveyed into the site as overland flow. Furthermore, the western culvert outlet location is unknown. For hydraulic modelling, flows from the western catchment entered the subject site as overland flow, joining the main overland flow route through the site.

4.3.2 By-pass Culvert

The by-pass culvert was not included in the HEC-RAS hydraulic model, however was estimated to convey around $0.4 \text{ m}^3/\text{s}$ when the water level is at the top of the inlet headwall gabions (based on Culvertmaster calculations). This pipe has a much higher invert level at the inlet than the main culvert.

Whilst not significantly changing the return period capacity of the culvert system, this pipe would provide some additional system capacity, conveying an additional 15% to the flow in the main culvert. The capacity of the combined culvert system is therefore approximately equal to a 12 year design return period for Methods 2 and 3. In larger events, the additional $0.4 \text{ m}^3/\text{s}$ becomes more significant – for example, representing the additional runoff between the 150 year and 200 year events (Method 3).

4.3.3 Stone Arch Bridge

The stone-arch bridge was included in the HEC-RAS hydraulic model, and has a capacity of around $7.6 \text{ m}^3/\text{s}$, assuming a headwater level equal to the arch soffit (47.8m AOD). Allowing some surcharging, up to an arbitrary level of 48.2 mAOD (200mm below floor levels), would allow around $10 \text{ m}^3/\text{s}$ to be discharged through the arch.

5.0 Assessment of Options

5.1 Option Overview

As outlined in Section 2.3, the four culvert remediation options proposed in the 2008 report are broadly described as follows:

- Option 1** Inlet and outlet improvement works, with internal bend reforming;
- Option 2** Inlet and outlet improvement works, with culvert duplication;
- Option 3** By-pass channel to east of site with new culvert;
- Option 4** Alterations to road levels and outlet improvement works to improve overland conveyance.

The hydraulic models were amended to allow assessment of each remedial option.

5.2 Betterment of Culvert Remedial Options

5.2.1 Option 1

Undertaking inlet and outlet improvement works would impact on the inlet and outlet “losses” of the culvert. It is however noted that the culvert is “outlet controlled”, and as such its capacity is predominantly a function of pipe length and friction losses, rather than inlet or outlet losses.

A hydraulic analysis of this option was performed using both the HEC-RAS and Culvertmaster models. The extent of inlet and outlet losses was varied, with a maximum improvement in culvert total capacity estimated to be around 5-10% (culvert capacity increasing to around 3.2 m³/s), depending on losses modelled. This generally does not result in a significant change in the return period capacity of the culvert: increasing to approximately a 12 year design event for the main culvert (or to a 17 year event for the combined culvert system). This option does not significantly improve the flood risk to the site.

The existing culvert deformations may reduce the culvert capacity further than that modelled. Remediation of these deformations would reduce these existing flow constrictions, and removing the sharp internal bends, as proposed for Option 1, may add a similar minor improvement, but would not significantly change the return period capacity of the culvert.

The capacity of the stone-arch bridge exceeds that of the culvert through the site, therefore lengthening the distance between these structures or providing stilling basins would have little effect. Water jetting from the culvert outlet should be allowed to be conveyed beneath the Freuchie Mill road bridge before slowing down on the northern side of the road and away from the site.

Damage near the culvert outlet during the August 13 event, would likely have occurred from overland flows scouring the downstream face of the site. Rip-rap or other scour protection on this downstream face would assist in preventing future scour from overland flows.

5.2.2 Option 2

Provision of a duplicate culvert would effectively double the capacity of the combined culvert system, i.e. the two culverts would convey around $5.6 \text{ m}^3/\text{s}$, which is equal to a 120 year return period event (Method 2). It was assumed that culvert duplication would require the removal of the bypass culvert, therefore its capacity has not been included.

It is noted that this option need not be limited to duplicating the existing culvert, and alternative culvert sizes could be selected. For example, a 2.1m diameter pipe at a reduced slope of 0.5% would convey around $9 \text{ m}^3/\text{s}$ and would still maintain around 600mm cover at the centre of the site. Providing a 1800mm x 1500mm box culvert, would pass around $6 \text{ m}^3/\text{s}$, and this could be used to either replace the existing culvert, or be constructed in parallel, thereby providing a combined capacity of around $9 \text{ m}^3/\text{s}$, which is significantly greater than the 200 year return period event.

A number of sub-options could therefore be investigated further, including the provision of a new culvert to the rear of properties to the east, thereby potentially avoiding existing services.

Inlet and outlet structure improvement works as well as bend reformation are unlikely to have a significant impact, as outlined for Option 1. However, such works may be necessary to accommodate the duplicate culvert.

The capacity of the stone-arch bridge exceeds that which would be conveyed through the duplicate culverts. As discussed previously, culvert flows should be conveyed efficiently beneath the stone-bridge to reduce losses.

5.2.3 Option 3

This option involves a new by-pass channel near the eastern boundary of the site. Option 3 is shown as a channel with flood protection wall on the western side. At this feasibility stage, channel dimensions have not been proposed, therefore the following was assumed:

- The channel was assumed to have a 1m minimum offset from existing buildings, leaving a total available width of 2m. This could be widened if the adjoining land was acquired.
- Based on the above, and on the option schematic shown in the 2008 report, the channel was modelled as having a 1m wide base,
- The right hand side was assumed to be a 1 vertical to 1 horizontal (typical) stone wall, matching into a ground level of around 48.0 m AOD, which is approximately equal to minimum existing ground levels along the eastern boundary.
- A vertical flood control wall has been included on the left hand side, with a maximum level of around 49m, approximately equal to the adjoining existing floor levels, and up to 1.0m above the existing ground level. The legislative implication of constructing new flood defences may need to be considered.

- It was assumed that the flood relief channel would be constructed with an invert level of 47 mAOD at the upstream end, decreasing to 46.75 mAOD at the start of the proposed new culvert.
- The new 2.1m diameter culvert passes beneath the access road, discharging just upstream of the stone-arch bridge at Freuchie Mill Road.
- The existing culvert was assumed to function normally.

Option 3 would likely impact on (or result in removal of) the bypass culvert, therefore its capacity has not been included.

With the water level near ground level (48m AOD) mid-way along the flood relief channel (or a slightly higher water level of 48.2 mAOD at the start of the channel), the channel would be able to convey around 2.4 m³/s. This represents a combined flow for the channel and existing culvert, of around 5 m³/s. This is approximately equal to a 75 year return period event (design event, Method 2).

Allowing 500mm freeboard (ie. a water level of around 48.5 mAOD at the start of the flood relief channel) the channel would convey approximately 4.3 m³/s. This represents a combined flow for the channel and existing culvert of around 6.8 m³/s. This is in excess of the 200 year return period event (design event, Method 2).

Downstream of the channel, the channel would discharge to a new 2.1m diameter culvert beneath the access road. The culvert was included in the model with a total length of 17m, an upstream invert level of 46.75mAOD, and a downstream invert level of 46.5 mAOD. The capacity would be around 9 – 10 m³/s and significantly exceeds the capacity of the flood relief channel. Consideration could be given to varying channel and culvert dimensions to maximize the efficiency of this option.

Whilst Option 3 is shown as a channel with flood protection wall, alternative designs may prove to be hydraulically similar and less expensive, although may have a different land-take. Route variations may also be possible. It is noted that any overflows from the channel would still spill into the site, and be conveyed as per the existing scenario. Some earthworks would be required to raise ground levels around the existing upstream culvert headwall gabions, to prevent water entering the site, and direct flow into the new channel.

5.2.4 Option 4

Undertaking site works to provide flood relief as shown in the 2008 report, has been examined using the hydraulic models.

Option 4 proposes a new culvert beneath Freuchie Mill Road, replacing the existing stone-arch culvert. The existing bridge has a capacity of around 7.6 m³/s, with a water level equal to the arch soffit, or 10 m³/s with surcharging at, say, 48.2 mAOD (200mm below floor levels). It is recommended that Option 4 be revised to exclude extensive road or culvert works along Freuchie Mill Road, however these works would add additional certainty to this Option.

For the purposes of this assessment it was assumed that re-grading of the site access road would occur from Section S-5 northwards to Freuchie Mill Road. It was assumed that ground levels would be lowered to 48.2m AOD, which is around the existing lowest level in

the centre of the site. The result is a lowering of the "weir level" by around 0.5m. Flood levels in this scenario would vary from around 48.4 mAOD in a 30 year event (approximately equal to floor levels in the north west of the site) to 48.6 mAOD in a 200 year event (approximately 400mm below floor levels over the majority of the site, but 200mm inundation of floors in the north west of the site).

Higher water levels would occur near the southern end of the site where existing ground levels are around 48.5m AOD. Consideration may also need to be given to earthworks in this area. Further reductions in flood levels could be achieved through more extensive re-grading (lowering) of site levels, but would be dependent on detailed investigations and design of the works.

5.3 Summary

A summary of the betterment of each option is provided in Table 5, with return periods stated for the Method 2 'design' event.

Table 5: Summary of Options

Option	Return Period at which site begins to flood *	Return Period at which water reaches existing floor levels (48.4 mAOD) *	Comment
1	12	approx. 15	Governed by existing 'weir' levels, which would cause inundation in the north-west of the site . Additional re-grade works recommended to alleviate this.
2	120	approx. 120	
3	75 to >200	>200	>200 assuming flood wall not overtopped.
4	12	30	Further design proposals required to fully assess this option. Greater protection possible if re-graded to a lower level (48.2m AOD assumed)

* By-pass culvert capacity not included

The low-point of the site is located near the centre of the site, and is at around 48 m AOD. If re-grade works were undertaken from this area towards the north to lower the existing "weir level", (currently at around 48.5 - 48.7mAOD), then overland flows could be readily conveyed towards the stone-arch bridge. It is likely that the depth of potential ponding over the site in the August 13 event would have been reduced by around 0.9m.

It is therefore likely that a combination of options would be required. It is also noted that variations in the four options have been recommended in the preceding sections, and should be considered so as to provide an optimal solution.

In all instances, provision of an overland path through the site is recommended for the western culvert inlet towards the main culvert outlet (near the stone arch bridge). Re-grading as per Option 4 would also assist should the main culvert become blocked.

The flood risk to the site will change based on whether culvert blockage is considered. Culvert blockage was not been included in the modelling, however should be an important consideration when choosing a remedial option. All four options rely on culverts that could potentially block, however in general, the risk decreases with each option considered:

- Option 4 provides the greatest overland relief in the event of culvert blockage. Whilst the existing culvert may block, earthworks to a lower ground levels, preferably below those modeled in this report, could result in a significant reduction in flood risk to houses.
- Option 3 provides a significantly reduced flood risk from culvert blockage since the large dimensions of the proposed culvert beneath the access road mean blockage is less likely.
- Option 2 effectively halves the risk of blockage, however does not provide any additional overland flow relief.
- Option 1 provides limited safeguard against culvert blockage.

Further consideration to these issues should be given when formulating a preferred remedial option.

6.0 Conclusions

Scott Wilson Scotland Ltd was engaged by Kingdom Housing Association to undertake an analysis of catchment hydrology and investigation into the effectiveness of culvert remediation measures at the Freuchie Mill housing development, Freuchie, Fife.

Flooding of the Freuchie Mill site occurred on the 13th August 2008 and resulted in damage to residential properties, road and drainage infrastructure. Details are discussed in the report *Damage to Culvert Outlet Structure and Suggested Remedial Options for Kingdom Housing Association at Freuchie Mill, Freuchie, Fife (September 2008)*. That report identified four potential culvert remediation measures, and their effectiveness was assessed in this investigation against a series of design flood events as well as the August flood event.

The following conclusions were determined:

- The culvert has a capacity of approximately $2.8 \text{ m}^3/\text{s}$. This is approximately equal to a 5 – 10 return period flood event, excluding the capacity of the by-pass culvert (of around $0.4 \text{ m}^3/\text{s}$).
- The August 13 storm rainfall would have been approximately a 20 – 25 year return period event. However, the antecedent soil conditions (ie., the high and sustained runoff in the preceding days and weeks) would have caused a higher peak runoff than indicated by the rainfall alone. The hydrologic modelling carried out in this study indicates that the peak flow during this event was approximately equivalent to a 130 year return period flood event.
- The maximum water level during the August 13 event is estimated to have been 49.4mAOD. Hydraulic modeling indicates that a flow of around $6 \text{ m}^3/\text{s}$ would be required to reach this water level. This would be approximately equal to a 200 year return period flood event. Culvert blockages may influence this.
- The August 13 event was therefore likely to have been between a 130 to 200 year return period flood event (significantly greater than the return period for the storm rainfall due to the particular antecedent soil conditions).

In assessing the betterment of the four proposed remedial options, the options may be ranked in the following order from greatest to least improvement: Option 3 – 2 – 4 – 1. Details of proposals were not available at this stage, and the betterment of each option may be significantly altered once detailed design proposals are available. Variations on each option are possible, and a combination of options may provide the most robust solution. It is recommended that Options 2 and 3 be further refined in tandem with detailed flow modelling, in order to achieve the most cost effective flood management solution.

Consideration should also be given to culvert blockage when selecting the preferred option. Re-grading of the internal access road as per Option 4, so as to provide an overland flood path for the main Freuchie Mill Burn flows, would assist should the main culvert become blocked.

In all instances, provision of an overland flow path through the site is recommended from the western culvert inlet towards the main culvert outlet (near the stone arch bridge).

Figure 1 – Site Survey Plan and Modelled Cross Sections

Figure 2 – Catchment Plan

Attachments

- Selected Hydraulic output