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SURFACE WATER MANAGEMENT PLAN UPDATE AND FOUL DRAINAGE STRATEGY FOR PROPOSED DEVELOPMENT AT LINGEN NURSERY, LINGEN, HEREFORDSHIRE, SY7 0DY

Dear Graham,

Thank you instructing Hydro-Logic Services International (HLSI) to update the surface water management plan (SWMP) and foul drainage strategy for the proposed development at the above location.

Our findings for the latest proposals may be summarised as follows.

1) The proposed development comprises eight residential properties, each with gardens, garages and open parking. The proposals include a new access road running from the south-western corner of the site.
2) The area of the site enclosed by the planning boundary is approximately 7,570 m2. The impermeable surface would include the proposed main access road, car parks and the combined roof area of the proposed buildings. The total proposed impermeable surface area is approximately 1,847 m2, representing approximately 24.4% of the site.
3) Based on the Environment Agency flood map for planning, the proposed development is located predominantly within Flood Zone 1, outside the limits of the 1 in 1,000 year fluvial flood event, with small areas of Flood Zone 2 and 3 adjacent to the watercourse at the eastern boundary of the site. The proposed development is classified as *More Vulnerable* and would be appropriate in Flood Zone 1. Neither the Sequential Test nor the Exception Test is required for this assessment.
4) The anticipated increase in peak river flows due to climate change could increase the risk of fluvial flooding at the site. The published Environment Agency flood maps do not include, and not are they amenable to, an allowance for climate change. Accordingly, in order to develop maps that account for the anticipated effects of climate change, hydrological analysis and hydraulic modelling has been undertaken at the site and are summarised in Appendix D and E.
5) Appendix E presents modelled flood levels for a range of storm scenarios, a model sensitivity test and a blockage scenario. The flood levels may be used for setting of finished floor levels of the proposed residential development.
6) Flood maps for 100yr+35%CC and 1,000yr storms have been developed and are presented in Appendix E. The flood maps demonstrate that the site is located substantially in Flood Zone 1. A sequential approach to land use would ensure the site is largely available for residential development.

7) The risk of surface water flooding is generally very low, with a small area of medium and high risk at the centre of the site. No flow routes are indicated at the site.

8) Soil mapping indicates that the soil beneath the whole site is clayey and slowly permeable.

9) Infiltration tests were carried out at 3 locations, with infiltration rates showing to be poor overall.

10) A deeper pit was also dug down to a depth of 3.0 m for groundwater level assessment. After 30 minutes the groundwater had risen to 1.70 below ground level. This means that infiltration SuDS are not a viable option for this particular development

11) Based on infiltration testing and groundwater level assessment it is considered that surface water runoff can't be managed by infiltration SuDS. Discharge of attenuated flows into the river is therefore necessary.

12) A surface water management plan has been proposed including cellular storage tank for the residential units and an oversized twin pipe storage for the access road. The scheme has been shown to accommodate runoff resulting from rainfall events up to and including the 100 year plus 40% climate change design rainfall event. The scheme would keep any post-development runoff rates to no greater than 2.0 l/s.

13) Residual risks for the scheme include the possibility of the occurrence of rainstorms in excess of the 100 year plus 40% climate change design storm, and a blockage of any SuDS features. Additional reassurance has been provided by keeping free board to accommodate rainfall in excess of 100 year plus 40% climate change event. However, due to the local topography of the site, any exceedance flows will be directed to the river.

14) Due to high ground water level, foul water management plan has been proposed by implementing treatment plant. The outflow from the treatment plant will be discharged to the watercourse.

15) Two alternate options have been proposed for foul water treatment under the Foul Drainage Strategy presented in Appendix F. Option A considers the implementation of a Biokube wastewater treatment system to reach an average phosphate concentration of 1.2mg/L followed by a reedbed to further decrease the phosphate concentration, while option B includes a Klagester Biodisc treatment plant to attain a concentration of phosphates of 2mg/L, followed by a reed bed to polish the treated effluent. In both cases, the treated effluent will be discharged to the watercourse that runs throughout the whole year.

16) To further improve the impacts of phosphates in the eutrophic-sensitive environment, it is proposed that the foul water flows from the old building, falling within the site blue boundary, are directed to the proposed drainage system serving the new development. Currently, the old building drains the foul water flows into a septic tank and the improvement of this system would result in the improvement of phosphate concentrations.

17) The function of both the surface water management and foul water management system must be understood by those responsible for maintenance. Performance deterioration can usually be minimised if the system is properly designed, monitored and maintained. The responsibility of maintaining the attenuation structures would be with the property owner(s).

In summary, flood risks at the site are generally low or very low for all sources, and if the measures recommended within this report are implemented, surface water at and runoff from the site would be managed to comply with the flood risk provisions of the NPPF.

Yours sincerely,

[redacted]

Joao Gil
Senior Flood Risk Engineer & Project Manager

Limitation of liability and use

The work described in this report was undertaken for the party or parties stated; for the purpose or purposes stated; to the time and budget constraints stated. No liability is accepted for use by other parties or for other purposes, or unreasonably beyond the terms and parameters of its commission and its delivery to normal professional standards.

Flood Risk Assessment Template Based on the NPPF Practice Guide[1]

1 Development description and location

1a. What type of development is proposed and where will it be located?

- A location plan at an appropriate scale should be provided with the FRA, or cross referenced to the main application when it is submitted.

The location of the site of proposed development (the site) is Lingen Nursery, Lingen, Shropshire, SY7 0DY. The site is currently a nursery under commercial/agricultural use.

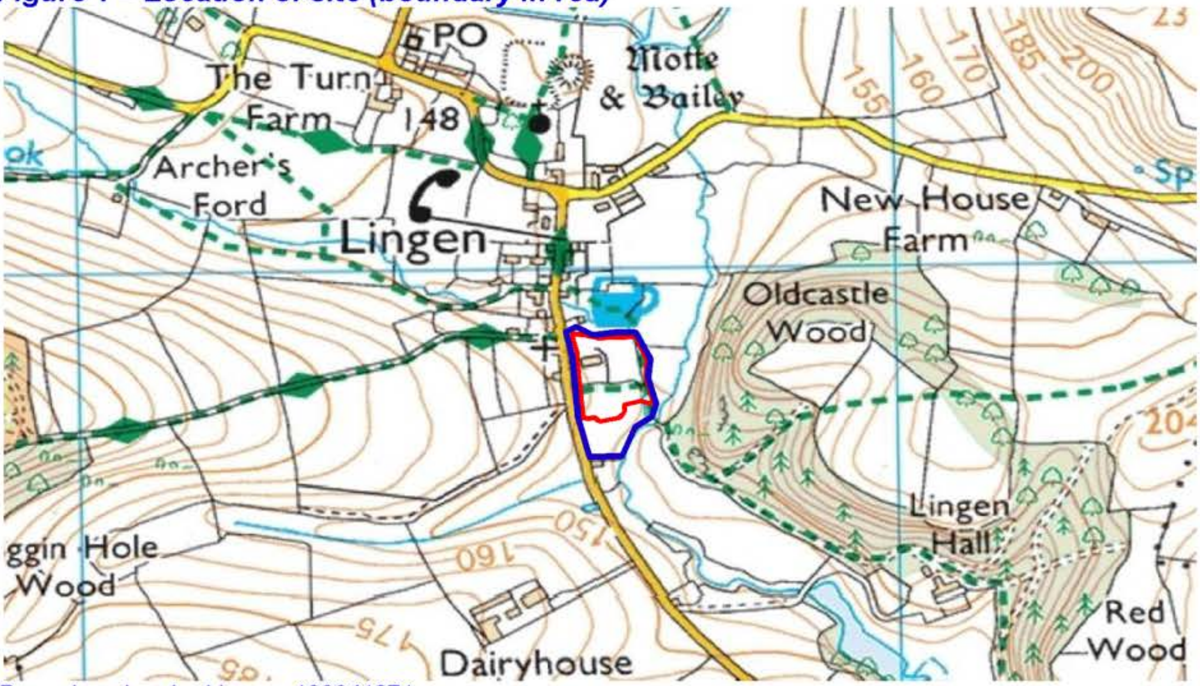
The proposed development comprises eight residential properties, each with gardens, garages and open (uncovered) parking spaces. The proposals include a new access road running from the south-western corner of the site. The access road up to the car park is to be adopted by Herefordshire City Council.

The approximate coordinates of the site are reproduced in Table 1. The location of the site is shown in Figure 1, the location plan is shown in Figure 2. An aerial photograph of the site is shown in Figure 3.

Table 1 – Coordinates of Site

Easting, Northing	336657, 266851
Nearest Post Code	SY7 0DY

Figure 1 – Location of site (boundary in red)



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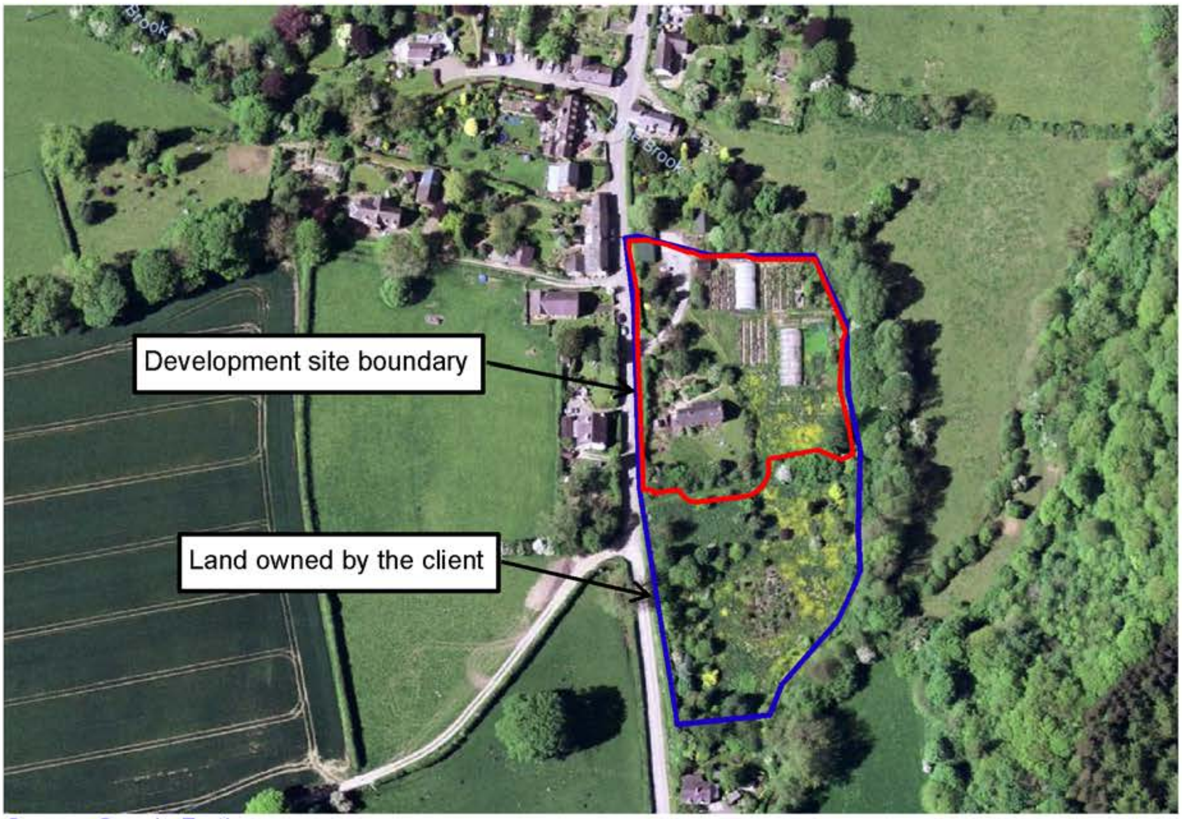
[1] http://planningguidance.planningportal.gov.uk/blog/guidance/flood-risk-and-coastal-change/

Figure 2 – Location plan of site (boundary in red)

Source: Berrys

Figure 3 – Aerial photograph of site (boundary in red)



Source: Google Earth

1b. What is its vulnerability classification?

- Vulnerability classifications are provided in Table 2, NPPF Technical Guide

In terms of the National Planning Policy Framework (NPPF), land and buildings used for dwelling houses are classified as "More Vulnerable".

1c. Is the proposed development consistent with the Local Development Documents?

- Where the site is allocated in an existing LDD the allocation should be referred to. Your Local Planning Authority planning officer should be able to provide site-specific guidance on this issue.

The location of the site is covered by Herefordshire Council. The Herefordshire Local Plan Core Strategy was adopted in October 2015 as a guide for the county's development and change up to 2031. The proposed development is consistent with the Local Development Framework.

1d. Please provide evidence that the Sequential Test or Exception Test has been applied in the selection of this site for this development type?

- Evidence is required that the Sequential Test has been used in allocating the proposed land use proposed for the site and that reference has been made to the relevant Strategic Flood Risk Assessment (SFRA) in selecting development type and design (See paragraphs 100-104, NPPF and paragraphs 3-5, NPPF Technical Guide). Your Local Planning Authority planning officer should be able to provide site-specific guidance on this issue.
- Where use of the Exception Test is required, evidence should be provided that both elements of this test have been passed (see paragraphs 102, NPPF and paragraphs 4-5, NPPF Technical Guide). Your Local Planning Authority planning officer should be able to provide site-specific guidance on this issue.

The proposed development is located largely within Flood Zone 1 and classified as "More Vulnerable". The proposed development would be appropriate in the flood zone and therefore neither the Sequential Test nor the Exception Test is required for this assessment.

1e. *[Particularly relevant to minor developments (alterations & extensions) & changes of use]* Will your proposal increase overall the number of occupants and/or users of the building/land; or the nature or times of occupation or use, such that it may affect the degree of flood risk to these people?

The number of people living at this site would increase as a result of the proposals. The proposed works are located entirely within Flood Zone 1. Any additional risk from flooding to occupants of the site would be low since and safe access/egress in assured (see section 6).

2. Definition of the flood hazard

2a. What sources of flooding could affect the site? (see paragraph 2, NPPF Technical Guide).

- This may include hazards such as the sea, reservoirs or canals, which are remote from the site itself, but which have the potential to affect flood risk (see Section 1 of the NPPF Practice Guide).

The possible sources of flood risk which could affect this site are summarised in Table 2.

Table 2 – Sources of Flooding that could affect the site

Key Sources of Flooding	Possibility at Site
Fluvial	Generally low risk, with small areas of medium risk in the north east and south east of the development site
Tidal	Very Low
Groundwater	Very Low
Sewers	Very Low
Surface water	Generally very low risk, with small areas of medium and high risk through the centre of the development site
Infrastructure failure	None known whose failure could cause flooding

List taken from NPPF Practice Guide, Paragraph 2

The site is located predominantly within fluvial Flood Zone 1, further discussed in Section 3a.

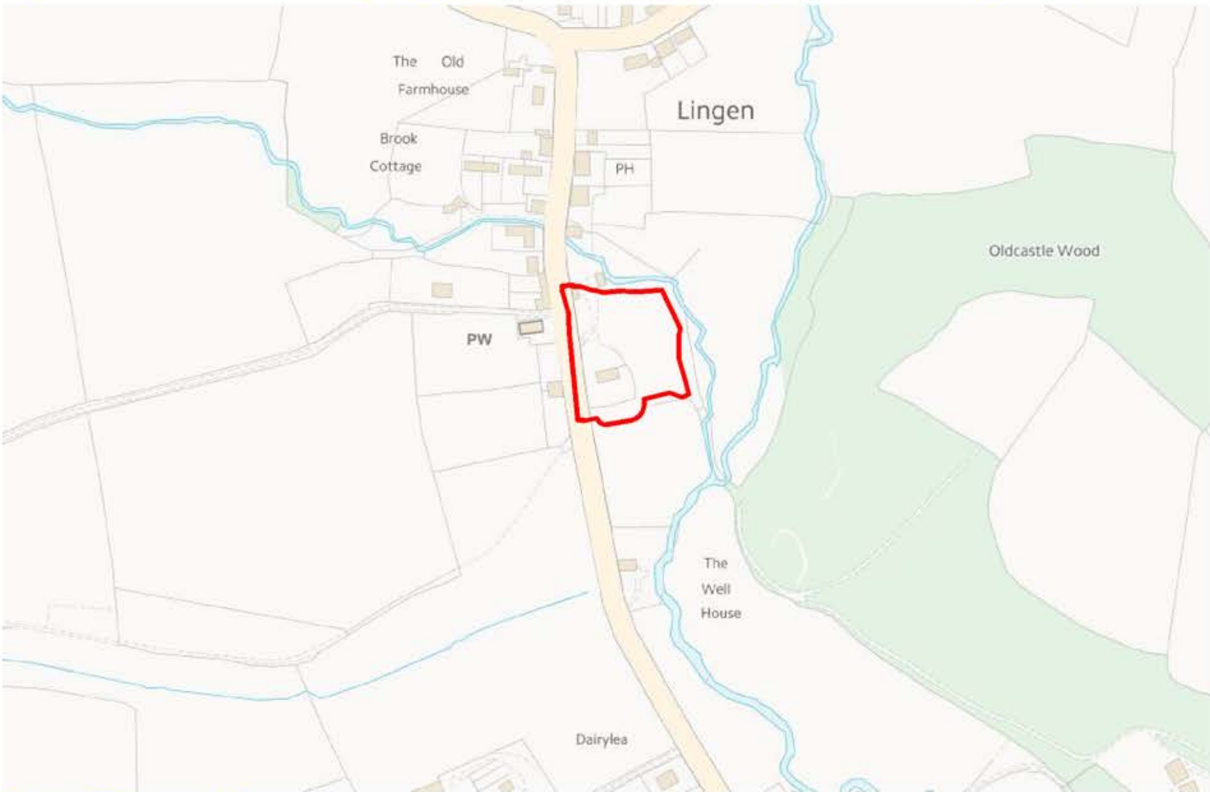
The Environment Agency "Flood Risk from Surface Water" map at the site is shown in Figure 4. In general, the risk of surface water flooding at the site is very low, although small areas of medium and high risk through the centre of the development site are indicated.

The Environment Agency "Flood Risk from Reservoirs" map at the site is shown in Figure 5. The site is not at risk of flooding from infrastructure (reservoir) failure.

Figure 4 – Environment Agency surface water flood risk map (site boundary in red)

Source: Environment Agency

Figure 5 – Environment Agency reservoir flood risk map (site boundary in red)



Source: Environment Agency

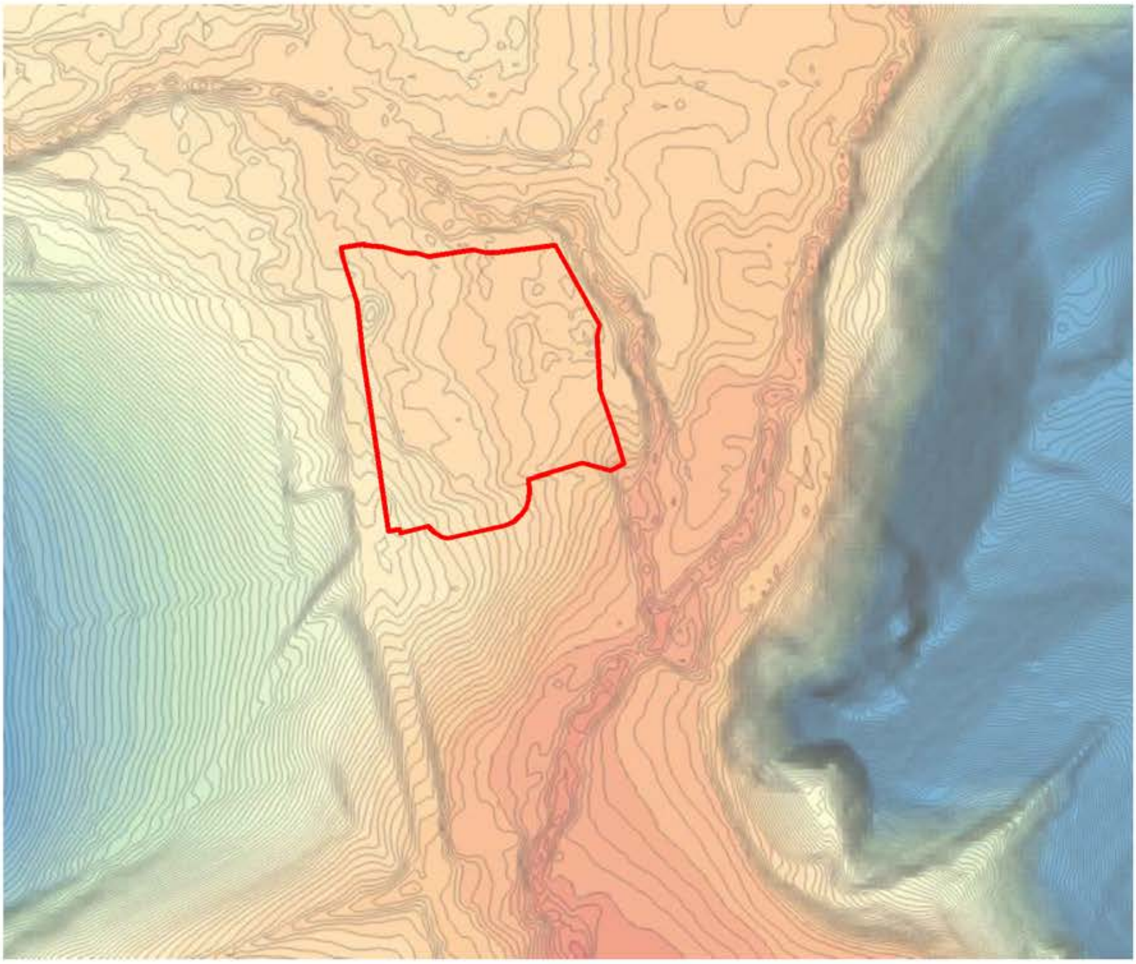
2b. For each identified source, describe how flooding would occur, with reference to any historic records wherever these are available.

- An appraisal of each identified source, the mechanisms that could lead to a flood occurring and the pathways that flood water would take to, and across, the site.
- Inundation plans, and textural commentary, for historic flood events showing any information available on the mechanisms responsible for flooding, the depth to which the site was inundated, the velocity of the flood water, the routes taken by the flood water and the rate at which flooding occurred.

The flood risk from fluvial sources derives from the proximity of the development site to Lime Brook. The site falls predominantly within Flood Zone 1, with small areas at the north-eastern side and south-western side of the site located in Flood Zones 2 and 3 respectively.

Flood risk from surface water is related to topographic depressions within the site. The LiDAR image shown on Figure 6 – LiDAR image of the site (site boundary in red) displays the presence of a low elevation area in the middle of the site.

Figure 6 – LiDAR image of the site (site boundary in red)



2c. What are the existing surface water drainage arrangements for the site?

- Details of any existing surface water management measures already in place, such as sewers and drains and their capacity.

The site is currently a nursery under commercial/agricultural use, and for much of the site no special surface water drainage arrangements are in place. At least one soakaway exists to manage runoff from the exiting building, but the effectiveness of this this structure is not known. However, soil mapping indicates that the structure is unlikely to be effective.

3. Probability

3a Which flood zone is the site within?

- The flood zones are defined in Table 2, NPPF Technical Guide.

The Environment Agency "Flood Risk from Rivers or Sea" map reproduced in Figure 7 shows that the site falls predominantly within Flood Zone 1, beyond the limits of the 1 in 1,000 year *fluvial flood event. However, small areas close to the north-eastern side and south-western boundary of the site are located in Flood Zones 2 and 3 respectively.*

The Environment Agency flood risk mapping does not include, and is not amenable to, extension due to climate change allowances. Accordingly, fluvial flood risk at the site has been assessed using site-specific hydrological analysis (Appendix D) and hydraulic modelling (Appendix E).

Hydraulic Modelling has shown the site to be substantially beyond the 100 year plus 35% climate change, and 1,000 year flood extents (see Appendix E, Figure D-9 and D-10).

Figure 7 – Environment Agency flood risk from rivers or sea (site boundary in red)

Source: Environment Agency

3b If there is a Strategic Flood Risk Assessment covering this site, what does it show?

- The planning authority can advise on the existence and status of the SFRA.

The Herefordshire Council Strategic Flood Risk Assessment (SFRA) Final Report (2009) makes no references to Lingen or any other locations close to the site.

3c What is the probability of the site flooding taking account of the contents of the SFRA and of any further site-specific assessment?

This may need to include

- a description of how any existing flood risk management measures affect the probability of a flood occurring at the site FRA Pro-forma
- supporting evidence and calculations for the derivation of flood levels for events with a range of annual probability
- inundation plans of, and cross sections through, the existing site showing flood extents and levels associated with events with a range of annual probability
- a plan and description of any structures which may influence the probability of a flood occurring at the site. This may include bridges, pipes/ducts crossing a watercourse, culverts, screens, embankments or walls, overgrown or collapsing channels and their likelihood to choke with debris.
- details of any modelling studies completed to define the exiting degree of flood risk

As indicated in Table 2, the risk of flooding at the site is generally low from rivers and very low from all remaining sources.

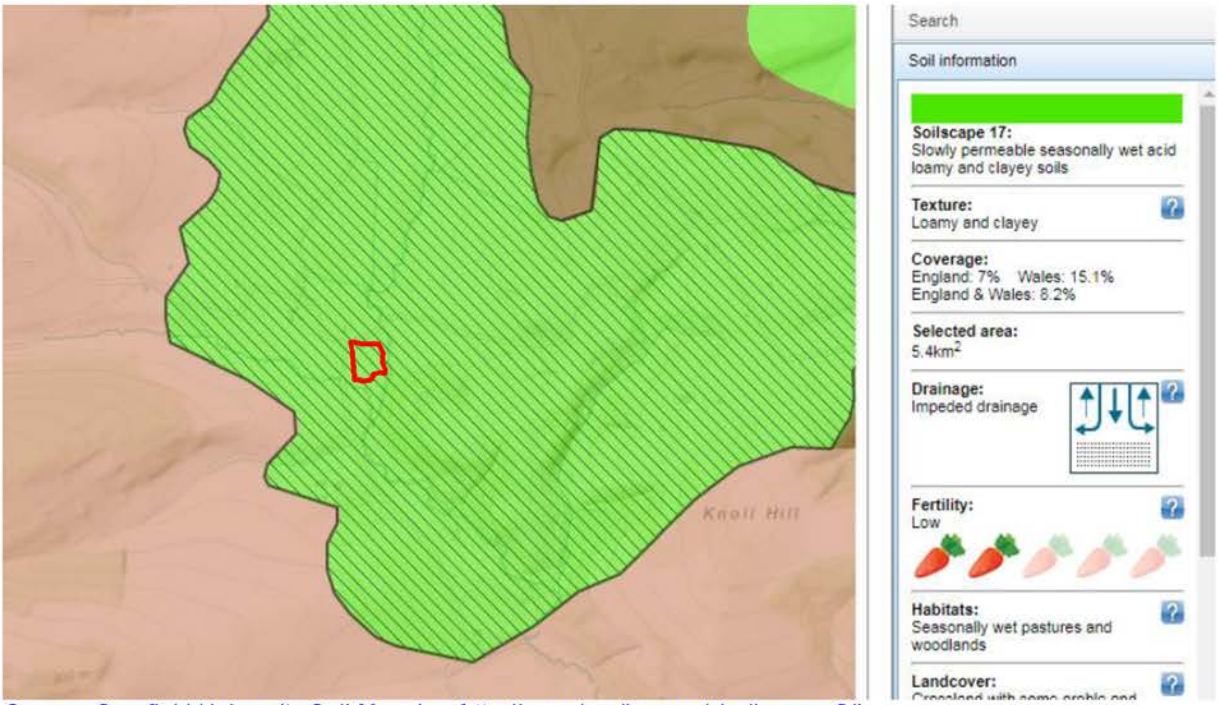
3d What are the existing rates and volumes of run-off generated by the site?

- This should generally be accompanied by calculations of run-off rates and volumes from the existing site for a range of annual probability events (see Section 21 of the NPPF Practice Guide).

The site is located on land under commercial/agricultural use. Runoff rates and volumes are expected to be close to Greenfield rates and volumes. The effectiveness of the soakaway(s) serving the existing building is not known but is unlikely to substantially increase runoff rates above Greenfield.

The Soilscapes regional soil mapping reproduced in Figure 8 shows that the site lies on soils characterised as "slowly permeable seasonally wet acid loamy and clayey soils".

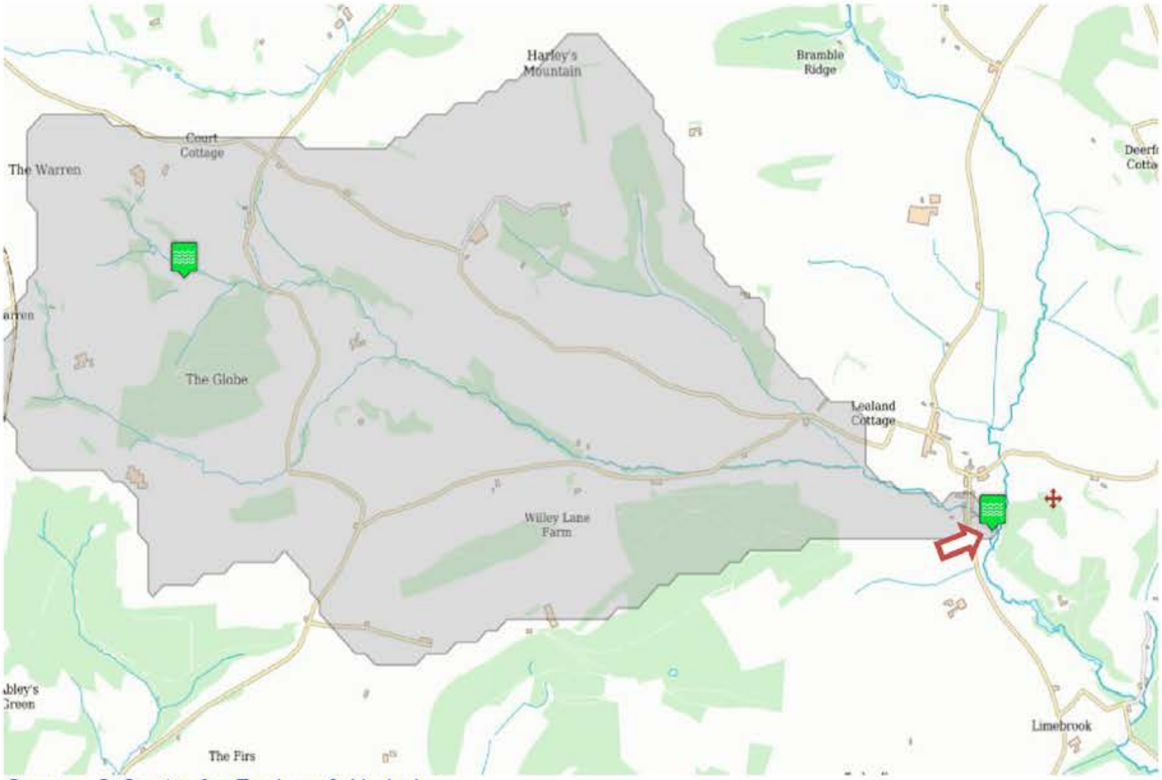
Figure 8 – Soil Map at location of site (site boundary in red)



Source: Cranfield University Soil Mapping (http://www.landis.org.uk/soilscapes2/)

The Flood Estimation Handbook (FEH) Web Service was used to retrieve the catchment descriptors at the site. The catchment boundary is shown in Figure 9, with the site indicated by the red arrow. A subset of the catchment descriptors at the site is shown in Table 3.

Figure 9 – FEH Catchment containing the site



Source: © Centre for Ecology & Hydrology

Table 3 – Selected Catchment Characteristics

	Location:	**Lingen**
	NGR (catchment outlet):	**336700, 266850**
	NGR (catchment centroid):	**SO 36700 66850**
AREA	Catchment area (km^2)	7.7725
ALTBAR	Base flow index (m)	268
ASPBAR	Base flow index (degrees)	113
ASPVAR	Base flow index	0.31
BFIHOST	**Base flow index**	**0.702**
DPLBAR	Mean drainage path length (km)	3.57
DPSBAR	Mean drainage path slope (m/km)	166.90
FARL	Index of lakes	1.00
LDP	Longest drainage path (km)	6.51
PROPWET	Proportion of time soil is wet	0.490
RMED-1H	Median 1 hour rainfall (mm)	10.3
RMED-1D	Median 1 day rainfall (mm)	33.9
RMED-2D	Median 2 day rainfall (mm)	44.5
SAAR6190	SAAR for the period 1961-1990	890
SAAR4170	SAAR for the period 1941-1970	977
SPRHOST	**Percentage runoff**	**15.68**
URBEXT2000	Urban extent 2000	0.000

Source: © Centre for Ecology & Hydrology

The site boundary encloses an area of approximately 7,570 m^2. Referencing a subset of the catchment descriptors reproduced in Table 3, Greenfield runoff rates were undertaken for a range of rainfall events as outlined in Appendix A. The resultant peak Greenfield runoff rates, and the corresponding runoff volumes, are shown in Table 4 below. The target runoff rates scaled for the impermeable areas are also shown in Appendix A.

Table 4 – Peak Greenfield runoff rates and corresponding volumes for the existing site

Return period (years)	Runoff rate (l/s/ha)	Runoff volume (m^3)
2	3.95	43.3
30	11.50	102.6
100	16.42	140.4

** for 6 hour duration rainfall events*

4. Climate change

How is flood risk at the site likely to be affected by climate change?

- Paragraphs 11-15, of the NPPF Technical Guide provide guidance on how to assess the impacts of climate change.

The Environment Agency and NPPF require a consideration of the impacts of climate change on flood risk for any proposed development. In February 2016, the Environment Agency updated the climate change allowances required in Flood Risk Assessments (Environment Agency, 2016). This advice supersedes previous climate change allowances to support the NPPF (DCLG, 2012). The Environment Agency (2016) state:

> *"Making an allowance for climate change in your flood risk assessment will help to minimise vulnerability and provide resilience to flooding and coastal change in the future. The climate change allowances are predictions of anticipated change for:*
>
> - *peak river flow by river basin district;*
> - *peak rainfall intensity;*
> - *sea level rise;*
> - *offshore wind speed and extreme wave height."*

For this site, located predominantly in Flood Zone 1 and outside the limits of the 1 in 1,000 year fluvial flood, but adjacent to a watercourse and therefore including areas of Flood Zone 2 and 3, the anticipated climate change allowances for increase in peak river flows could increase the risk of fluvial flooding. The 'Higher Central' and 'Upper End' allowances have been considered for assessment of the effects of climate change on 100 year peak flood flows.

Table 5 – Peak river flow climate change allowances for the Severn River Basin district

Allowance Category	Total potential change anticipated		
	2015 to 2039	2040 to 2069	2070 to 2115
Upper End	25%	40%	70%
Higher Central	15%	25%	35%
Central	10%	20%	25%

Source: Environment Agency (2016)

The site is distant from the coast therefore unaffected by the anticipated changes in sea level rise, offshore wind speed and extreme wave height. For rainfall, Table 6 shows the anticipated changes in small catchments, recommending a progressive increase, reaching 40% for the 'Upper End' allowance by 2115. This allowance is recommended in line with NPPF guidance, in view of the low vulnerability of the site and the relatively minor consequences of exceedance. The 'Central' allowance based on the 50th percentile, i.e. that there is a 50% chance that rainfall will increase by less than this value (20%) and a 50% chance that rainfall will increase by more.

Table 6 – Peak rainfall intensity allowance in small and urban catchments

	Total potential change anticipated		
Applies across all of England	***2010 to 2039***	***2040 to 2059***	***2060 to 2115***
Upper end	*10%*	*20%*	*40%*
Central	*5%*	*10%*	*20%*

Source: Environment Agency (2016)

5. Detailed development proposals

Where appropriate, are you able to demonstrate how land uses most sensitive to flood damage have been placed in areas within the site that are at least risk of flooding, including providing details of the development layout?

- Reference should be made to vulnerability classification, Table 2 of the NPPF Technical Guide.
- Section 4 of the NPPF Practice Guide provides guidance on how the sequential approach can be used to inform the lay-out of new development sites.

The site layout plan showing the planning boundary is shown in Figure 10. The area of the site enclosed by the planning boundary is approximately 7,570 m².

Figure 10 – Layout of the Proposed Development

Source: Berrys

The site is located predominantly within Flood Zone 1. The proposed development is classified as "More Vulnerable" and, adopting a sequential approach, all development has been placed entirely within Flood Zone 1.

The proposed development comprises eight residential properties of various designs, each with gardens, garages and open parking. The central paved area would be constructed using permeable materials. The access road and the roof areas constitute impermeable surfaces for the purpose of surface water management (see section 7b).

A summary of the impermeable surface areas at the site is presented in Table 7. The total proposed impermeable surface area is approximately 1,627 m².

Table 7 – Proposed Impermeable Surface Areas on Site

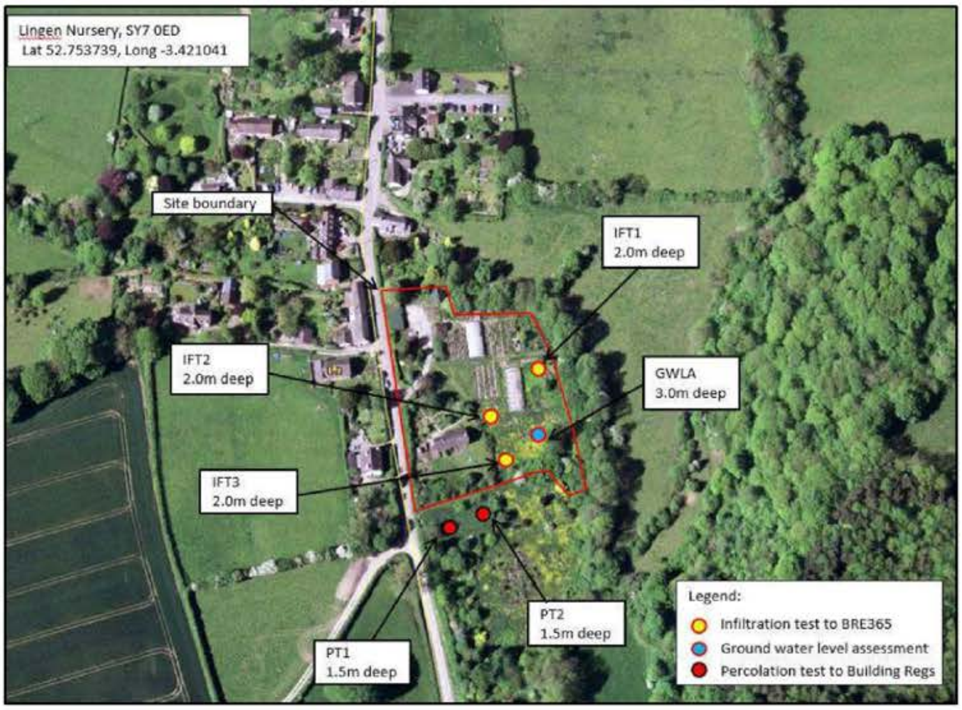
Surface Area Description	***Area (m2)***
Plot 1	*108*
Plot 2	*114*
Plot 3	*150*
Plot 4	*175*
Plot 5	*140*
Plots 6	*120*
Plots 7	*155*
Plot 8	*145*
Access Road	*520*
Total impermeable	***1627***

Source: Berrys

6. Flood risk management measures
How will the site be protected from flooding, including the potential impacts of climate change, over the development's lifetime? • This should show that the flood risk management hierarchy has been followed and that flood defences are a necessary solution. This should include details of any proposed flood defences, access/egress arrangements, site drainage systems (including what consideration has been given to the use of sustainable drainage systems) and how these will be accessed, inspected, operated and maintained over the lifetime of the development. This may need to include details of any modelling work undertaken in order to derive design flood levels for the development, taking into account the presence of any new infrastructure proposed.
The site is located predominantly within Flood Zone 1, with small areas of Flood Zone 2 and 3. No special measures to protect the site from fluvial flooding are proposed. *A sequential approach to land use is achievable, ensuring all development is located in Flood Zone 1, beyond the limits of the 1 in 1,000 year flood. The 100 year plus 35% climate change and 1,000 year flood levels are modelled and presented in Appendix E.* *Flood levels vary across the site, reflecting the slope of the adjacent watercourses. Setting finished floor levels sufficiently (i.e. 300 mm) above modelled 100yr+35%CC peak flood levels would provide further reassurance of flood resilience.*

7. Off Site impacts
7a How will you ensure that your proposed development and the measures to protect your site from flooding will not increase flood risk elsewhere? This should be over the lifetime of the development taking climate change into account. The assessment may need to include: • Details of the design basis for any mitigation measures (for example trash screens, compensatory flood storage works and measures to improve flood conveyance). A description of how the design quality of these measures will be assured and of how the access, operation, inspection and maintenance issues will be managed over the lifetime of the development. • Evidence that the mitigation measures will work, generally in the form of a hydrological and hydraulic modelling report. • An assessment of the potential impact of the development on the river, estuary or sea environment and fluvial/coastal geomorphology. A description of how any impacts will be mitigated and of the likely longer-term sustainability of the proposals.
With the exception of runoff from the proposed impermeable surfaces (Section 7b), flood risk elsewhere would not be affected by the proposed development.
7b How will you prevent run-off from the completed development causing an impact elsewhere? • Evidence should be provided that drainage of the site will not result in an increase in the peak rate or in the volumes of run-off generated by the site prior to the development proceeding.
In order not to increase flood risk elsewhere in the catchment, runoff from impermeable surfaces would need to be managed within the site. *To investigate drainage options three infiltration tests and two percolation tests were carried out within the proposed development site. The locations of the test holes are shown in Figure 11.*

Figure 11 – Test hole locations



Groundwater was observed in the groundwater level assessment pit (GWLA), with water table stabilising at 1.7m below ground and soil found to be damp at a depth of 1m below ground. Groundwater was also observed in PT1, with a narrow seam of broken shale running in this location. From the ground investigations it was concluded that runoff won't be managed by infiltration SuDS features. It is therefore believed that the solution will be a combined system between attenuation and infiltration.

The infiltration and percolation rates for the test holes are shown in Table 8. The details of the infiltration tests are shown in Appendix B.

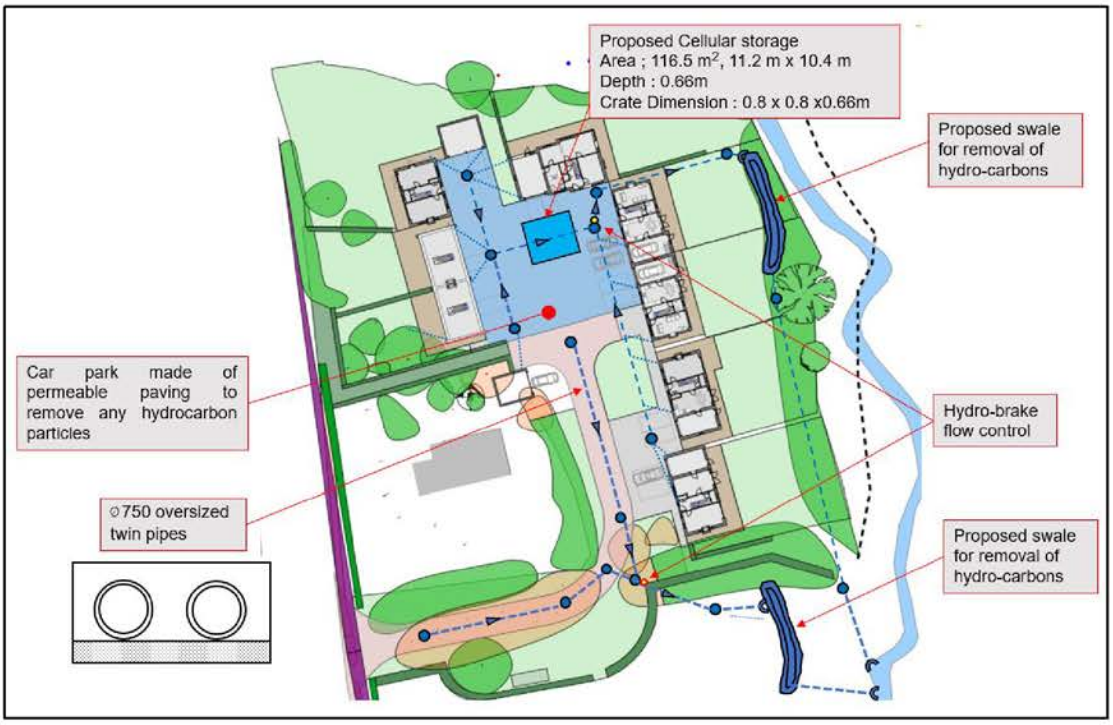
Table 8 - Infiltration Results

Test	*Comments*	*Infiltration Rate(m/hr)*
GWLA	*Groundwater found at 1.7m BGL*	*-*
IFT1	*No Ground water observed*	*1.82E-02*
IFT2	*No Ground water observed*	*2.18E-02*
IFT3	*No Ground water observed*	*1.57E-01*
PT1	*Ground water observed*	*-*
PT2	*Pit drained well*	*9.0 s/mm to 19.5s/mm*

As discussed above infiltration is not a feasible option to drain the entire area due to shallow ground water level within the site. Accordingly, a surface water management plan has been proposed by attenuating flows with cellular storage tanks, over-sized pipes and swales, followed by discharge into the brook. The roof area covering an approximate area 1,105 m² will be attenuated by a cellular storage tank and outflows will be disposed into a swale for further treatment before discharge into the water course. As attenuating flows to pre-development greenfield runoff rates would lead to small flow control outlets, post-development flows will be restricted to no greater than 2.0 l/s. The access road covering an area of approximately 520 m² will be attenuated by means of 2 No. 750 mm diameter pipes with outflows controlled by hydro-brakes to the proposed swale – with outflows no greater than 2/0l/s. The blue hatched area shown in the drainage layout will be built using permeable pavement and hence not considered *for drainage strategy. The permeable pavement will be formed using granular materials which will remove any hydro carbon particles from the flow.*

The proposed surface water management plan is shown in

Figure 12 – Layout of proposed surface water management plan



The proposed surface water management scheme therefore includes a cellular storage tank and 2 No. oversized pipes – modelled via Micro-drainage WinDES software. The descriptions of the drainage features and their performances are described below. The details of the cellular storage tank including the associated structures are summarised in Table 9 and Table 10. The performance of the cellular storage tank is summarised in Table 11. The scheme has been shown to accommodate runoff resulting from rainfall events up to and including the 100 year plus 40% climate change design rainfall event.

Table 9 - Cellular storage specification

Structure	Stormbloc
Base Area	116.5 m² (10.4 m x 11.2 m)
No. blocks	182 (0.8 x 0.8 x 0.66)
Depth	0.66 m
Side Slope (1: x)	0

Table 10 - outflow characteristics

Outflow Control	Hydro-Brake
Diameter	73 mm
Design head	0.60 m
Design flow	2.0/s
Invert Level *	0.0 m

Table 11 – Geo-cellular Storage tak Performance

Return period (years)	Maximum depth (m)	Maximum outflow (l/s)	Half Drain Time (hour)
2year +40%CC	0.155	2.0	1.40
30 year +40%	0.428	2.0	3.45
100 year +40%CC	0.605	2.0	4.77

Table 11 shows that the outflow from the cellular storage is lower than the target rate for all the event. However, it is to be noted that the outflow will go through a swale to remove any contaminants, where it is believed to also be attenuated.

Other combinations of attenuation structure(s) are of course possible, but this analysis illustrates one way in which the necessary attenuation scheme can be configured.

The drainage of the access road covering an approximate area of 520m^2 has been designed considering oversized twin pipes – with controlled outflows – to a proposed swale before discharging into the brook. The dimension of the pipes and outflow structures are shown in Table 12 and Table 13.

Table 12 - Dimension of Over Size Pipe

Diameter (mm)	750
Type	Twin pipe
Length(m)	30
Slope (1:X)	500

Table 13 - Parameters of Hydro-brake flow controls

Flow control	Hydro-Brake Optimum Outflow Control
Design head	0.70 m
Design flow	2.0l/s
Invert level	0.0 m
Diameter	71 mm

The performance of the twin pipe storage is shown in Table 14, which shows that the outflow from the twin pipes is no greater than 2.0l/s, as meeting pre-development greenfield runoff rates would lead to a small flow control outlet diameter. The maximum depth is also within the pipe diameter allowing a free board of 41mm for the 100 year plus 40% climate change event, before any pipe surcharge occurs. The detailed output from the Micro Drainage source control is shown in Appendix C.

Table 14 - Performance of twin-pipes storage

Return period (years)	Maximum depth (m)	Out flow(l/s)
2year +40%CC	0.205	2.0
30 year +40%	0.501	2.0
100 year +40%CC	0.719	2.0

8. Residual risks
8a What flood-related risks will remain after you have implemented the measures to protect the site from flooding? • Guidance on residual risks is provided in Section 14 of the NPPF Practice Guide.
Residual risks for the scheme include the possibility of the occurrence of rainstorms in excess of the 100 year plus 40% climate change design storm, and a blockage of the attenuation system. *In order to manage this extra risk, a free board is available for providing additional storage capacity in excess of 100 year plus 40% climate change event. The cellular storage tank has a free board of 55 mm to accommodate the rainfall above 100 year plus 40% event. The storage provided by the twin pipes for the access road drainage also has a free board of 36 mm for the same event.*
8b How, and by whom, will these risks be managed over the lifetime of the development? • Reference should be made to flood warning and evacuation procedures, where appropriate, and to likely above ground flow routes should sewers or other conveyance systems become blocked or overloaded. This may need to include a description of the potential economic, social and environmental consequences of a flood event occurring which exceeds the design standard of the flood risk management infrastructure proposed and of how the design has sought to minimize these – including an appraisal of health and safety issues.
The function of the surface water management scheme must be understood by those responsible for maintenance, regardless of whether individual components are on the surface or below ground. Performance deterioration can usually be minimised if the system is properly designed, monitored and maintained. *The responsibility of maintaining the attenuation structures would be with the property owner(s).*

References

Author	Date	Title/Description
Centre for Ecology and Hydrology	2016	The Flood Estimation Web Service. Available at: https://fehweb.ceh.ac.uk/
Cranfield University	2018	Soilscapes Map. Available at: http://www.landis.org.uk/soilscapes/ Access date: 13th November 2018
CIRIA	2015	The SUDS Manual - CIRIA Report C753.
DCLG	Mar 2012(a)	National Planning Policy Framework.
DCLG	Mar 2012(b)	Technical Guidance to the National Planning Policy Framework.
Environment Agency/ UK Government	2018	Interactive Flood Maps. Available at: https://flood-warning-information.service.gov.uk/long-term-flood-risk/ Access date: 13th November 2018
Herefordshire Council	2009	Surface water Management: interim guide for developers Access date: 13th November 2018
Herefordshire County Council	July 2016	Sustainable Drainage System (SuDS) Handbook Access date: 13th November 2018
Wallingford HydroSolutions Ltd	2015	ReFH2: The Revitalised Flood Hydrograph Modelling Tool Version 2.

Appendix A ESTIMATION OF RUNOFF RATES

A.1 Overview

An estimate of current Greenfield runoff rates and volumes at the site is required in order to complete section 3d of the main report. Scaling the Greenfield rates to the size of the proposed impermeable surface area effectively provides target rates for attenuated discharge (see Appendix B).

The following appendix summarises the methodology and results of the estimation of the Greenfield rates for the site.

A.2 Catchment

In order to estimate the Greenfield rates and volumes at the site, the Flood Estimation Handbook (FEH) Web Service was used to retrieve the catchment descriptors for the site. The catchment boundary is shown in Figure A-1, and a selection of the catchment descriptors are listed in Table A-1.

Figure A-1 FEH Catchment containing the site

Source: © Centre for Ecology & Hydrology 2016

With the exception of the AREA and BFIHOST descriptors, the retrieved catchment descriptors are considered likely to be generally representative of the site.

The AREA descriptor was changed to the size of the site as part of the plot scale adjustment described in section A.3.

Table A-1 Selected Catchment Characteristics

	Location:	Lingen
	NGR (catchment outlet):	336700, 266850
	NGR (catchment centroid):	SO 36700 66850
AREA	Catchment area (km2)	7.7725
ALTBAR	Base flow index (m)	268
ASPBAR	Base flow index (degrees)	113
ASPVAR	Base flow index	0.31
BFIHOST	Base flow index	0.702
DPLBAR	Mean drainage path length (km)	3.57
DPSBAR	Mean drainage path slope (m/km)	166.90
FARL	Index of lakes	1.00
LDP	Longest drainage path (km)	6.51
PROPWET	Proportion of time soil is wet	0.490
RMED-1H	Median 1 hour rainfall (mm)	10.3
RMED-1D	Median 1 day rainfall (mm)	33.9
RMED-2D	Median 2 day rainfall (mm)	44.5
SAAR	SAAR for the period 1961-1990 (mm)	890
SAAR4170	SAAR for the period 1941-1970 (mm)	977
SPRHOST	Percentage runoff	15.68
URBEXT2000	Urban extent 2000	0.000

Source: © Centre for Ecology & Hydrology 2016

A comparison of the soil map (Figure 8 of the main report) and the catchment map indicates that while the site is located on soils described as *slowly permeable seasonally wet acid loamy and clayey soils*, approximately 90% of the catchment upstream of the site is located on *freely draining slightly acid loamy soils*. As a result, the base flow descriptor BFIHOST for the catchment as a whole is likely to significantly overestimate baseflow for the site, and therefore significantly underestimate runoff rates at the site. By referencing a similar sized catchment immediately to the east and located on a similar soil type to the subject catchment[2], the value of BFIHOST was changed from 0.702 to 0.500 to better reflect the soil type at the site.

A.3 Methodology

The estimation of peak rates of pre-development runoff (e.g. Greenfield runoff) has previously used the IH 124[3] (Marshall and Bayliss) method. This method uses parameters related to catchment and soil characteristics to establish a peak rate of runoff. More recently, the rainfall runoff modelling approach of ReFH version 2 (ReFH2) has been used. This method was found in work by the CEH (2015) to give a closer match to observed peak rates of runoff, and also provides a full hydrograph, rather than simply the peak flow derived by the former method. Following additional research and testing, ReFH2 was released in 2015. In particular, and with significance for the current site, ReFH2 incorporates a set of adjustments for “plot scale” conditions. These adjustments are applied to catchment level data to better reflect the scale of individual development plots. This is important since such plots tend to be much smaller than topographic catchments.

ReFH2 runoff calculations reference a subset of catchment descriptors generated by the FEH Web Service, described above, imported as an .xml file. Several descriptors may then be changed in order to achieve the “plot-scale” adjustments.

[2] Catchment 339600E, 267250N

[3] IH124: Institute of Hydrology Report No. 124 Flood Estimation for Small Catchments, June 1994

The AREA descriptor for the catchment was changed from 7.7725 km^2 to 0.5 km^2, in order to determine the time-to-peak (T_P) and base flow lag (B_L) parameters corresponding to an area of 0.5 km^2. Finally, the AREA descriptor was set to 0.00757 km^2 to reflect the area of the site (7,570 m^2) in km to five decimal places, and the time-to-peak (T_P) and base flow lag (B_L) parameters reset to the values determined for 0.5 km^2.

As described in section A.2, the value of BFIHOST was changed from 0.702 to 0.500 to better reflect the soil type at the site.

The default duration determined by the ReFH2 software (2 hours) was used to generate the Greenfield direct runoff hydrographs. In order to determine the Greenfield runoff volumes, the duration of the rainfall events was reset to 6 hours, as recommended in the SuDS Manual.

A.4 Results

Any assessment of runoff rates and volumes for the current site is likely to be associated with a high level of uncertainty. The following assessment has been made:

- Greenfield peak runoff rates, which should be interpreted as the runoff rates target for the post-development surface water management scheme.
- The results for the peak Greenfield runoff rates, and the corresponding runoff volumes, are shown in Table A-2. These results are also presented in section 3d of the main report.

Table A-2 Estimated peak Greenfield runoff rates and volumes* for existing site

Return period (years)	Runoff rate (l/s/ha)	Runoff volume (m^3)*
2	3.95	43.3
30	11.50	102.6
100	16.42	140.4

* for 6 hour duration rainfall events

The target rates for attenuated discharge of rainfall runoff from the proposed 1,847m^2 impermeable surfaces are shown in Table A-3.

Table A-3 Attenuated discharge target rates for 1,847 m^2 of impermeable surface

Return period (years)	Runoff rate (l/s/ha)	Target greenfield runoff rate (l/s)
2	3.95	0.73
30	11.50	2.12
100	16.42	3.03

Appendix B INFILTRATION TEST DETAILS

H+H Drainage

Tremayne, Mortimer's Cross, Herefordshire HR6 9TG
Phone: 0845 2008421. Mobile: 07837 628764

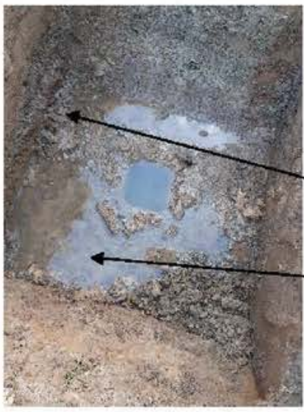
The GWLA trial hole was dug to a depth of 3m below ground level in the position shown on the associated plan. The uppermost 800mm was found to be a coarse sandy loam. Below 800mm, there were river deposits primarily ranging between fine sand and 60mm diameter pebbles with occasional larger stones.

At approximately 1m below ground level, the excavated spoil was found to be damp and the ground was found to be wet at a depth of 2m. At 2.8m below ground, free-running groundwater was found. After 30mins the groundwater had risen to 1.7m below ground level.

In addition to the infiltration trial holes, two percolation test holes (PT) were also dug in the locations shown on the associated plan.

As can be seen from the associated record sheets, the water level in PT1 increased. Some minor further excavation and investigation showed that there is a narrow seam of broken shale running through PT1. Ground water is flowing through this seam.

H+H Drainage
Tremayne, Mortimer's Cross, Herefordshire HR6 9TG
Phone: 0845 2008421. Mobile: 07837 628764



PT2 was excavated in the location shown on the associated plan. This test hole was found to drain well.

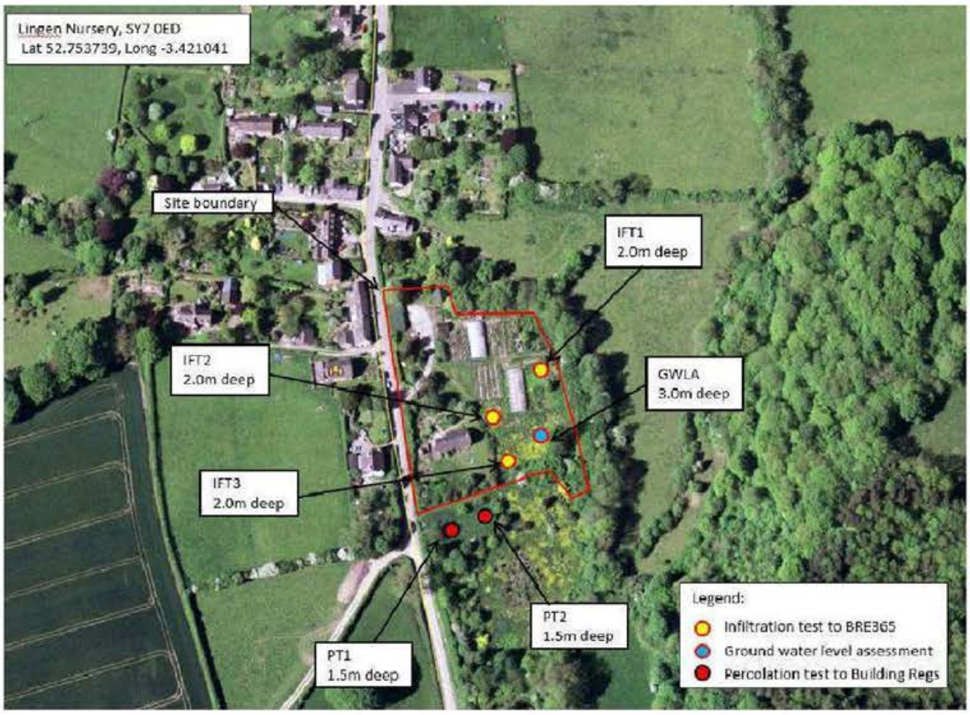
We hope that the enclosed results and associated information is more than sufficient for your needs. We look forward to hearing from you in the near future and if we can be of any further assistance with this or any other drainage matter, please do not hesitate to contact the undersigned.

Yours Faithfully,

Alex Taysum-Hunter.

H+H Drainage

Tremayne, Mortimer's Cross, Herefordshire HR6 9TG
Phone: 0845 2008421. Mobile: 07837 628764



Site Name: **Lingen Nursery** Date: **4th March 2019**

Test Hole: **IFT1**

Length	**1500mm**
Width	**500mm**
Depth	**2000mm**

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Test 1

Start	Time in mins	Water level below GL in mm	Comments
1	**10.50**	**220mm**	**Start**
2	**11.20**	**480mm**	
3	**11.50**	**870mm**	
4	**12.20**	**950mm**	
5	**12.50**	**1050mm**	
6	**13.20**	**1060mm**	
7	**13.50**	**1120mm**	
8	**14.20**	**1160mm**	
9	**14.50**	**1170mm**	
10	**15.20**	**1170mm**	
11	**15.50**	**1170mm**	
12	**16.20**	**1180mm**	
13	**16.50**	**1180mm**	
	11.00	**1320mm**	**24-hours**

Site Name: **Lingen Nursery** Date: **4th March 2019**

Test Hole: **IFT2**

Length	1550mm
Width	500mm
Depth	2000mm

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Test 1

Start	Time in mins	Water level below GL in mm	Comments
1	10.50	280mm	Start
2	11.20	1100mm	
3	11.50	1205mm	
4	12.20	1300mm	
5	12.50	1300mm	
6	13.20	1330mm	
7	13.50	1350mm	
8	14.20	1370mm	
9	14.50	1380mm	
10	15.20	1390mm	
11	15.50	1420mm	
12	16.20	1440mm	
13	16.50	1490mm	
	11.00	1660mm	24-hours

Site Name: Lingen Nursery Date: 4th March 2019

Test Hole: IFT3

Length	1580mm
Width	500mm
Depth	2000mm

Test 1

Start	Time in mins	Water level below GL in mm	Comments
1	10.50	200mm	Start
2	11.20	1340mm	
3	11.50	1480mm	
4	12.20	1660mm	
5	12.50	1740mm	
6	13.20	1830mm	
7	13.50	Empty	

Site Name: Lingen Nursery Date: 4th March 2019

Test Hole: IFT3

Length	1580mm
Width	500mm
Depth	2000mm

Test 2

Start	Time in mins	Water level below GL in mm	Comments
1	14.20	400mm	START
2	14.50	1390mm	
3	15.20	1510mm	
4	15.50	1640mm	
5	16.20	1730mm	
6	16.50	1800mm	

Site Name: **Lingen Nursery** Date: **4th March 2019**

Test Hole: **PT2**

Length	300mm
Width	300mm
Depth	300mm

Test 1

Start	Time in mins	Water level below GL in mm	Comments
1	12.35	50mm	Start
2	12.45	150mm	
3	12.55	210mm	
4	13.05	265mm	
5	13.15	300mm	

Test 2

Start	Time in mins	Water level below GL in mm	Comments
1	13.15	50mm	Start
2	13.25	150mm	
3	13.35	190mm	
4	13.45	230mm	
5	13.55	300mm	

Test 3

Start	Time in mins	Water level below GL in mm	Comments
1	14.05	50mm	Start
2	14.15	130mm	
3	14.25	180mm	
4	14.35	200mm	
5	14.45	230mm	
6	14.55	260mm	
7	15.05	300mm	

Test 4

Start	Time in mins	Water level below GL in mm	Comments
1	15.10	50mm	Start
2	15.20	140mm	
3	15.30	185mm	
4	15.40	210mm	
5	15.50	230mm	
6	16.00	240mm	
7	16.10	255mm	
6	16.20	280mm	
8	16.30	300mm	

Site Name: **Lingen Nursery** Date: **4th March 2019**

Test Hole: **PT1**

Length	300mm
Width	300mm
Depth	300mm

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Test 1

Start	Time in mins	Water level below GL in mm	Comments
1	12.30	50mm	Start
2	12.40	60mm	
3	12.50	65mm	
4	13.00	50mm	
5	13.10	30mm	
6	13.20	0mm	
7	13.30	-10mm	
8	13.40	-15mm	
9	13.50	-15mm	Abandoned

Appendix C Micro-drainage Outputs

C1. Cellular Storage Design

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Innovyze	Source Control 2017.1	

Summary of Results for 2 year Return Period (+40%)

Half Drain Time : 84 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control Σ (l/s)	Max Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	0.074	0.074	0.0	1.5	1.5	8.2	O K
30 min Summer	0.091	0.091	0.0	1.8	1.8	10.1	O K
60 min Summer	0.106	0.106	0.0	1.9	1.9	11.7	O K
120 min Summer	0.130	0.130	0.0	2.0	2.0	14.3	O K
180 min Summer	0.139	0.139	0.0	2.0	2.0	15.4	O K
240 min Summer	0.143	0.143	0.0	2.0	2.0	15.8	O K
360 min Summer	0.142	0.142	0.0	2.0	2.0	15.7	O K
480 min Summer	0.136	0.136	0.0	2.0	2.0	15.0	O K
600 min Summer	0.128	0.128	0.0	2.0	2.0	14.2	O K
720 min Summer	0.121	0.121	0.0	1.9	1.9	13.4	O K
960 min Summer	0.106	0.106	0.0	1.9	1.9	11.8	O K
1440 min Summer	0.088	0.088	0.0	1.8	1.8	9.7	O K
2160 min Summer	0.072	0.072	0.0	1.5	1.5	8.0	O K
2880 min Summer	0.064	0.064	0.0	1.3	1.3	7.0	O K
4320 min Summer	0.054	0.054	0.0	1.0	1.0	6.0	O K
5760 min Summer	0.049	0.049	0.0	0.9	0.9	5.4	O K
7200 min Summer	0.045	0.045	0.0	0.8	0.8	5.0	O K
8640 min Summer	0.043	0.043	0.0	0.7	0.7	4.7	O K
10080 min Summer	0.041	0.041	0.0	0.7	0.7	4.5	O K
15 min Winter	0.083	0.083	0.0	1.7	1.7	9.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	43.684	0.0	8.8	20
30 min Summer	28.548	0.0	11.6	32
60 min Summer	18.373	0.0	15.1	52
120 min Summer	12.695	0.0	21.0	88
180 min Summer	10.011	0.0	24.8	124
240 min Summer	8.396	0.0	27.8	158
360 min Summer	6.490	0.0	32.3	226
480 min Summer	5.378	0.0	35.7	292
600 min Summer	4.635	0.0	38.4	356
720 min Summer	4.096	0.0	40.7	418
960 min Summer	3.359	0.0	44.5	540
1440 min Summer	2.530	0.0	50.3	770
2160 min Summer	1.902	0.0	56.9	1128
2880 min Summer	1.559	0.0	62.2	1496
4320 min Summer	1.191	0.0	71.1	2208
5760 min Summer	0.994	0.0	79.4	2936
7200 min Summer	0.873	0.0	87.1	3672
8640 min Summer	0.791	0.0	94.7	4400
10080 min Summer	0.731	0.0	102.0	5136
15 min Winter	43.684	0.0	9.9	20

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Innovyze	Source Control 2017.1	

Summary of Results for 2 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
30 min Winter	0.103	0.103	0.0	1.9	1.9	11.4	O K
60 min Winter	0.120	0.120	0.0	1.9	1.9	13.3	O K
120 min Winter	0.145	0.145	0.0	2.0	2.0	16.0	O K
180 min Winter	0.153	0.153	0.0	2.0	2.0	17.0	O K
240 min Winter	0.155	0.155	0.0	2.0	2.0	17.1	O K
360 min Winter	0.148	0.148	0.0	2.0	2.0	16.3	O K
480 min Winter	0.136	0.136	0.0	2.0	2.0	15.0	O K
600 min Winter	0.123	0.123	0.0	1.9	1.9	13.7	O K
720 min Winter	0.111	0.111	0.0	1.9	1.9	12.3	O K
960 min Winter	0.093	0.093	0.0	1.9	1.9	10.3	O K
1440 min Winter	0.074	0.074	0.0	1.5	1.5	8.2	O K
2160 min Winter	0.060	0.060	0.0	1.2	1.2	6.7	O K
2880 min Winter	0.053	0.053	0.0	1.0	1.0	5.9	O K
4320 min Winter	0.045	0.045	0.0	0.8	0.8	5.0	O K
5760 min Winter	0.040	0.040	0.0	0.6	0.6	4.5	O K
7200 min Winter	0.038	0.038	0.0	0.6	0.6	4.1	O K
8640 min Winter	0.035	0.035	0.0	0.5	0.5	3.9	O K
10080 min Winter	0.034	0.034	0.0	0.5	0.5	3.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
30 min Winter	28.548	0.0	13.0	32
60 min Winter	18.373	0.0	17.0	58
120 min Winter	12.695	0.0	23.5	96
180 min Winter	10.011	0.0	27.8	134
240 min Winter	8.396	0.0	31.2	172
360 min Winter	6.490	0.0	36.1	244
480 min Winter	5.378	0.0	39.9	312
600 min Winter	4.635	0.0	43.0	376
720 min Winter	4.096	0.0	45.6	438
960 min Winter	3.359	0.0	49.9	548
1440 min Winter	2.530	0.0	56.3	786
2160 min Winter	1.902	0.0	63.7	1144
2880 min Winter	1.559	0.0	69.7	1504
4320 min Winter	1.191	0.0	79.7	2212
5760 min Winter	0.994	0.0	88.9	2944
7200 min Winter	0.873	0.0	97.6	3672
8640 min Winter	0.791	0.0	106.1	4416
10080 min Winter	0.731	0.0	114.3	5144

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	2
FEH Rainfall Version	2013
Site Location	GB 336700 266850 SO 36700 66850
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.111

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	0.056	4	8	0.055

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Model Details

Storage is Online Cover Level (m) 0.660

Cellular Storage Structure

Invert Level (m) 0.000 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	116.5	0.0	0.660	116.5	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0073-2000-0600-2000
Design Head (m)	0.600
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	73
Invert Level (m)	0.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.600	2.0
Flush-Flo™	0.177	2.0
Kick-Flo®	0.397	1.7
Mean Flow over Head Range	-	1.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.9	1.200	2.7	3.000	4.2	7.000	6.3
0.200	2.0	1.400	2.9	3.500	4.5	7.500	6.5
0.300	1.9	1.600	3.1	4.000	4.8	8.000	6.7
0.400	1.7	1.800	3.3	4.500	5.1	8.500	6.9
0.500	1.8	2.000	3.5	5.000	5.3	9.000	7.1
0.600	2.0	2.200	3.6	5.500	5.6	9.500	7.3
0.800	2.3	2.400	3.8	6.000	5.8		
1.000	2.5	2.600	3.9	6.500	6.0		

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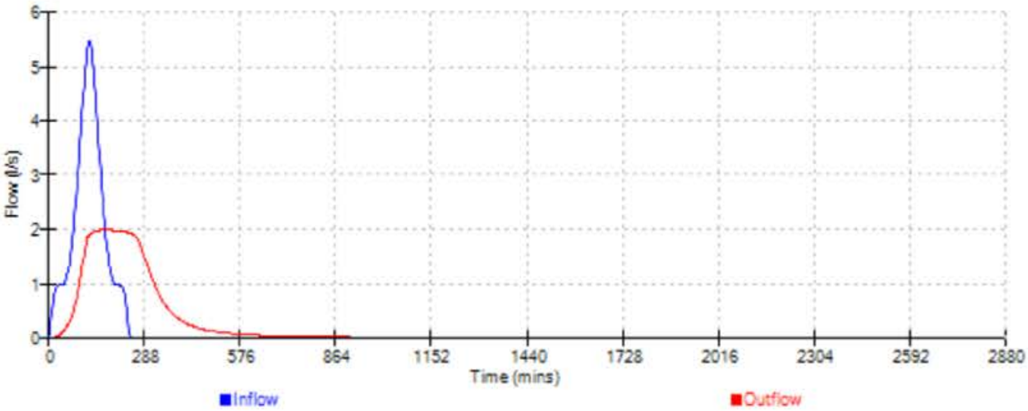
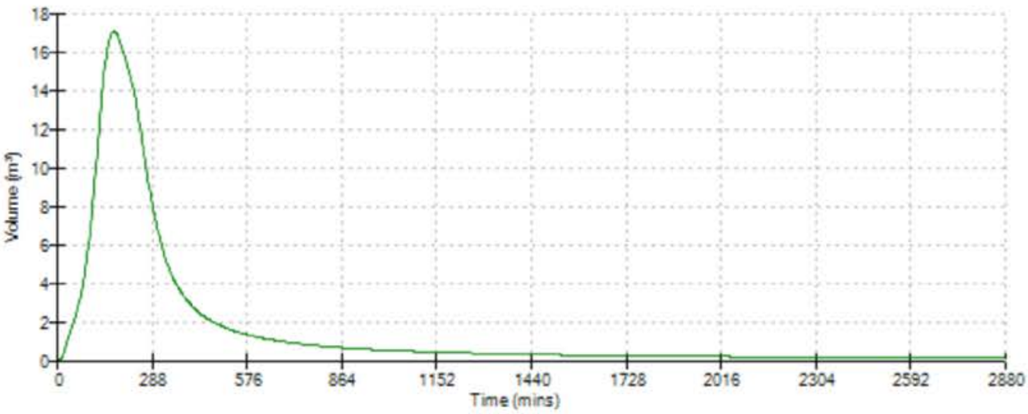
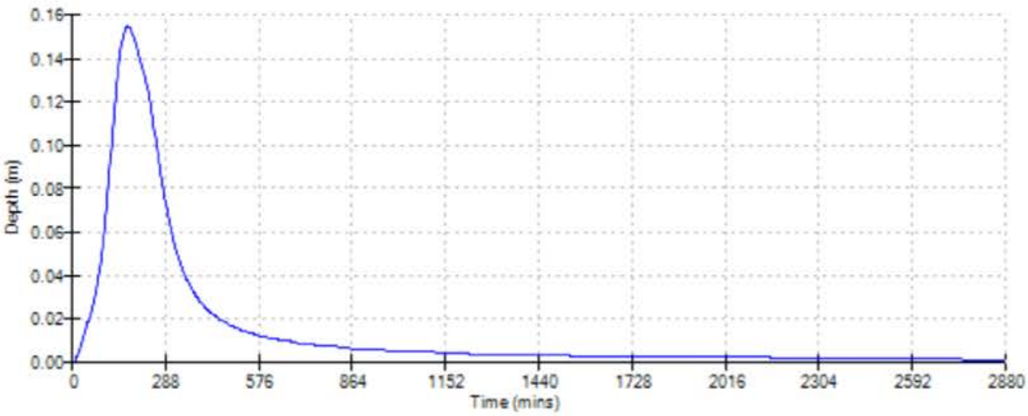
Innovyze	Source Control 2017.1	

Event: 240 min Winter







Hydrologic Services		Page 1
Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
Date 24/07/2019 14:05 File 30yr-Cellular-storage-P...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Summary of Results for 30 year Return Period (+40%)

Half Drain Time : 207 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	0.205	0.205	0.0	2.0	2.0	22.6	O K
30 min Summer	0.269	0.269	0.0	2.0	2.0	29.8	O K
60 min Summer	0.333	0.333	0.0	2.0	2.0	36.9	O K
120 min Summer	0.367	0.367	0.0	2.0	2.0	40.6	Flood Risk
180 min Summer	0.368	0.368	0.0	2.0	2.0	40.7	Flood Risk
240 min Summer	0.361	0.361	0.0	2.0	2.0	39.9	Flood Risk
360 min Summer	0.342	0.342	0.0	2.0	2.0	37.8	O K
480 min Summer	0.321	0.321	0.0	2.0	2.0	35.5	O K
600 min Summer	0.300	0.300	0.0	2.0	2.0	33.3	O K
720 min Summer	0.281	0.281	0.0	2.0	2.0	31.1	O K
960 min Summer	0.244	0.244	0.0	2.0	2.0	27.0	O K
1440 min Summer	0.185	0.185	0.0	2.0	2.0	20.4	O K
2160 min Summer	0.128	0.128	0.0	2.0	2.0	14.2	O K
2880 min Summer	0.098	0.098	0.0	1.9	1.9	10.8	O K
4320 min Summer	0.076	0.076	0.0	1.6	1.6	8.4	O K
5760 min Summer	0.066	0.066	0.0	1.3	1.3	7.3	O K
7200 min Summer	0.060	0.060	0.0	1.2	1.2	6.6	O K
8640 min Summer	0.056	0.056	0.0	1.1	1.1	6.1	O K
10080 min Summer	0.053	0.053	0.0	1.0	1.0	5.8	O K
15 min Winter	0.230	0.230	0.0	2.0	2.0	25.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	116.414	0.0	23.9	21
30 min Summer	78.153	0.0	32.2	35
60 min Summer	50.437	0.0	41.8	64
120 min Summer	30.313	0.0	50.3	122
180 min Summer	22.173	0.0	55.2	174
240 min Summer	17.672	0.0	58.7	200
360 min Summer	12.762	0.0	63.6	260
480 min Summer	10.112	0.0	67.2	326
600 min Summer	8.440	0.0	70.1	392
720 min Summer	7.281	0.0	72.5	458
960 min Summer	5.772	0.0	76.7	588
1440 min Summer	4.178	0.0	83.2	836
2160 min Summer	3.048	0.0	91.2	1176
2880 min Summer	2.456	0.0	98.0	1504
4320 min Summer	1.840	0.0	110.1	2208
5760 min Summer	1.520	0.0	121.4	2936
7200 min Summer	1.326	0.0	132.4	3672
8640 min Summer	1.195	0.0	143.2	4408
10080 min Summer	1.102	0.0	153.8	5136
15 min Winter	116.414	0.0	26.8	21

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Innovyze	Source Control 2017.1	

Summary of Results for 30 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
30 min Winter	0.304	0.304	0.0	2.0	2.0	33.7	O K
60 min Winter	0.379	0.379	0.0	2.0	2.0	41.9	Flood Risk
120 min Winter	0.422	0.422	0.0	2.0	2.0	46.7	Flood Risk
180 min Winter	0.428	0.428	0.0	2.0	2.0	47.4	Flood Risk
240 min Winter	0.421	0.421	0.0	2.0	2.0	46.6	Flood Risk
360 min Winter	0.393	0.393	0.0	2.0	2.0	43.5	Flood Risk
480 min Winter	0.362	0.362	0.0	2.0	2.0	40.0	Flood Risk
600 min Winter	0.330	0.330	0.0	2.0	2.0	36.5	O K
720 min Winter	0.299	0.299	0.0	2.0	2.0	33.1	O K
960 min Winter	0.243	0.243	0.0	2.0	2.0	26.9	O K
1440 min Winter	0.158	0.158	0.0	2.0	2.0	17.5	O K
2160 min Winter	0.094	0.094	0.0	1.9	1.9	10.4	O K
2880 min Winter	0.076	0.076	0.0	1.6	1.6	8.4	O K
4320 min Winter	0.060	0.060	0.0	1.2	1.2	6.6	O K
5760 min Winter	0.053	0.053	0.0	1.0	1.0	5.8	O K
7200 min Winter	0.048	0.048	0.0	0.9	0.9	5.3	O K
8640 min Winter	0.045	0.045	0.0	0.8	0.8	5.0	O K
10080 min Winter	0.043	0.043	0.0	0.7	0.7	4.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
30 min Winter	78.153	0.0	36.1	35
60 min Winter	50.437	0.0	46.9	64
120 min Winter	30.313	0.0	56.4	120
180 min Winter	22.173	0.0	61.8	176
240 min Winter	17.672	0.0	65.7	230
360 min Winter	12.762	0.0	71.2	286
480 min Winter	10.112	0.0	75.2	358
600 min Winter	8.440	0.0	78.5	428
720 min Winter	7.281	0.0	81.3	498
960 min Winter	5.772	0.0	85.9	630
1440 min Winter	4.178	0.0	93.2	868
2160 min Winter	3.048	0.0	102.2	1168
2880 min Winter	2.456	0.0	109.8	1504
4320 min Winter	1.840	0.0	123.3	2212
5760 min Winter	1.520	0.0	136.0	2952
7200 min Winter	1.326	0.0	148.3	3696
8640 min Winter	1.195	0.0	160.4	4344
10080 min Winter	1.102	0.0	172.3	5104

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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		
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Rainfall Details

Rainfall Model	FEH
Return Period (years)	30
FEH Rainfall Version	2013
Site Location	GB 336700 266850 SO 36700 66850
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.111

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	0.056	4	8	0.055

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Model Details

Storage is Online Cover Level (m) 0.660

Cellular Storage Structure

Invert Level (m) 0.000 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	116.5	0.0	0.660	116.5	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0073-2000-0600-2000
Design Head (m)	0.600
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	73
Invert Level (m)	0.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.600	2.0
Flush-Flo™	0.177	2.0
Kick-Flo®	0.397	1.7
Mean Flow over Head Range	-	1.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.9	1.200	2.7	3.000	4.2	7.000	6.3
0.200	2.0	1.400	2.9	3.500	4.5	7.500	6.5
0.300	1.9	1.600	3.1	4.000	4.8	8.000	6.7
0.400	1.7	1.800	3.3	4.500	5.1	8.500	6.9
0.500	1.8	2.000	3.5	5.000	5.3	9.000	7.1
0.600	2.0	2.200	3.6	5.500	5.6	9.500	7.3
0.800	2.3	2.400	3.8	6.000	5.8		
1.000	2.5	2.600	3.9	6.500	6.0		

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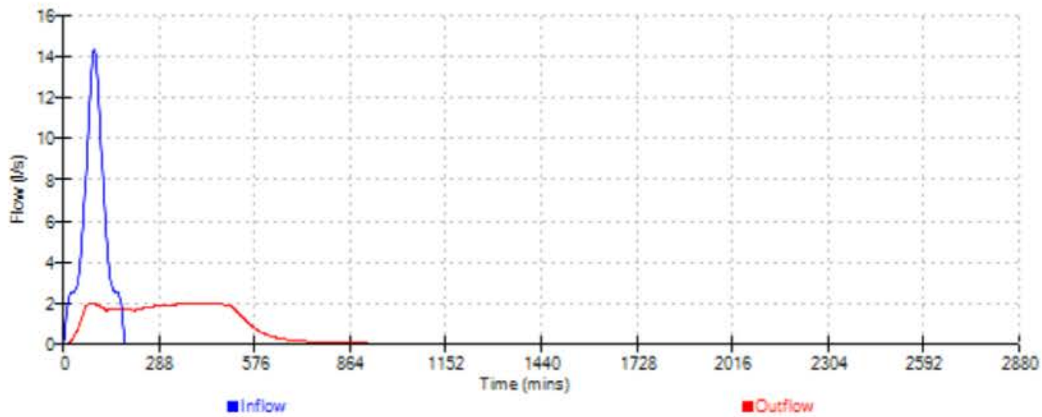
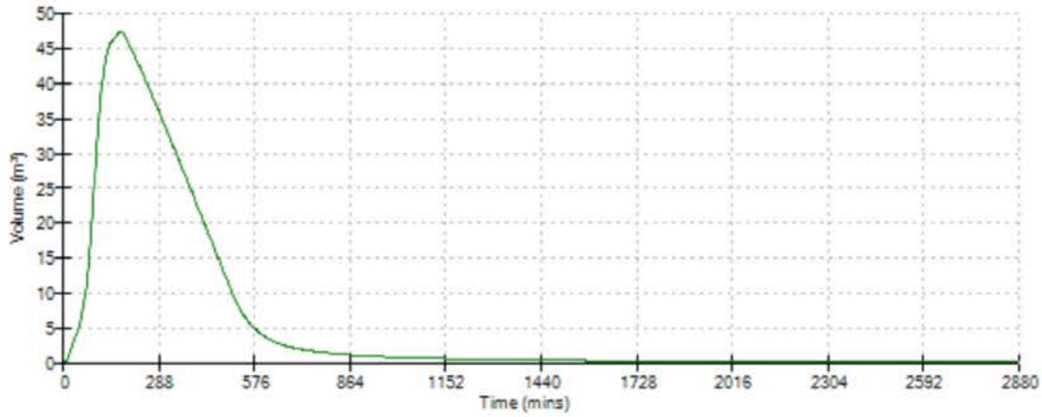
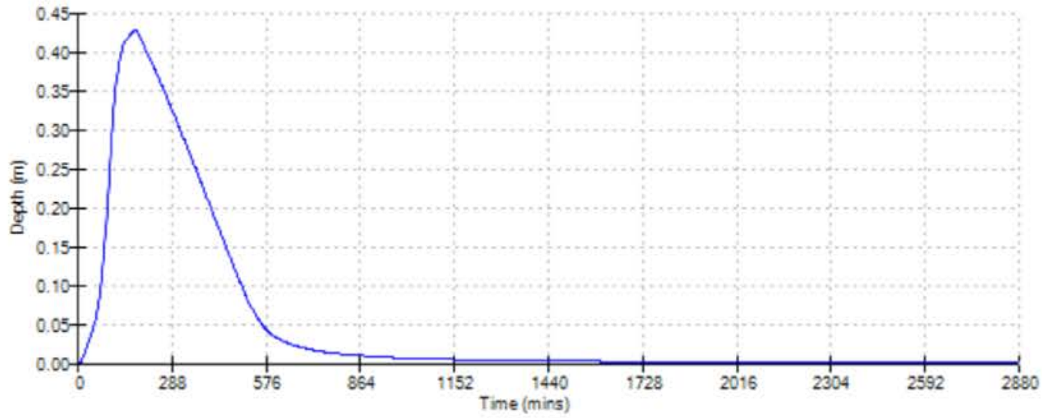
Innovyze	Source Control 2017.1	

Event: 180 min Winter







Hydrologic Services		Page 1
Unit 6, Commerce Park Brunel Way Theale RG7 4AB		
Date 24/07/2019 14:02 File 100yr-Cellular-storage-...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Summary of Results for 100 year Return Period (+40%)

Half Drain Time : 286 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	0.287	0.287	0.0	2.0	2.0	31.8	O K
30 min Summer	0.386	0.386	0.0	2.0	2.0	42.8	Flood Risk
60 min Summer	0.489	0.489	0.0	2.0	2.0	54.1	Flood Risk
120 min Summer	0.528	0.528	0.0	2.0	2.0	58.5	Flood Risk
180 min Summer	0.529	0.529	0.0	2.0	2.0	58.5	Flood Risk
240 min Summer	0.516	0.516	0.0	2.0	2.0	57.1	Flood Risk
360 min Summer	0.484	0.484	0.0	2.0	2.0	53.6	Flood Risk
480 min Summer	0.455	0.455	0.0	2.0	2.0	50.3	Flood Risk
600 min Summer	0.427	0.427	0.0	2.0	2.0	47.2	Flood Risk
720 min Summer	0.399	0.399	0.0	2.0	2.0	44.2	Flood Risk
960 min Summer	0.345	0.345	0.0	2.0	2.0	38.2	O K
1440 min Summer	0.263	0.263	0.0	2.0	2.0	29.1	O K
2160 min Summer	0.180	0.180	0.0	2.0	2.0	19.9	O K
2880 min Summer	0.130	0.130	0.0	2.0	2.0	14.4	O K
4320 min Summer	0.088	0.088	0.0	1.8	1.8	9.8	O K
5760 min Summer	0.075	0.075	0.0	1.5	1.5	8.3	O K
7200 min Summer	0.067	0.067	0.0	1.4	1.4	7.4	O K
8640 min Summer	0.061	0.061	0.0	1.2	1.2	6.8	O K
10080 min Summer	0.058	0.058	0.0	1.1	1.1	6.4	O K
15 min Winter	0.323	0.323	0.0	2.0	2.0	35.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	160.720	0.0	33.1	22
30 min Summer	109.480	0.0	45.2	36
60 min Summer	71.400	0.0	59.3	66
120 min Summer	41.510	0.0	68.9	124
180 min Summer	29.789	0.0	74.2	182
240 min Summer	23.415	0.0	77.8	232
360 min Summer	16.567	0.0	82.6	288
480 min Summer	12.930	0.0	85.9	350
600 min Summer	10.665	0.0	88.6	418
720 min Summer	9.112	0.0	90.8	488
960 min Summer	7.113	0.0	94.5	612
1440 min Summer	5.052	0.0	100.7	866
2160 min Summer	3.625	0.0	108.5	1216
2880 min Summer	2.890	0.0	115.4	1556
4320 min Summer	2.141	0.0	128.1	2212
5760 min Summer	1.759	0.0	140.5	2944
7200 min Summer	1.528	0.0	152.6	3672
8640 min Summer	1.374	0.0	164.6	4408
10080 min Summer	1.264	0.0	176.6	5136
15 min Winter	160.720	0.0	37.1	22

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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
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Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
30 min Winter	0.436	0.436	0.0	2.0	2.0	48.2	Flood Risk
60 min Winter	0.552	0.552	0.0	2.0	2.0	61.0	Flood Risk
120 min Winter	0.600	0.600	0.0	2.0	2.0	66.4	Flood Risk
180 min Winter	0.605	0.605	0.0	2.0	2.0	67.0	Flood Risk
240 min Winter	0.594	0.594	0.0	2.0	2.0	65.7	Flood Risk
360 min Winter	0.554	0.554	0.0	2.0	2.0	61.3	Flood Risk
480 min Winter	0.518	0.518	0.0	2.0	2.0	57.4	Flood Risk
600 min Winter	0.482	0.482	0.0	2.0	2.0	53.4	Flood Risk
720 min Winter	0.447	0.447	0.0	2.0	2.0	49.5	Flood Risk
960 min Winter	0.370	0.370	0.0	2.0	2.0	41.0	Flood Risk
1440 min Winter	0.246	0.246	0.0	2.0	2.0	27.2	O K
2160 min Winter	0.134	0.134	0.0	2.0	2.0	14.9	O K
2880 min Winter	0.090	0.090	0.0	1.8	1.8	10.0	O K
4320 min Winter	0.068	0.068	0.0	1.4	1.4	7.5	O K
5760 min Winter	0.058	0.058	0.0	1.2	1.2	6.4	O K
7200 min Winter	0.053	0.053	0.0	1.0	1.0	5.8	O K
8640 min Winter	0.049	0.049	0.0	0.9	0.9	5.4	O K
10080 min Winter	0.047	0.047	0.0	0.8	0.8	5.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
30 min Winter	109.480	0.0	50.6	36
60 min Winter	71.400	0.0	66.4	64
120 min Winter	41.510	0.0	77.2	122
180 min Winter	29.789	0.0	83.1	178
240 min Winter	23.415	0.0	87.1	232
360 min Winter	16.567	0.0	92.5	304
480 min Winter	12.930	0.0	96.2	372
600 min Winter	10.665	0.0	99.2	450
720 min Winter	9.112	0.0	101.7	528
960 min Winter	7.113	0.0	105.9	670
1440 min Winter	5.052	0.0	112.8	914
2160 min Winter	3.625	0.0	121.6	1236
2880 min Winter	2.890	0.0	129.2	1528
4320 min Winter	2.141	0.0	143.5	2244
5760 min Winter	1.759	0.0	157.4	2936
7200 min Winter	1.528	0.0	170.9	3672
8640 min Winter	1.374	0.0	184.4	4408
10080 min Winter	1.264	0.0	197.8	5136

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 336700 266850 SO 36700 66850
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.111

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	0.056	4	8	0.055

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Innovyze	Source Control 2017.1	

Model Details

Storage is Online Cover Level (m) 0.660

Cellular Storage Structure

Invert Level (m) 0.000 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	**Area (m²)**	**Inf. Area (m²)**	**Depth (m)**	**Area (m²)**	**Inf. Area (m²)**
0.000	116.5	0.0	0.660	116.5	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0073-2000-0600-2000
Design Head (m)	0.600
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	73
Invert Level (m)	0.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	0.600	2.0
Flush-Flo™	0.177	2.0
Kick-Flo®	0.397	1.7
Mean Flow over Head Range	-	1.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**
0.100	1.9	1.200	2.7	3.000	4.2	7.000	6.3
0.200	2.0	1.400	2.9	3.500	4.5	7.500	6.5
0.300	1.9	1.600	3.1	4.000	4.8	8.000	6.7
0.400	1.7	1.800	3.3	4.500	5.1	8.500	6.9
0.500	1.8	2.000	3.5	5.000	5.3	9.000	7.1
0.600	2.0	2.200	3.6	5.500	5.6	9.500	7.3
0.800	2.3	2.400	3.8	6.000	5.8		
1.000	2.5	2.600	3.9	6.500	6.0		

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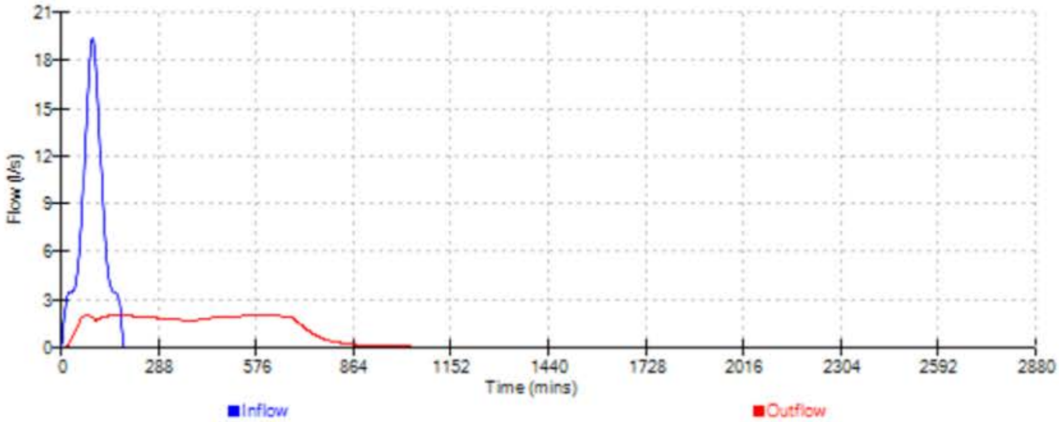
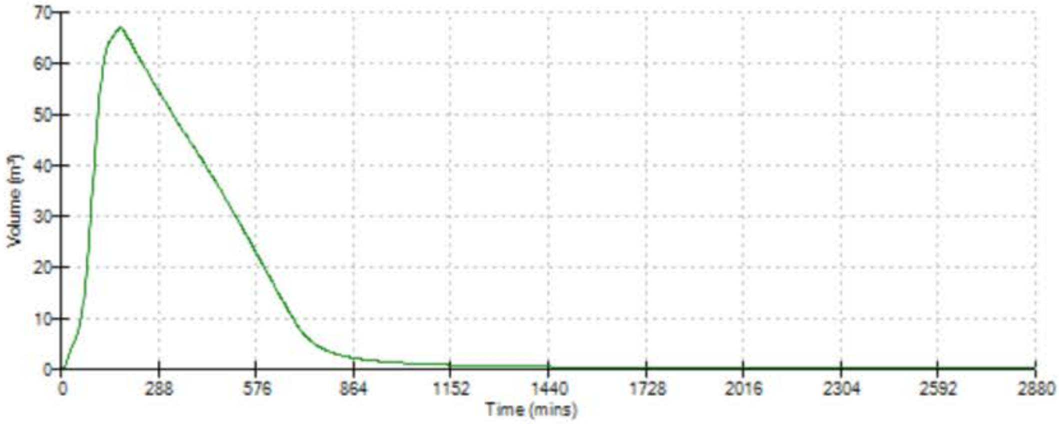
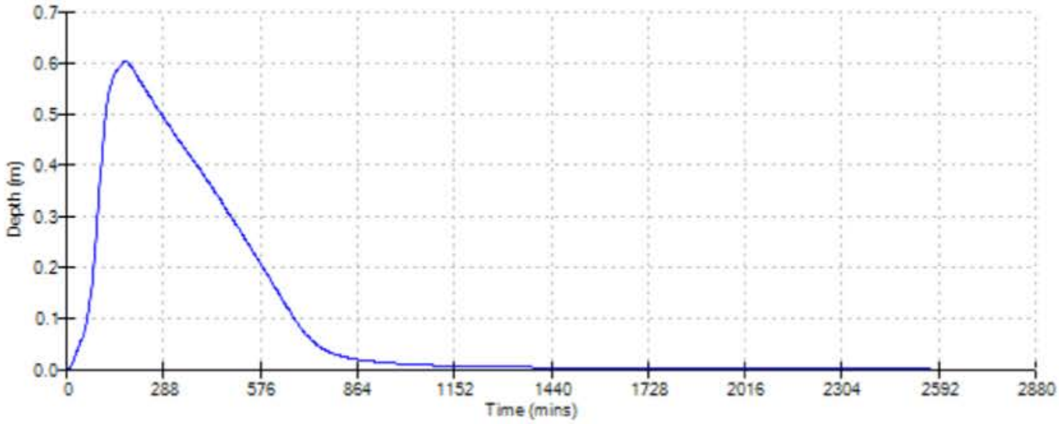
Unit 6, Commerce Park Brunel Way Theale RG7 4AB		
Date 24/07/2019 14:02 File 100yr-Cellular-storage-...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Event: 180 min Winter







C2. Pipe Storage Output for Access Road

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Date 24/07/2019 12:26 File Access-Road-2year+40%-A...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Summary of Results for 2 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	0.184	0.184	2.0	2.9	O K
30 min Summer	0.191	0.191	2.0	3.1	O K
60 min Summer	0.189	0.189	2.0	3.0	O K
120 min Summer	0.188	0.188	2.0	3.0	O K
180 min Summer	0.177	0.177	2.0	2.6	O K
240 min Summer	0.163	0.163	2.0	2.2	O K
360 min Summer	0.136	0.136	1.9	1.5	O K
480 min Summer	0.112	0.112	1.9	1.0	O K
600 min Summer	0.095	0.095	1.8	0.7	O K
720 min Summer	0.085	0.085	1.7	0.5	O K
960 min Summer	0.071	0.071	1.4	0.4	O K
1440 min Summer	0.056	0.056	1.1	0.3	O K
2160 min Summer	0.047	0.047	0.8	0.2	O K
2880 min Summer	0.041	0.041	0.7	0.1	O K
4320 min Summer	0.036	0.036	0.5	0.1	O K
5760 min Summer	0.032	0.032	0.4	0.1	O K
7200 min Summer	0.030	0.030	0.4	0.1	O K
8640 min Summer	0.028	0.028	0.3	0.1	O K
10080 min Summer	0.027	0.027	0.3	0.1	O K
15 min Winter	0.198	0.198	2.0	3.3	O K
30 min Winter	0.205	0.205	2.0	3.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	43.684	0.0	4.3	15
30 min Summer	28.548	0.0	5.6	24
60 min Summer	18.373	0.0	7.2	40
120 min Summer	12.695	0.0	9.9	76
180 min Summer	10.011	0.0	11.7	108
240 min Summer	8.396	0.0	13.1	138
360 min Summer	6.490	0.0	15.2	198
480 min Summer	5.378	0.0	16.8	256
600 min Summer	4.635	0.0	18.1	310
720 min Summer	4.096	0.0	19.2	370
960 min Summer	3.359	0.0	21.0	490
1440 min Summer	2.530	0.0	23.7	734
2160 min Summer	1.902	0.0	26.7	1084
2880 min Summer	1.559	0.0	29.2	1468
4320 min Summer	1.191	0.0	33.4	2188
5760 min Summer	0.994	0.0	37.2	2872
7200 min Summer	0.873	0.0	40.9	3608
8640 min Summer	0.791	0.0	44.4	4400
10080 min Summer	0.731	0.0	47.9	4984
15 min Winter	43.684	0.0	4.8	15
30 min Winter	28.548	0.0	6.2	25

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Innovyze	Source Control 2017.1	

Summary of Results for 2 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	0.200	0.200	2.0	3.4	O K
120 min Winter	0.189	0.189	2.0	3.0	O K
180 min Winter	0.167	0.167	2.0	2.3	O K
240 min Winter	0.144	0.144	2.0	1.7	O K
360 min Winter	0.102	0.102	1.8	0.8	O K
480 min Winter	0.082	0.082	1.6	0.5	O K
600 min Winter	0.071	0.071	1.4	0.4	O K
720 min Winter	0.064	0.064	1.3	0.3	O K
960 min Winter	0.055	0.055	1.0	0.2	O K
1440 min Winter	0.046	0.046	0.8	0.2	O K
2160 min Winter	0.039	0.039	0.6	0.1	O K
2880 min Winter	0.034	0.034	0.5	0.1	O K
4320 min Winter	0.030	0.030	0.4	0.1	O K
5760 min Winter	0.027	0.027	0.3	0.1	O K
7200 min Winter	0.025	0.025	0.3	0.1	O K
8640 min Winter	0.024	0.024	0.2	0.1	O K
10080 min Winter	0.023	0.023	0.2	0.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	18.373	0.0	8.0	44
120 min Winter	12.695	0.0	11.1	80
180 min Winter	10.011	0.0	13.1	114
240 min Winter	8.396	0.0	14.7	144
360 min Winter	6.490	0.0	17.0	198
480 min Winter	5.378	0.0	18.8	250
600 min Winter	4.635	0.0	20.2	308
720 min Winter	4.096	0.0	21.5	368
960 min Winter	3.359	0.0	23.5	490
1440 min Winter	2.530	0.0	26.5	718
2160 min Winter	1.902	0.0	29.9	1088
2880 min Winter	1.559	0.0	32.7	1424
4320 min Winter	1.191	0.0	37.4	2148
5760 min Winter	0.994	0.0	41.7	2912
7200 min Winter	0.873	0.0	45.8	3656
8640 min Winter	0.791	0.0	49.7	4304
10080 min Winter	0.731	0.0	53.6	5136

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Innovyze	Source Control 2017.1	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	2
FEH Rainfall Version	2013
Site Location	GB 336700 266850 SO 36700 66850
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.052

Time From:	**(mins) To:**	**Area (ha)**
0	4	0.052

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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
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Model Details

Storage is Online Cover Level (m) 2.000

Pipe Structure

Diameter (m) Conduit Section   Length (m) 30.000
Slope (1:X)   500.000 Invert Level (m) 0.000

Section Number -1 Minor Dimn (mm) 750 4 * Hyd Radius (mm) 0.750
Conduit Type oo Side Slope (Deg)   XSect Area (m²) 0.884
Major Dimn (mm) 1500 Corner Splay (mm)

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0071-2000-0700-2000
Design Head (m)	0.700
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	71
Invert Level (m)	0.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	0.700	2.0
Flush-Flo™	0.207	2.0
Kick-Flo®	0.450	1.6
Mean Flow over Head Range	-	1.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**
0.100	1.8	1.200	2.6	3.000	3.9	7.000	5.8
0.200	2.0	1.400	2.7	3.500	4.2	7.500	6.0
0.300	2.0	1.600	2.9	4.000	4.5	8.000	6.2
0.400	1.8	1.800	3.1	4.500	4.7	8.500	6.4
0.500	1.7	2.000	3.2	5.000	5.0	9.000	6.6
0.600	1.9	2.200	3.4	5.500	5.2	9.500	6.8
0.800	2.1	2.400	3.5	6.000	5.4		
1.000	2.4	2.600	3.7	6.500	5.6		

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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
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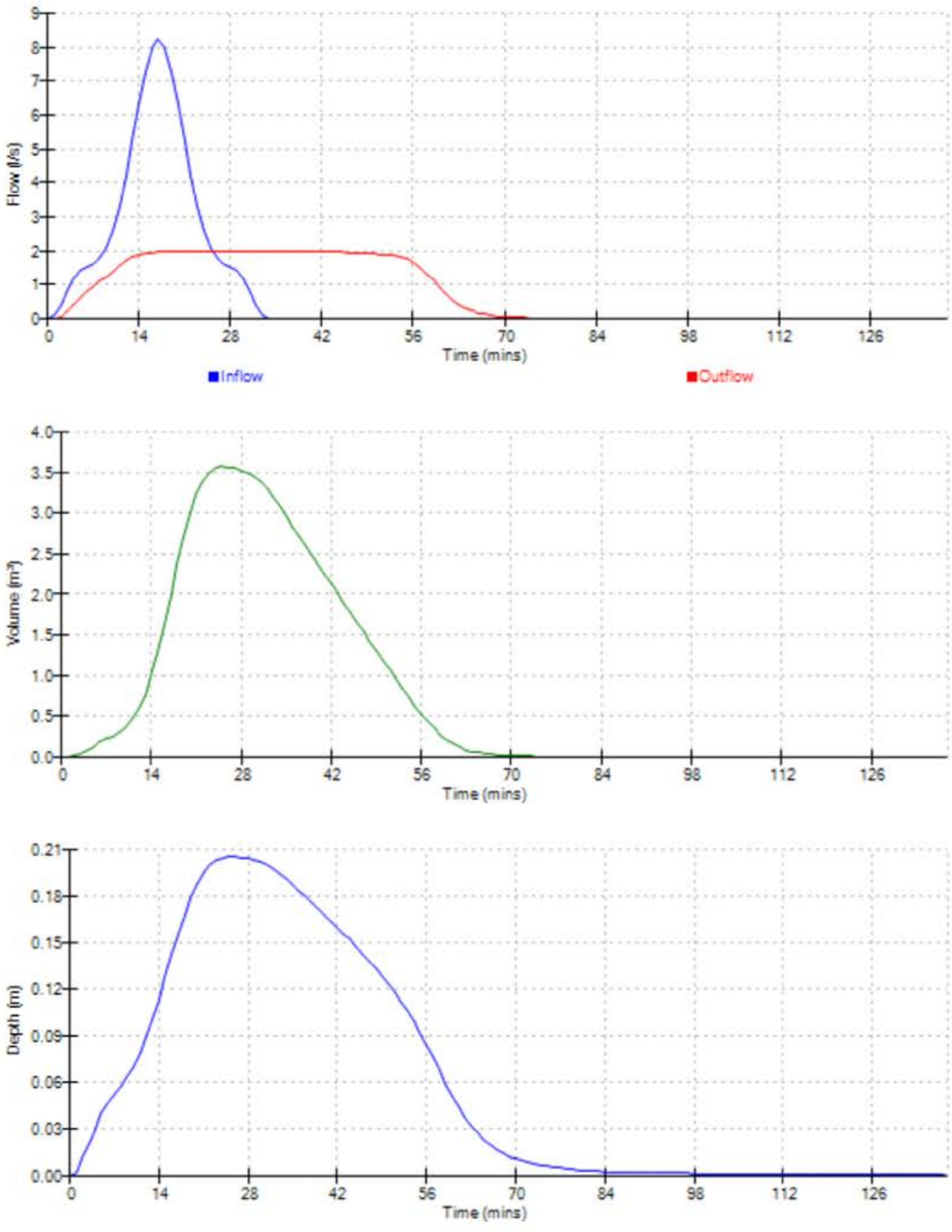
Event: 30 min Winter



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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
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Innovyze	Source Control 2017.1	

Summary of Results for 30 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	0.358	0.358	2.0	9.6	O K
30 min Summer	0.412	0.412	2.0	12.0	O K
60 min Summer	0.447	0.447	2.0	13.6	O K
120 min Summer	0.429	0.429	2.0	12.8	O K
180 min Summer	0.404	0.404	2.0	11.7	O K
240 min Summer	0.379	0.379	2.0	10.5	O K
360 min Summer	0.328	0.328	2.0	8.3	O K
480 min Summer	0.281	0.281	2.0	6.4	O K
600 min Summer	0.239	0.239	2.0	4.8	O K
720 min Summer	0.203	0.203	2.0	3.5	O K
960 min Summer	0.148	0.148	2.0	1.8	O K
1440 min Summer	0.090	0.090	1.7	0.6	O K
2160 min Summer	0.066	0.066	1.3	0.3	O K
2880 min Summer	0.055	0.055	1.0	0.2	O K
4320 min Summer	0.046	0.046	0.8	0.2	O K
5760 min Summer	0.041	0.041	0.6	0.1	O K
7200 min Summer	0.038	0.038	0.6	0.1	O K
8640 min Summer	0.036	0.036	0.5	0.1	O K
10080 min Summer	0.034	0.034	0.5	0.1	O K
15 min Winter	0.389	0.389	2.0	11.0	O K
30 min Winter	0.455	0.455	2.0	13.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	116.414	0.0	11.3	17
30 min Summer	78.153	0.0	15.2	32
60 min Summer	50.437	0.0	19.7	60
120 min Summer	30.313	0.0	23.6	92
180 min Summer	22.173	0.0	25.9	124
240 min Summer	17.672	0.0	27.6	158
360 min Summer	12.762	0.0	29.9	222
480 min Summer	10.112	0.0	31.5	286
600 min Summer	8.440	0.0	32.9	344
720 min Summer	7.281	0.0	34.1	400
960 min Summer	5.772	0.0	36.0	510
1440 min Summer	4.178	0.0	39.1	734
2160 min Summer	3.048	0.0	42.8	1092
2880 min Summer	2.456	0.0	46.0	1428
4320 min Summer	1.840	0.0	51.7	2188
5760 min Summer	1.520	0.0	56.9	2856
7200 min Summer	1.326	0.0	62.1	3656
8640 min Summer	1.195	0.0	67.1	4312
10080 min Summer	1.102	0.0	72.2	5136
15 min Winter	116.414	0.0	12.7	17
30 min Winter	78.153	0.0	17.1	31

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Summary of Results for 30 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	0.501	0.501	2.0	16.0	O K
120 min Winter	0.482	0.482	2.0	15.1	O K
180 min Winter	0.448	0.448	2.0	13.6	O K
240 min Winter	0.407	0.407	2.0	11.8	O K
360 min Winter	0.328	0.328	2.0	8.3	O K
480 min Winter	0.256	0.256	2.0	5.4	O K
600 min Winter	0.195	0.195	2.0	3.2	O K
720 min Winter	0.147	0.147	2.0	1.8	O K
960 min Winter	0.090	0.090	1.8	0.6	O K
1440 min Winter	0.065	0.065	1.3	0.3	O K
2160 min Winter	0.051	0.051	0.9	0.2	O K
2880 min Winter	0.045	0.045	0.8	0.2	O K
4320 min Winter	0.038	0.038	0.6	0.1	O K
5760 min Winter	0.034	0.034	0.5	0.1	O K
7200 min Winter	0.031	0.031	0.4	0.1	O K
8640 min Winter	0.030	0.030	0.4	0.1	O K
10080 min Winter	0.028	0.028	0.3	0.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	50.437	0.0	22.0	60
120 min Winter	30.313	0.0	26.5	100
180 min Winter	22.173	0.0	29.1	138
240 min Winter	17.672	0.0	30.9	172
360 min Winter	12.762	0.0	33.4	240
480 min Winter	10.112	0.0	35.3	300
600 min Winter	8.440	0.0	36.9	356
720 min Winter	7.281	0.0	38.2	404
960 min Winter	5.772	0.0	40.3	492
1440 min Winter	4.178	0.0	43.8	730
2160 min Winter	3.048	0.0	47.9	1100
2880 min Winter	2.456	0.0	51.5	1456
4320 min Winter	1.840	0.0	57.9	2160
5760 min Winter	1.520	0.0	63.8	2840
7200 min Winter	1.326	0.0	69.5	3624
8640 min Winter	1.195	0.0	75.2	4408
10080 min Winter	1.102	0.0	80.8	5000

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	30
FEH Rainfall Version	2013
Site Location	GB 336700 266850 SO 36700 66850
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.052

Time (mins) From:	To:	Area (ha)
0	4	0.052

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Model Details

Storage is Online Cover Level (m) 2.000

Pipe Structure

Diameter (m) Conduit Section Length (m) 30.000
Slope (1:X) 500.000 Invert Level (m) 0.000

Section Number -1 Minor Dimn (mm) 750 4 * Hyd Radius (mm) 0.750
Conduit Type oo Side Slope (Deg) XSect Area (m²) 0.884
Major Dimn (mm) 1500 Corner Splay (mm)

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0071-2000-0700-2000
Design Head (m)	0.700
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	71
Invert Level (m)	0.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	0.700	2.0
Flush-Flo™	0.207	2.0
Kick-Flo®	0.450	1.6
Mean Flow over Head Range	-	1.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**
0.100	1.8	1.200	2.6	3.000	3.9	7.000	5.8
0.200	2.0	1.400	2.7	3.500	4.2	7.500	6.0
0.300	2.0	1.600	2.9	4.000	4.5	8.000	6.2
0.400	1.8	1.800	3.1	4.500	4.7	8.500	6.4
0.500	1.7	2.000	3.2	5.000	5.0	9.000	6.6
0.600	1.9	2.200	3.4	5.500	5.2	9.500	6.8
0.800	2.1	2.400	3.5	6.000	5.4		
1.000	2.4	2.600	3.7	6.500	5.6		

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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
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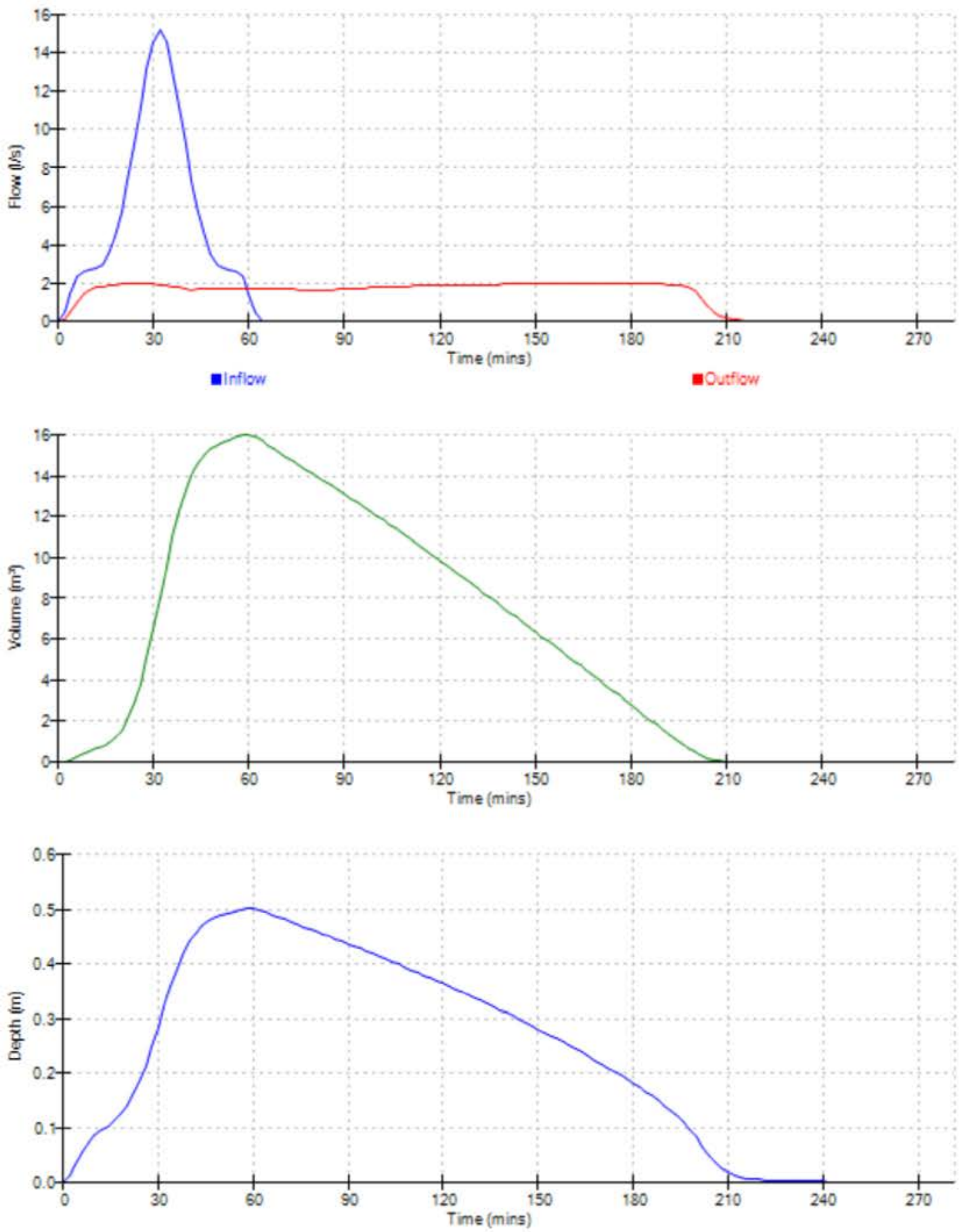
Innovyze	Source Control 2017.1	

Event: 60 min Winter



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Unit 6, Commerce Park Brunel Way Theale RG7 4AB		
Date 24/07/2019 12:21 File Access-Road-100year+40%...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	0.455	0.455	2.0	13.9	O K
30 min Summer	0.547	0.547	2.0	18.0	O K
60 min Summer	0.625	0.625	2.0	21.3	O K
120 min Summer	0.595	0.595	2.0	20.1	O K
180 min Summer	0.558	0.558	2.0	18.5	O K
240 min Summer	0.525	0.525	2.0	17.1	O K
360 min Summer	0.464	0.464	2.0	14.3	O K
480 min Summer	0.399	0.399	2.0	11.4	O K
600 min Summer	0.346	0.346	2.0	9.1	O K
720 min Summer	0.298	0.298	2.0	7.1	O K
960 min Summer	0.219	0.219	2.0	4.1	O K
1440 min Summer	0.125	0.125	1.9	1.2	O K
2160 min Summer	0.078	0.078	1.5	0.4	O K
2880 min Summer	0.063	0.063	1.2	0.3	O K
4320 min Summer	0.051	0.051	0.9	0.2	O K
5760 min Summer	0.045	0.045	0.8	0.2	O K
7200 min Summer	0.041	0.041	0.7	0.1	O K
8640 min Summer	0.039	0.039	0.6	0.1	O K
10080 min Summer	0.037	0.037	0.5	0.1	O K
15 min Winter	0.497	0.497	2.0	15.8	O K
30 min Winter	0.609	0.609	2.0	20.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	160.720	0.0	15.7	18
30 min Summer	109.480	0.0	21.3	32
60 min Summer	71.400	0.0	27.8	62
120 min Summer	41.510	0.0	32.4	104
180 min Summer	29.789	0.0	34.9	134
240 min Summer	23.415	0.0	36.5	168
360 min Summer	16.567	0.0	38.8	238
480 min Summer	12.930	0.0	40.3	300
600 min Summer	10.665	0.0	41.6	360
720 min Summer	9.112	0.0	42.6	420
960 min Summer	7.113	0.0	44.4	530
1440 min Summer	5.052	0.0	47.3	750
2160 min Summer	3.625	0.0	50.9	1100
2880 min Summer	2.890	0.0	54.1	1460
4320 min Summer	2.141	0.0	60.1	2180
5760 min Summer	1.759	0.0	65.8	2856
7200 min Summer	1.528	0.0	71.5	3576
8640 min Summer	1.374	0.0	77.2	4320
10080 min Summer	1.264	0.0	82.8	5064
15 min Winter	160.720	0.0	17.6	18
30 min Winter	109.480	0.0	23.9	32

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Innovyze	Source Control 2017.1	

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	0.719	0.719	2.0	24.6	O K
120 min Winter	0.689	0.689	2.0	23.7	O K
180 min Winter	0.634	0.634	2.0	21.6	O K
240 min Winter	0.587	0.587	2.0	19.7	O K
360 min Winter	0.498	0.498	2.0	15.9	O K
480 min Winter	0.403	0.403	2.0	11.6	O K
600 min Winter	0.321	0.321	2.0	8.0	O K
720 min Winter	0.251	0.251	2.0	5.2	O K
960 min Winter	0.148	0.148	2.0	1.8	O K
1440 min Winter	0.078	0.078	1.5	0.5	O K
2160 min Winter	0.058	0.058	1.1	0.3	O K
2880 min Winter	0.050	0.050	0.9	0.2	O K
4320 min Winter	0.041	0.041	0.7	0.1	O K
5760 min Winter	0.037	0.037	0.5	0.1	O K
7200 min Winter	0.034	0.034	0.5	0.1	O K
8640 min Winter	0.032	0.032	0.4	0.1	O K
10080 min Winter	0.031	0.031	0.4	0.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	71.400	0.0	31.2	60
120 min Winter	41.510	0.0	36.3	114
180 min Winter	29.789	0.0	39.0	142
240 min Winter	23.415	0.0	40.9	180
360 min Winter	16.567	0.0	43.4	258
480 min Winter	12.930	0.0	45.2	324
600 min Winter	10.665	0.0	46.6	382
720 min Winter	9.112	0.0	47.8	436
960 min Winter	7.113	0.0	49.7	530
1440 min Winter	5.052	0.0	53.0	734
2160 min Winter	3.625	0.0	57.0	1092
2880 min Winter	2.890	0.0	60.6	1460
4320 min Winter	2.141	0.0	67.3	2192
5760 min Winter	1.759	0.0	73.7	2864
7200 min Winter	1.528	0.0	80.1	3568
8640 min Winter	1.374	0.0	86.4	4288
10080 min Winter	1.264	0.0	92.8	4976

Hydrologic Services		Page 3
Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
Date 24/07/2019 12:21 File Access-Road-100year+40%...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 336700 266850 SO 36700 66850
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.052

Time From:	**(mins) To:**	**Area (ha)**
0	4	0.052

Hydro-Logic Services (International) Ltd Registered in England No. 03036787 Registered Office: Shearwater House, Clevedon Hall Estate, Victoria Road, Clevedon, BS21 7RD

Hydrologic Services		Page 4
Unit 6, Commerce Park Brunel Way Theale RG7 4AB		Micro Drainage
Date 24/07/2019 12:21 File Access-Road-100year+40%...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Model Details

Storage is Online Cover Level (m) 2.000

Pipe Structure

Diameter (m)	Conduit Section	Length (m)	30.000
Slope (1:X)	500.000	Invert Level (m)	0.000

Section Number	-1	Minor Dimn (mm)	750	4 * Hyd Radius (mm)	0.750
Conduit Type	oo	Side Slope (Deg)		XSect Area (m²)	0.884
Major Dimn (mm)	1500	Corner Splay (mm)			

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0071-2000-0700-2000
Design Head (m)	0.700
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	71
Invert Level (m)	0.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

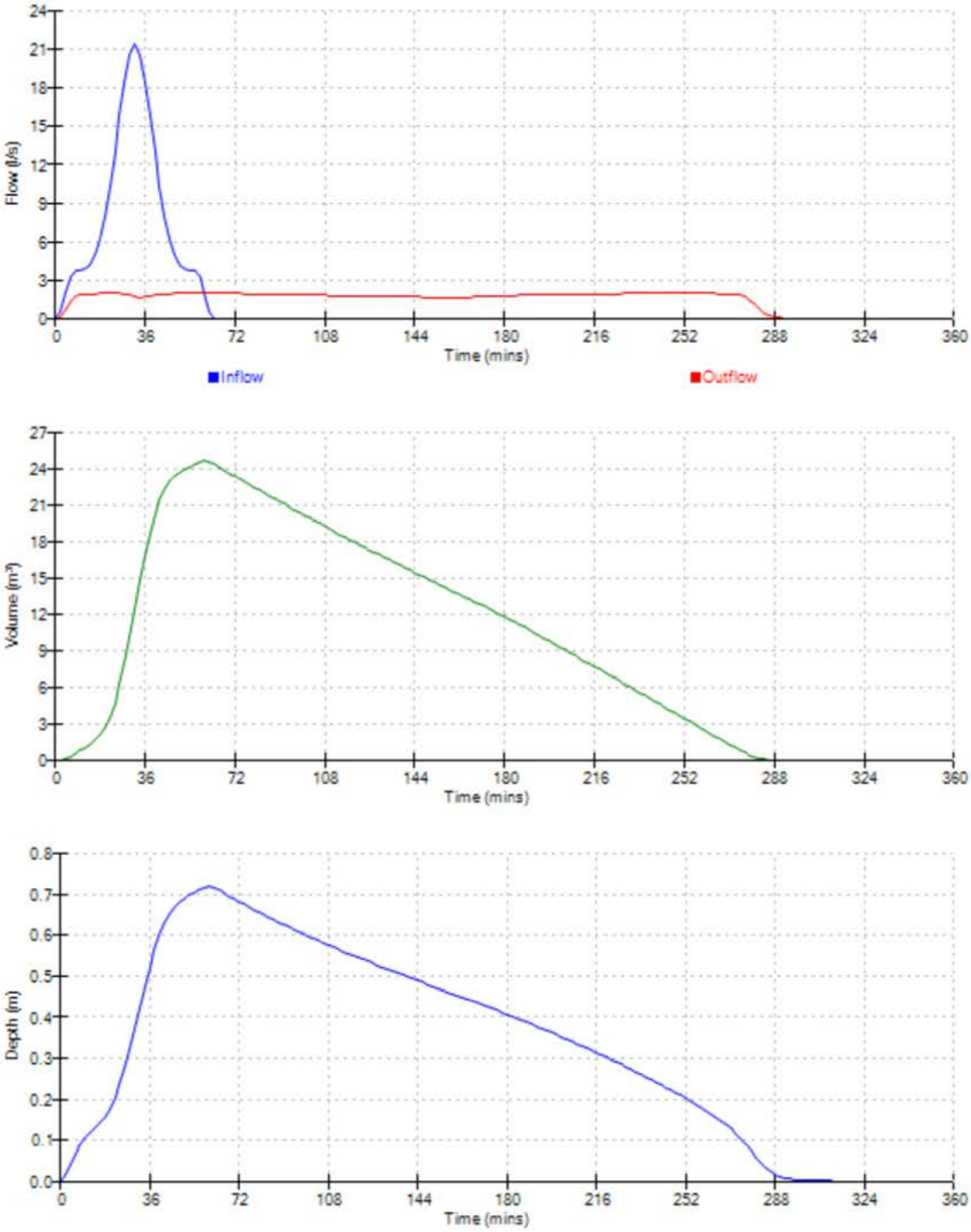
Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.700	2.0
Flush-Flo™	0.207	2.0
Kick-Flo®	0.450	1.6
Mean Flow over Head Range	-	1.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.8	1.200	2.6	3.000	3.9	7.000	5.8
0.200	2.0	1.400	2.7	3.500	4.2	7.500	6.0
0.300	2.0	1.600	2.9	4.000	4.5	8.000	6.2
0.400	1.8	1.800	3.1	4.500	4.7	8.500	6.4
0.500	1.7	2.000	3.2	5.000	5.0	9.000	6.6
0.600	1.9	2.200	3.4	5.500	5.2	9.500	6.8
0.800	2.1	2.400	3.5	6.000	5.4		
1.000	2.4	2.600	3.7	6.500	5.6		

Hydrologic Services		Page 5
Unit 6, Commerce Park Brunel Way Theale RG7 4AB		
Date 24/07/2019 12:21 File Access-Road-100year+40%...	Designed by mahmed Checked by	
Innovyze	Source Control 2017.1	

Event: 60 min Winter



APPENDIX D HYDROLOGICAL INVESTIGATIONS

General

Flood calculations have been undertaken in accordance with the Flood Estimation Handbook (CEH, 2013) and subsequent guidance using the following software and data:

- WinFAP FEHv4 (WHS 2016)
- ReFH2 (WHS 2015)
- FEH website (CEH 2018)
- NRFA HiFlows dataset v6 (CEH Feb 2018)

The various methods are described in this Appendix.

D.2 Catchment Descriptors

As discussed in the main report the Lingen Nursery site is beside the Lime Brook just downstream of a confluence of two tributaries. The following descriptors have been obtained from the FEH Website (CEH, 2018) for the Lime Brook downstream of the confluence and the individual tributaries.

Figure D-1 Catchment area of the Lime Brook Downstream of the Site (FEH Website)

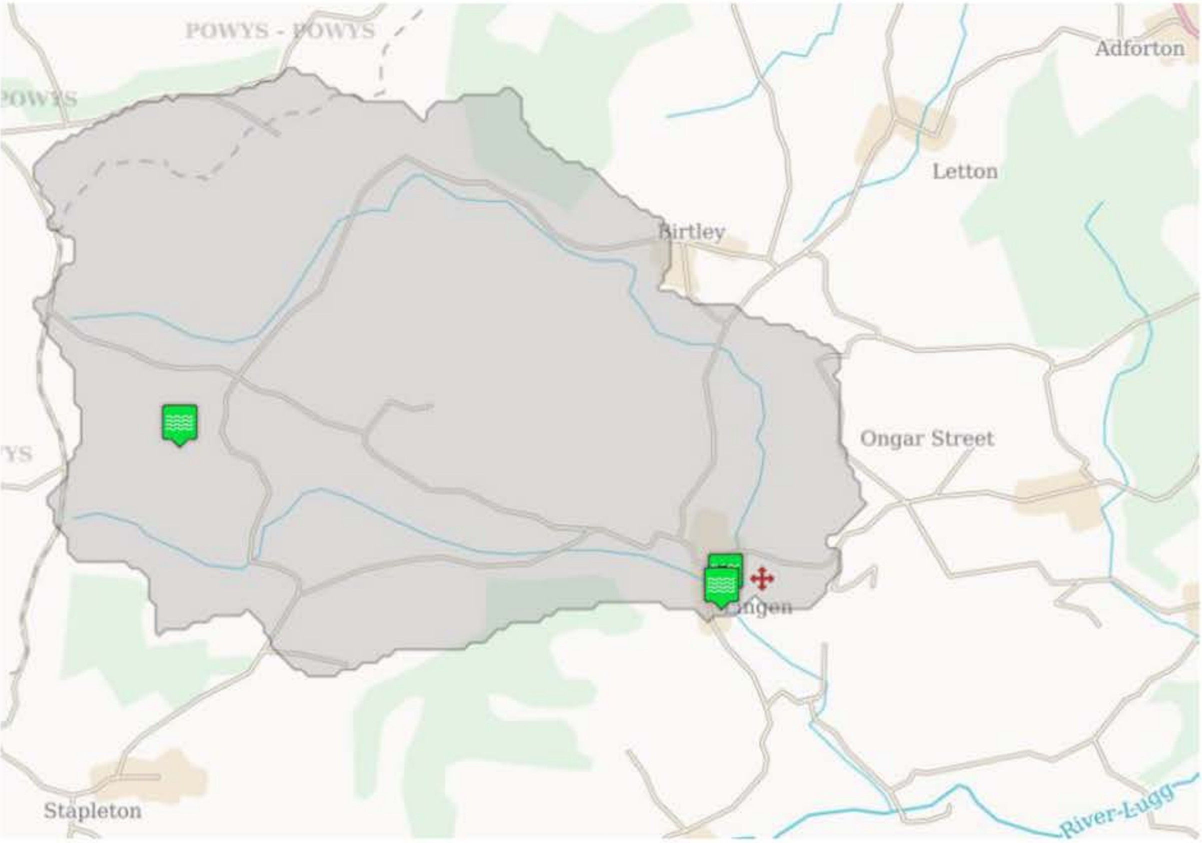


Figure D-2 Catchment area of the Northern Tributary Upstream of the Site (FEH Website)

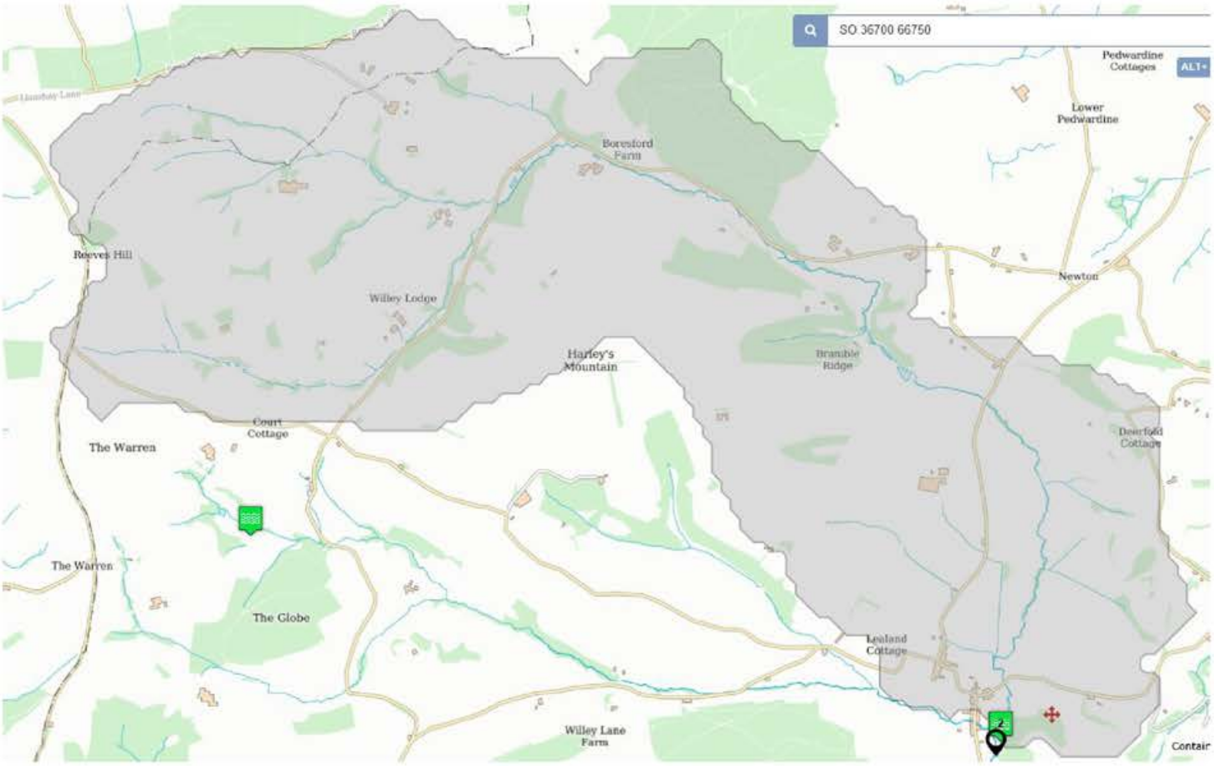


Figure D-3 Catchment area of the Western Tributary Upstream of the Site (FEH Website)

Table D-1 Catchment Descriptors for the catchments

	Location:	Lingen		
	River:	Lime Brook, downstream of site	Northern Tributary	Western Tributary,
	Reference:	Lingen Combined	Lingen North	Lingen West
	NGR:	SO 36700 66750	SO 36750 66850	SO 36700 66850
AREA	Catchment area (km2)	19.545	11.7225	7.7725
ALTBAR	Mean elevation (m)	264	261	268
ASPBAR	Mean aspect	112	111	113
ASPVAR	Variance of aspect	0.29	0.28	0.31
BFIHOST	Base flow index	0.645	0.608	0.702
DPLBAR	Mean drainage path length (km)	4.42	4.81	3.57
DPSBAR	Mean drainage path slope	154.6	146.7	166.9
FARL	Index of lakes	1	1	1
LDP	Longest drainage path (km)	0.0207	0.0244	0.0144
PROPWET	Proportion of time soil is wet	0.224	0.218	0.222
RMED-1H	Median 1 hour rainfall (mm)	0.578	0.534	0.631
RMED-1D	Median 1 day rainfall (mm)	9.26	9.13	6.51
RMED-2D	Median 2 day rainfall (mm)	0.49	0.49	0.49
SAAR	Average annual rainfall (mm)	10.2	10.1	10.3
SAAR4170	Ditto for 1941-1970 (mm)	33.8	33.7	33.9
SPRHOST	Percentage runoff	44.3	44.2	44.5
URBCONC1990	Urban concentration1990	882	877	890
URBEXT1990	Urban extent 1990	975	974	977
URBLOC1990	Urban location 1990	22.27	26.53	15.68
URBEXT2000	Urban extent 2000	0.0006	0.0004	0.0008
QMEDcds	(m3/s)	3.462	2.553	1.271

Source: FEH Website (Centre for Ecology and Hydrology, 2018)

D.3 WINFAP-FEH

D.3.1 Introduction

Flood frequency calculations were undertaken for the development site using the WinFAP FEHv4 software (WHS, 2016) in order to investigate 50%, 1% and 0.1% flood flows. The Environment Agency has stipulated that the flood risk assessment should be based on the FEH procedures. This section summarises hydrological analysis carried out to estimate the design flood peaks for the Lime Brook. The FEH statistical approach is adopted, whereby the design flood peak (Q) for any return period (T) is given by:

$$Q_T = QMED * GF_T$$

where QMED = median annual flood (50% annual exceedance probability), and
GF_T = the frequency curve growth factor.

D3.2 QMED Calculation

Following FEH procedures, QMED can be estimated from available flood records or from catchment descriptors such as given in Table C-1. The QMED equation should be calibrated (adjusted)by reference to gauged data, ideally from the site in question, but alternatively from a suitable donor catchment. There is no data available on the Lime Brook itself.

Research into the calibration of the QMED equation has indicated that the use of distant donor catchments is not an accurate technique. WinFAP FEH4 identifies donor sites based on the geographical distance between the centroids of the target catchment and donor catchment. Ten catchments were identified. It is recommended that donor sites should be no more than five times the size of the target site only one of the suggested sites fitted this description.

In the case of the Lime Brook, a suitable gauged donor site is available at Rhos-y-Pentref, near Llanidloes. This site is also a suitable donor for the northern tributary but is still too large to be ideal for the western tributary. The use of the donor catchment suggests a slight reduction from the estimate for QMED based upon catchment descriptors. (see Table D-2). In view of this, the more conservative catchment descriptor value of 3.46 m^3/s has been used for the analysis.

Table D-2 Donor Site for the Lime Brook at Lingen

Station	54025 (Dulas @ Rhos-y-pentref)	combined catchment	west	north
Distance*	39.04	-	-	-
Urb ext	0.001	0.0004	0	0.0006
Qmed obs/adjs	23.241	3.261	1.197	2.405
Qmed CDs	30.881	3.462	1.271	2.553
Centroid	296850 279005	334432 268451	333913 267583	334767 269034
Area	53.33	19.545	7.7725	11.7225
SAAR	1268	882	890	877
BFIHOST	0.439	0.645	0.702	0.608
FARL	1	1	1	1
Years	47	-	-	-
Qmed & Pooling	Yes	-	-	-
weight	0.211	-	-	-
f.s.e.		1.42	-	-

*From Target Catchment

D3.3 Growth Factors and Design Flood Flows

Return period flows can be estimated from site records, if available, but should only be used for return periods of up to half the length of record. For longer return periods, the so-called pooled analysis is recommended which derives a typical dimensionless growth curve from weighted parameters of catchments with similar hydrological characteristics to the target catchment. This growth curve can be applied to the catchment Qmed to calculate local design flood peaks. Data from the Hi-Flows UK database (v6) has been used in deriving a Pooling Group for the combined Lime Brook catchment at Lingen, which has been based upon a 100-year return period (i.e. using 500 station years of data). Larger pooling groups are less representative of the target catchment. The membership of the Pooling Group is shown in Table D-3.

The WINFAP-FEH software (v4) indicated that the initial pooling group was classed as "possibly heterogeneous" for L-CV/L-Skewness, for which a review is optional and "strongly heterogeneous" for L-CV. During the review process, it was noted that catchment 49005 had a short period of record, less than ten years (not recommended for pooling analysis) and plotted away from the group for L-moments and flood seasonality. In addition, catchment 28058 also plotted as an outlier for L-moments, had a low FARL value of 0.977, a high urban extent of 0.021 and a flat growth curve. Both stations were removed from the group and replaced to make up the required station years of data.

The final pooling group is classified as "acceptably homogeneous" for L-CV/L-Skewness, for which a review is not required. The Goodness of fit is classed as "an acceptable fit (absolute Z-value < 1.645)" with a Z-value for the Generalised Logistic of –0.5919 for the adjusted pooling group. Accordingly growth curves were derived based on the General Logistic distribution and growth factors identified. The results of the growth curve analysis and derived flood flows are presented in Figures D-4, D-5 and D-6, and Table D-5.

Table D-3 Final pooling group for the Lime Brook at Lingen, Station Parameters

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
27010 (Hodge Beck @ Bransdale Weir)	0.255	41	9.42	0.224	0.293	0.305
44008 (South Winterbourne @ Winterbourne Steepleton)	0.274	37	0.448	0.416	0.326	0.935
25019 (Leven @ Easby)	0.38	38	5.333	0.338	0.391	0.826
22003 (Usway Burn @ Shillmoor)	0.412	13	16.17	0.282	0.311	1.555
26802 (Gypsey Race @ Kirby Grindalythe)	0.428	17	0.116	0.274	0.24	0.01
203046 (Rathmore Burn @ Rathmore Bridge)	0.689	34	10.788	0.146	0.136	0.905
26803 (Water Forlornes @ Driffield)	0.807	17	0.437	0.3	0.112	1.445
44013 (Piddle @ Little Puddle)	0.816	23	1.103	0.463	0.254	2.128
27032 (Hebden Beck @ Hebden)	0.945	50	3.923	0.207	0.253	0.325
36010 (Bumpstead Brook @ Broad Green)	0.951	49	7.585	0.365	0.173	1.854
28041 (Hamps @ Waterhouses)	0.986	31	26.664	0.22	0.295	1.096
24006 (Rookhope Burn @ Eastgate)	0.994	20	24.62	0.152	0.117	1.148
73015 (Keer @ High Keer Weir)	1.007	25	12.239	0.174	0.191	0.467
48004 (Warleggan @ Trengoffe)	1.031	47	9.983	0.261	0.263	0.11
41020 (Bevern Stream @ Clappers Bridge)	1.037	47	13.9	0.205	0.17	0.727
48009 (st Neot @ Craigshill Wood)	1.06	12	8.469	0.245	0.373	1.557
39033 (Winterbourne Stream @ Bagnor)	1.247	54	0.404	0.344	0.386	1.607
Total	555					
Weighted means	1056		0.276	0.256		

Table D-4 Final pooling group for the Lime Brook at Lingen, Catchment Descriptors

Station	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
27010 (Hodge Beck @ Bransdale Weir)	0.255	18.82	987	0.009	1	0.001
44008 (South Winterbourne @ Winterbourne Steepleton)	0.274	20.18	1012	0.015	1	0.004
25019 (Leven @ Easby)	0.38	15.09	830	0.019	1	0.004
22003 (Usway Burn @ Shillmoor)	0.412	21.88	1056	0.006	1	0
26802 (Gypsey Race @ Kirby Grindalythe)	0.428	15.85	757	0.03	1	0
203046 (Rathmore Burn @ Rathmore Bridge)	0.689	22.5	1043	0.072	1	0
26803 (Water Forlornes @ Driffield)	0.807	32.42	721	0.016	1	0.007
44013 (Piddle @ Little Puddle)	0.816	34.09	1002	0.015	1	0.004
27032 (Hebden Beck @ Hebden)	0.945	22.25	1433	0.021	0.997	0
36010 (Bumpstead Brook @ Broad Green)	0.951	27.58	588	0.045	0.999	0.007
28041 (Hamps @ Waterhouses)	0.986	37.04	1085	0.033	1	0.004
24006 (Rookhope Burn @ Eastgate)	0.994	36.6	1126	0.018	0.994	0
73015 (Keer @ High Keer Weir)	1.007	30.04	1158	0.074	0.976	0.003
48004 (Warleggan @ Trengoffe)	1.031	25.26	1445	0.035	0.978	0.003
41020 (Bevern Stream @ Clappers Bridge)	1.037	35.48	886	0.076	0.993	0.013
48009 (st Neot @ Craigshill Wood)	1.06	22.97	1511	0.023	0.982	0.002
39033 (Winterbourne Stream @ Bagnor)	1.247	45.31	717	0.033	1	0.001

Figure D-4 Growth curves for the final pooling group

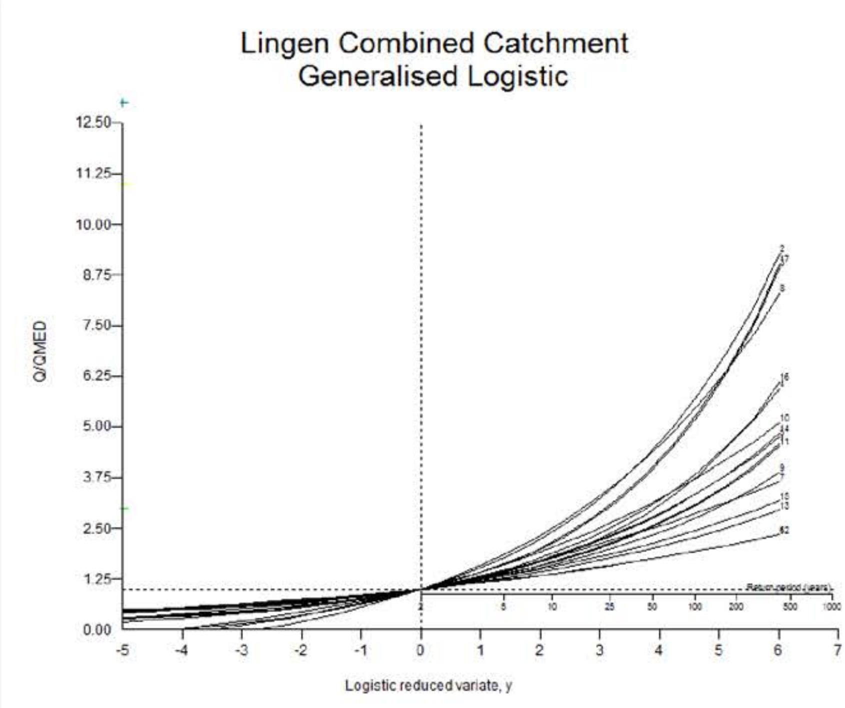


Figure D-5 Derived growth curve for the Lime Brook at Lingen

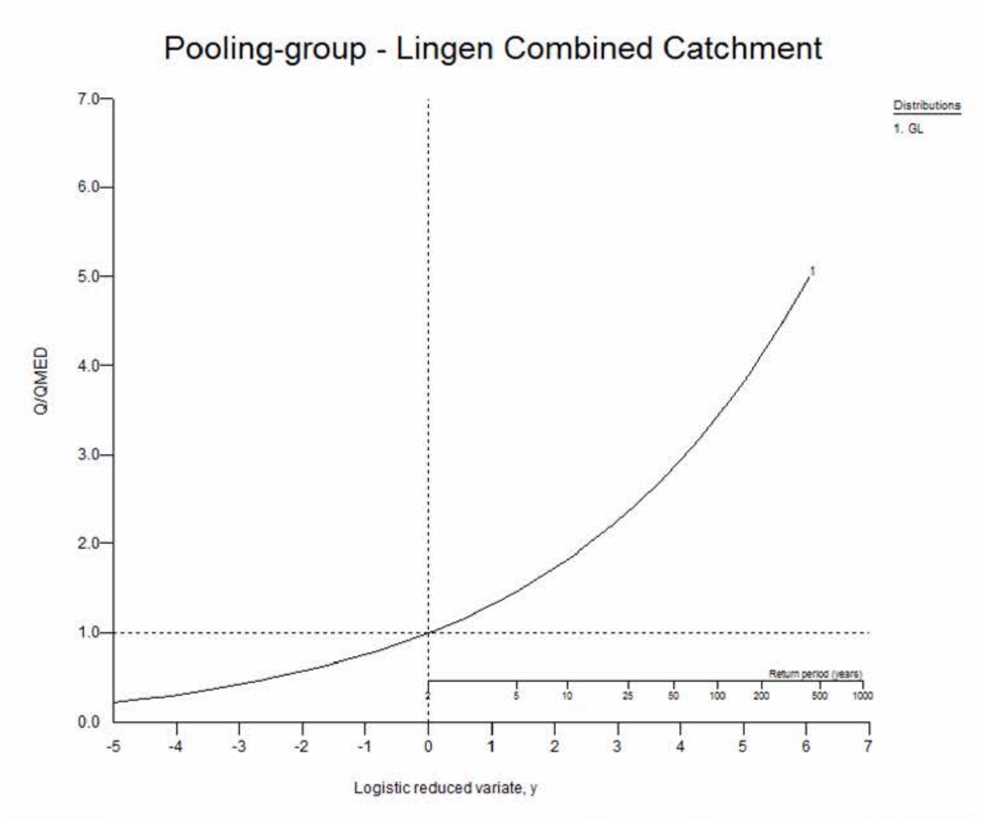
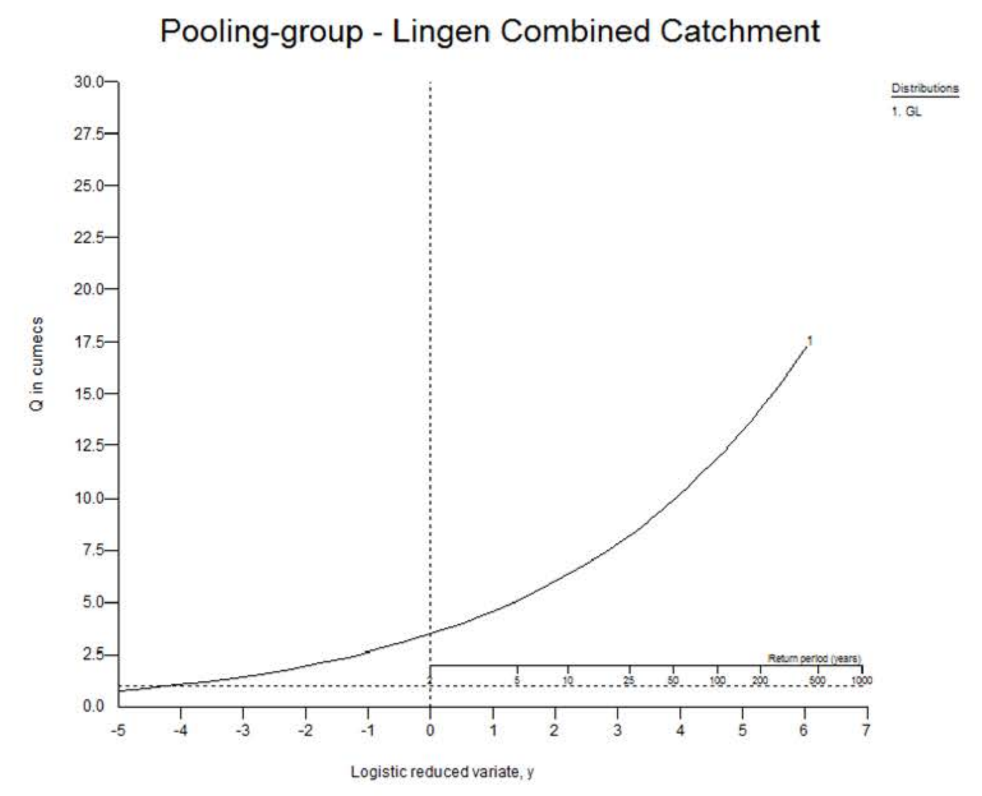


Figure D-6 Flood frequency curve for the Lime Brook at Lingen



D.5 ReFH2 Rainfall-Runoff Method

The Revitalised Rainfall Runoff Method (ReFH2) has recently been issued by WHS (2015). The software uses the catchment characteristics obtained from the FEH website to derive hydrographs, peak flows and runoff volumes in the target catchment, for a range of return periods. The flood estimates are somewhat higher than for the WINFAP-FEH method with an implied growth factor of 3.72.

Table 0-5 Flood frequency table for the Proposed Development

Return Period (yrs)	WinFEH GF	WINFAP-FEH4 (m^3/s)	ReFH (m^3/s)
2	1.000	3.462	4.85
100	3.435	11.894	18.06
1000	6.275	21.728	31.69

D.6 Recommended Results

The recommended flood estimates are those based on the WINFAP-FEH pooled analysis. These are based on measured data in similar catchments whereas the ReFH2 estimate is based solely on catchment characteristics. This gives a 1% flow for the target catchment of 11.89 m^3/s and a 0.1% flow of 21.73 m^3/s.

WinFEH produces design flow peaks while ReFH2 also produces hydrographs. These hydrographs can be scaled using the WinFEH flood peaks to produce corresponding hydrographs.

D.7 Derivation of Catchment Split

The hydraulic model of the Lime Brook at the site requires separate flow boundary conditions in the two tributaries at the upstream end of the site with a combined flow at the downstream end of the site. In order to produce internally consistent flows the combined flow has been split and allocated to each of the smaller catchments in proportion to their Qmed values. The western catchment has been apportioned 1/3 of the flow while the northern catchment carries 2/3 of the flow.

Table D-6 Qmed values and catchment flow split

Calculation Method	Combined catchment	Western Catchment	Northern Catchment	Western %	Northern %
Catchment Descriptors	3.462	1.271	2.553	33.23	66.76
Donor adjustment	3.261	1.197	2.405	33.23	66.77
ReFH2	4.854	1.727	3.4164	33.57	66.43

D.8 References

Author	Date	Title/Description
Centre for Ecology & Hydrology	2018	Flood Estimation Handbook (web-service). Available at: https://fehweb.ceh.ac.uk
Centre for Ecology & Hydrology	Feb 2018	WINFAP-FEH v6 data files from: https://nrfa.ceh.ac.uk/winfap-feh-files
Wallingford HydroSolutions	2016	WinFAP FEHv4
Wallingford HydroSolutions	2015	ReFH2

APPENDIX E HYDRAULIC MODELLING

E.1 STATEMENT OF OBJECTIVES

This Appendix summarises the approach and findings of hydraulic modelling undertaken in support of a Flood Risk Assessment (FRA) for the proposed development at Lingen Nursery, Lingen, undertaken by Hydro-Logic Services during August and September 2018.

The hydraulic modelling builds on an earlier model developed for a previous FRA for the site undertaken by Hydro-Logic Services in 2009 (*K0079_Lingen*).

A 1-dimensional (1D) hydro-dynamic hydraulic model has been developed in order to refine a range of flood extents deriving from Lime Brook and its northern tributary (referred to as Lime Brook west and Lime Brook north respectively), which flow adjacent to the site as shown in Figure E-1 .

The specific objectives of the modelling are to estimate flood levels and extents resulting from:

- the 100 year (1.0% AEP) storm plus 35% allowance for climate change;
- the 100 year (1.0% AEP) storm plus 70% allowance for climate change;
- the 1,000 year (0.1% AEP) storm.

The resultant flood levels and extents support the FRA by informing constraints on development such as the developable extent of the site, minimum finished floor levels, location of potential SuDS features, and access/egress routes.

The format of this Appendix is based on Environment Agency Best Practice Guidance *Using computer river modelling as part of a flood risk assessment* (undated).

E.2 MODEL BUILD AND CALIBRATION

E.2.1 Method Statement and Justification

The Environment Agency Flood Map for Planning shows the development site to be adjacent to Flood Zones 2 and 3, at risk of flooding from Lime Brook west (see Figure 4 of the main report). However, the published flood map does not include an allowance for increased river flows due to climate change. The published flood map was developed using the *J-Flow* software, which does not provide level data, and therefore does not enable the development of a stage-discharge rating for the watercourses at the site. Without a stage-discharge rating, it is not possible to interpolate/extrapolate flood levels resulting from increased flows due to climate change. It was concluded that the only feasible approach to determining the key flood extents was to develop a new hydraulic model for the site.

An earlier 1D hydraulic model was developed for a previous FRA for the site (Hydro-Logic Services, *K0079_Lingen,* 2009). Channel and floodplain cross-sections, incorporating key structures in and adjacent to the channels, were surveyed at intervals along Lime Brook west and north. The surveyed cross-sections are plotted in Figure E-1 .

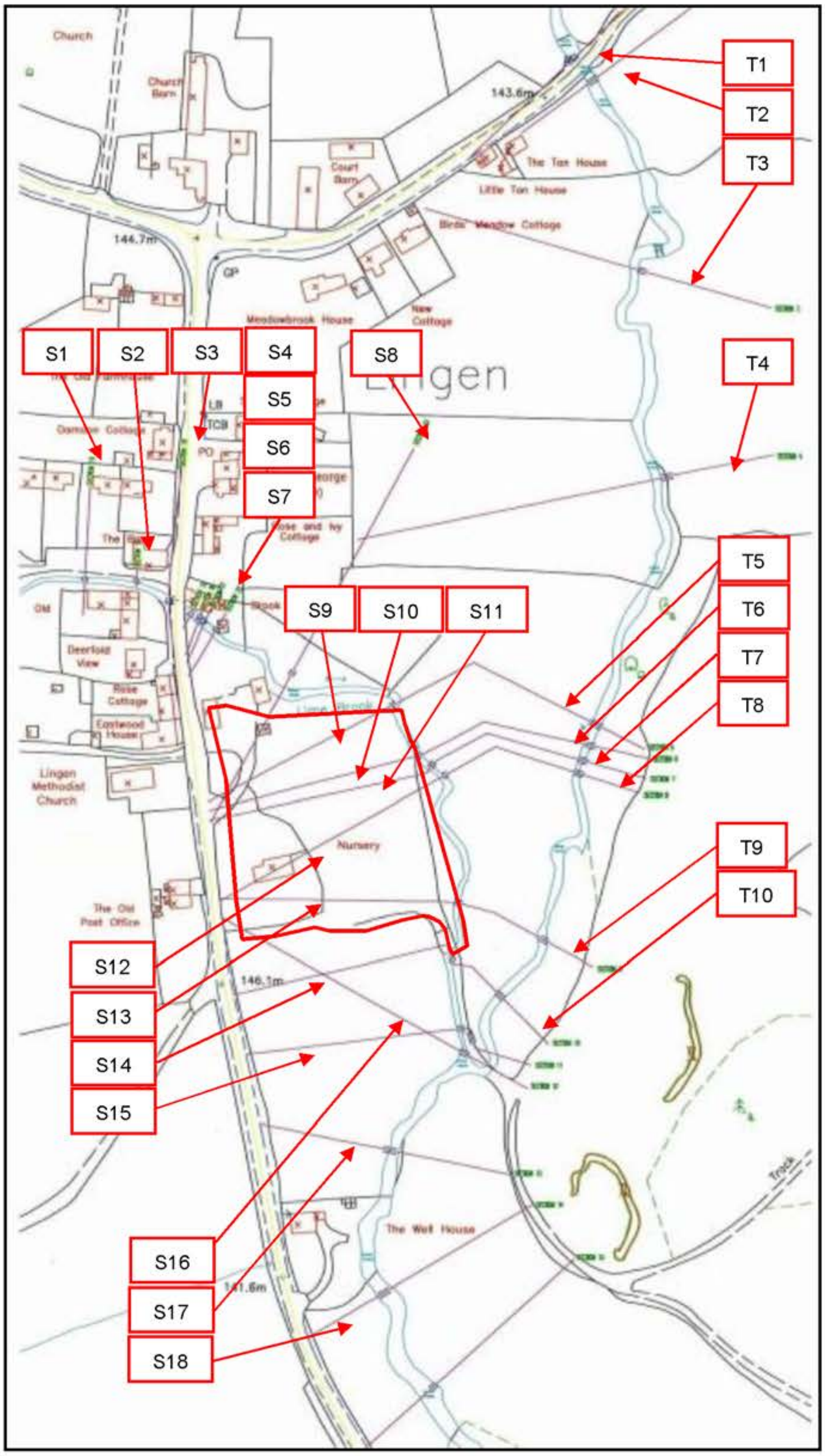
The survey results were imported into the hydraulic modelling software to create the earlier hydraulic model used as the basis for the current model. The following key changes where implemented for the new model.

- The catchment inflow hydrographs were redeveloped using the latest hydrological software (WinFAP FEHv4, ReFH2, FEH website) and NRFA HiFlows dataset v6.
- The earlier 1D model was imported into the latest *Flood Modeller* software.
- The model was revised to reflect recent changes to the structure of the *Irish Bridge*.

E.2.2 Data Sources

The original survey was undertaken by Invar Mapping of Shrewsbury, who provided the cross-sectional data in .csv and .dwg format, and a map of the modelled reaches (Figure E-1). Sections of Lime Brook west are designated by the upstream *River* units S1 to S19, and sections of Lime Brook north are designated by the upstream *River* units T0 to T12.

Figure E-1 The modelled stretches of the watercourses and survey locations



Note that sections are divided in two where they cross the watershed between the two watercourses. For example, cross-section S12 on Lime Brook west becomes cross-section T8 where it crosses Lime Brook north. This ensures that both streams are treated separately by the model, without double counting the area of the floodplain. The right-bank of cross-section S16 was deactivated (truncated) where it crosses the upstream cross-section S15 to eliminate double-counting of the floodplain area.

Photographs of the site were referenced to inform the selection of values for *Manning's n* (roughness) along the modelled channels and adjacent floodplain.

Flow data was derived using methods specified in the Flood Estimation Handbook and subsequent guidance, as described in Appendix D.

E.2.3 Design Flows

As summarised in Appendix D, the 100 year design flow was calculated for the combined catchment, and flow allocated to each sub-catchment in proportion to their respective QMed values, as determined by ReFH. On this basis, one third of the combined flow was supplied by Lime Brook west and two thirds by Lime Brook north. The analysis presented in Appendix D allowed the peak flows shown in Table E-1 to be determined.

Table E-1 Peak flows for combined and sub-catchments

Rainfall event return period	Combined catchment	Lime Brook west	Lime Brook east
	m^3s^{-1}		
100 year	11.894	3.965	7.929
100 year + 35% CC	16.057	5.352	10.705
100 year + 70% CC	20.220	6.740	13.480
1000 year	21.728	7.243	14.485

E.2.4 Model Build and Optimisation

The latest *Flood Modeller* 1D hydraulic modelling software (version 4.4.6743.18767) was used to develop and run the hydraulic model for the site. A schematic of the model network is shown in Table E-2. Note that the model schematic image has been generated using the *ISIS* free software, due to the poor resolution of schematic layouts produced by the *Flood Modeller* software used to develop and run the model.

REFHBDY units located at the upstream boundary of each reach have been assigned the respective catchment and flow parameters and the peaks adjusted to reflect the results of the analysis described Appendix D. Since the 100 year design flow was calculated for the combined catchment, flow was allocated to each sub-catchment in proportion to their respective QMed values, as determined by ReFH. By this measure, one third of the combined flow was supplied by Lime Brook west and two thirds by Lime Brook north.

The road bridges on the west and north channels were both modelled by an orifice unit and spill unit in parallel, the profiles of which were informed by survey. The view downstream of the road bridge near the upstream limit of Lime Brook west (S3) is shown in Figure E-3.

Two footbridges on Lime Brook west (Figure E-4 and Figure E-5) are located near the upstream and downstream limits of the site. The likely effect of these footbridges on flow regimes is considered to be negligible, and therefore they are not modelled explicitly.

Figure E-2 Model Schematic, showing the arrangement of model elements

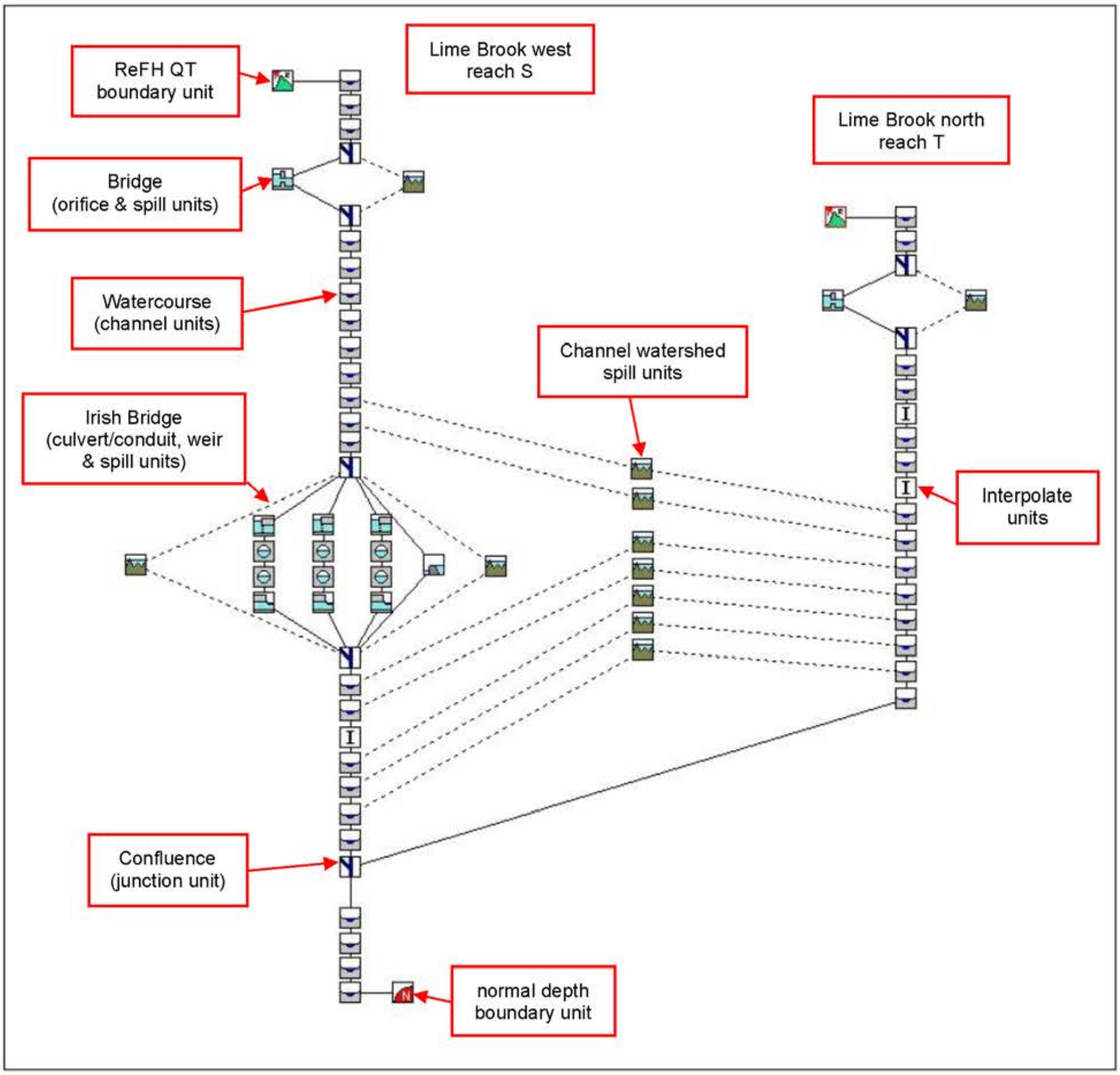


Figure E-3 The road bridge near the upstream end of the study reach on Lime Brook west

Figure E-4 The footbridge immediately downstream of the road

Figure E-5 The footbridge near the confluence of the two streams

The *Irish Bridge* crosses Lime Brook west at cross-section S11. This crossing has been modified since the original survey, and so the hydraulic model has been modified to reflect the current structure. The details of the current structure are reproduced in Table E-2 below. A photograph of the upstream face of the structure is shown in Figure E-6. It may be seen that the structure comprises a *Weir* unit (to model the surface of the crossing), four circular *Conduit* units each with inlet and outlet *Culvert* units, and two *Spill* units schematised based on surveyed cross-sections of the left and right bank (floodplain).

During optimisation of the model, it was not possible to achieve a steady state simulation of sufficient stability to produce steady state initial conditions. Removing the 150mm diameter conduit enabled the generation of the required steady state initial conditions. It should be noted that this precautionary solution would underestimate the conveyance through the culvert structure by up to 10%.

Figure E-6 View downstream to Irish bridge

Table E-2 Irish Bridge (S11) modelled structure

Structure	Invert (mAOD)	Dimension (m)	Manning's n	Comment
Culvert 1	140.41	0.375 Ø	0.025	-
Culvert 2	140.80	0.150 Ø	0.025	Not modelled
Culvert 3	140.41	0.375 Ø	0.025	-
Culvert 4	140.41	0.600 Ø	0.025	-
Weir	141.05	4.130 W	-	-

Flow between the two watersheds is enabled by spill units downstream of section S8/T5.

Instability in the early model runs was addressed by inserting additional, *Interpolate* units at critical locations.

The roughness coefficient (Manning's n) for the channel and flood plain was taken to be 0.035 and 0.050 respectively, as shown in Table E-3.

Table E-3 Manning's n roughness coefficients

Location	Description	Manning's n
Channel	Meandering, rifts and deep pools	0.035
Floodplain (left & right bank)	Some woodland and clear areas	0.050

Table E-4 Comments on survey methods

Requirement/feature	Comment
Upstream boundary	Three sections were measured along the 35 m upstream of the road bridge on Lime Brook west, in order to establish channel conditions within the model upstream of that bridge. The upstream boundary on Lime Brook north was at the road bridge, some 300 m upstream of the site.
Downstream boundary	The downstream boundary is 120 m below the confluence of the two streams, more than 200 m downstream of the section of interest.
Bridges and culverts	There are four bridges within the modelled reach. The two upstream bridges are of stone construction and expected to pose significant limitations to flow. The stone bridge on Lime Brook west is shown in Figure E-3, the bridge at the upstream end of Lime Brook north is of similar construction. These upstream bridges are expected to have little effect on the magnitude of flow reaching the site, although they may delay and attenuate the flood peak. The two bridges adjacent to the site pose a relatively small impediment to flow (Figure E-4 and Figure E-5) and are not included in the model. Additionally, the *Irish Bridge* (Figure E-6) lies adjacent to Lingen Nurseries. It is explicitly modelled but again, it is expected to have a limited influence of flood levels, since it is set low in the channel.
Floodplain on meander section	The flood plain sections were surveyed at right angles to the direction of flow on the flood plain.

E.2.5 Parameters and Model Build Assumptions

Table E-5 Boundary conditions and modelling assumptions

No	Assumption	Comment
1	Upstream and Downstream Boundary conditions	ReFH units located at the upstream boundary of each reach have been assigned the respective catchment and flow parameters, and the peaks adjusted to reflect the results of the analysis described Appendix D. Since the 100 year design flow was calculated for the combined catchment, flow was allocated to each sub-catchment in proportion to their respective QMed values, as determined by ReFH. On this basis, one third of the combined flow was supplied by Lime Brook west and two thirds by Lime Brook north. Downstream boundary conditions were simulated using a NCDBDY (Normal/Critical Depth boundary) at the downstream end of the study reach.
2	Solution type	The model was initially run in *steady* mode in order to generate the initial flow conditions. The model was then run in *unsteady* mode using a fixed time step to route the relevant flood hydrograph.
3	Interpolated Cross sections	It was found necessary to create three interpolated cross-sections in order to stabilise the model.

4	Levees	Levees were not found on the ground and were not used in the model.
5	Bridges	There are four bridges in the modelled reach, as described above. Only the bridges near the upstream end of each stream section been modelled explicitly.
6	Weirs	The *Irish Bridge* at cross-section S11 was modelled as a weir, a series of circular conduits, and spill units for the out-of-channel bank (floodplain).

E.2.6 Calibration

No high flow gauging results were available for the site and so it was not possible to calibrate the model. However, it is judged that two additional simulations, specifically the application of the 70% *Upper End* allowance for increased flows due to climate change (see section D2.3), and the increased *Manning n* values modelled as part of the sensitivity test (see section D2.7) enable precautionarily high estimates of flood levels and extents for the site.

E.2.7 Sensitivity Analysis

In order to investigate the sensitivity of the model, the model was configured with *Manning n* values increased by 20%, to establish the extent to which this affected conveyance and design flood levels.

E.2.8 Blockage Analysis

In order to investigate the effect of blockage at the bridge upstream of the site, the orifice unit at the bridge (cross-section S3) was reduced by 50%, from 2.4 m^2 to 1.2 m^2, to establish the extent to which this affected conveyance and design flood levels.

E.3 MODEL RESULTS

E.3.1 Key Return Period Results

The peak water levels produced by the following return period scenarios have been simulated.

- 100 year (1.0% AEP) storm plus 35% allowance for climate change;
- 100 year (1.0% AEP) storm plus 70% allowance for climate change;
- 1000 year (0.1% AEP) storm.

Peak water levels at each channel cross-section are reproduced in Table E-6 below. Values in **bold** indicate the channel cross-sections located within the site boundary.

Table E-6 Key return period simulated peak water levels

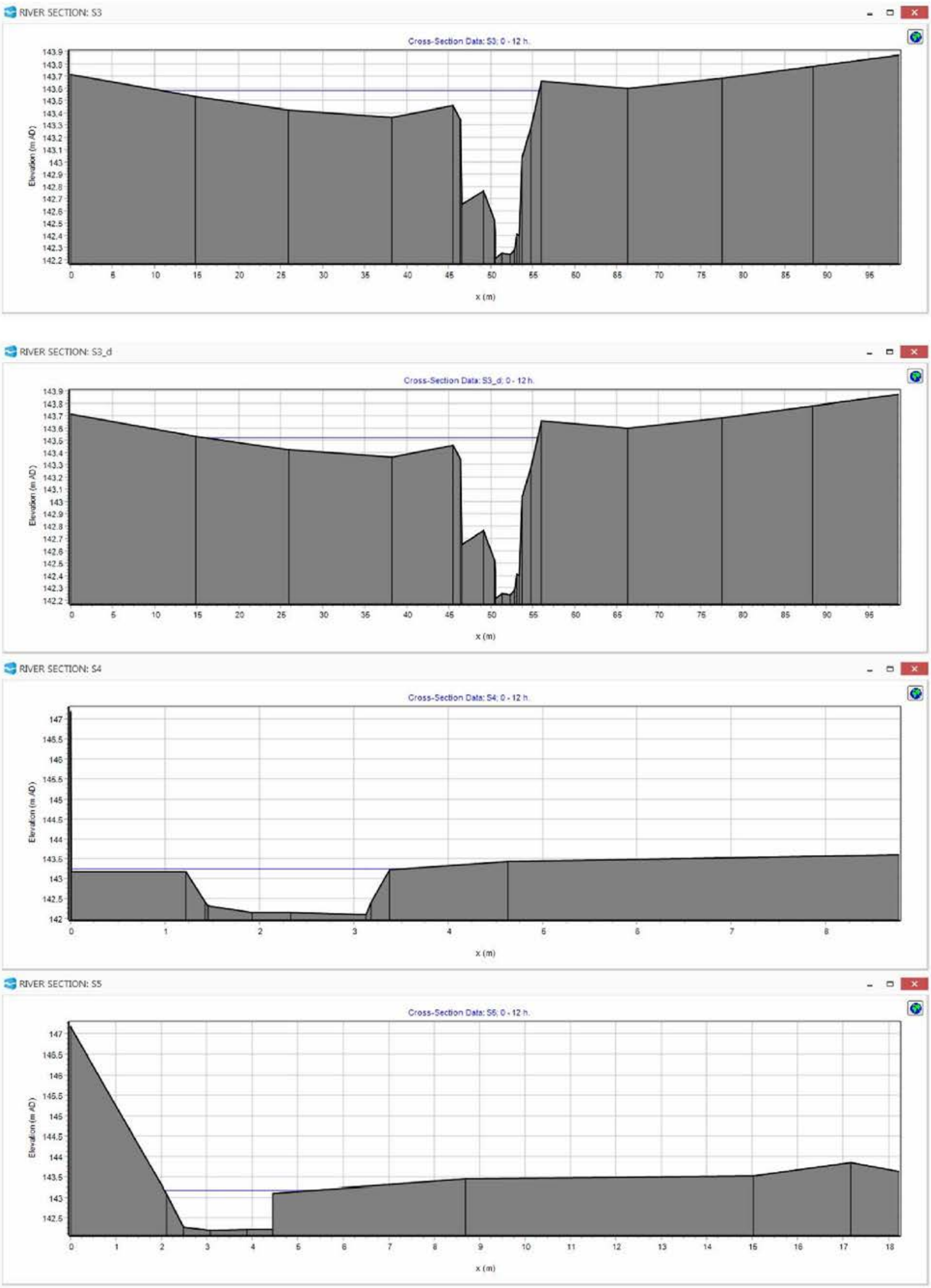
cross-section	100yr+35%CC	100yr+70%CC		1000yr	
	Elevation (mAOD)	Elevation (mAOD)	Δ (m)	Elevation (mAOD)	Δ (m)
S3	143.580	143.622	0.042	143.643	0.063
S3_d	**143.521**	**143.515**	**-0.006**	**143.518**	**-0.003**
S4	**143.258**	**143.366**	**0.108**	**143.399**	**0.141**
S5	**143.189**	**143.291**	**0.102**	**143.322**	**0.133**
S6	**143.141**	**143.244**	**0.103**	**143.276**	**0.135**
S7	**143.037**	**143.142**	**0.105**	**143.175**	**0.138**
S8	**142.379**	**142.468**	**0.089**	**142.498**	**0.119**
S9	**141.758**	**141.848**	**0.090**	**141.878**	**0.120**
S10	**141.573**	**141.670**	**0.097**	**141.700**	**0.127**
S11	**141.587**	**141.676**	**0.089**	**141.705**	**0.118**
S11_d	**141.086**	**141.173**	**0.087**	**141.202**	**0.116**
S12	**141.035**	**141.121**	**0.086**	**141.148**	**0.113**
S13	**140.262**	**140.325**	**0.063**	**140.346**	**0.084**
S14	**139.955**	**140.042**	**0.087**	**140.076**	**0.121**
S15	139.636	139.757	0.121	139.799	0.163
S16	139.719	139.833	0.114	139.866	0.147
S16_d	139.719	139.833	0.114	139.866	0.147
S17	139.329	139.442	0.113	139.477	0.148
S18	139.083	139.190	0.107	139.227	0.144

The following observations are made for the cross-sections that fall within the site boundary (i.e. S3_d to S14).

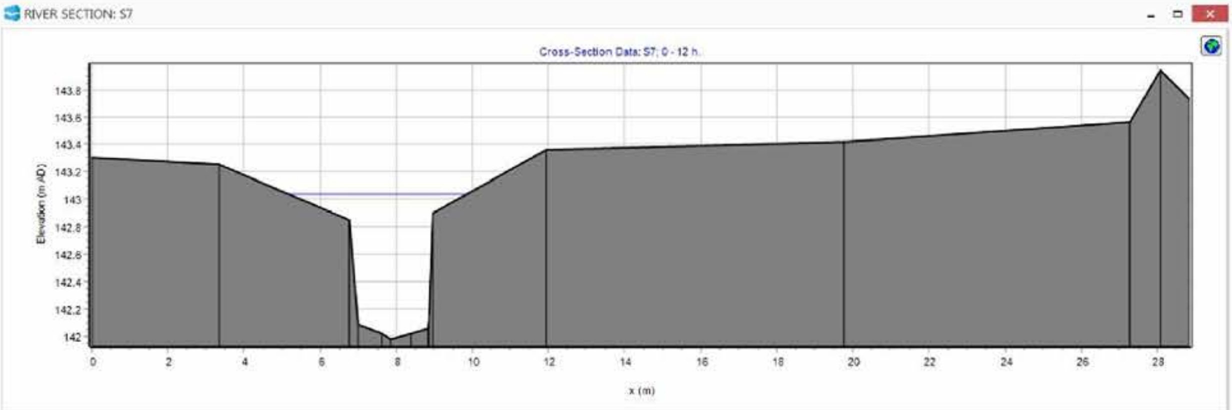
- The 100yr+70%CC peak levels are on average 85 mm higher than the 100yr+35%CC peak levels.
- The 1,000yr peak levels are on average 112 mm higher than the 100yr+35%CC peak levels.
- The 100yr+70%CC peak levels are on average 27 mm lower than, *and never greater than*, the 1,000yr levels.

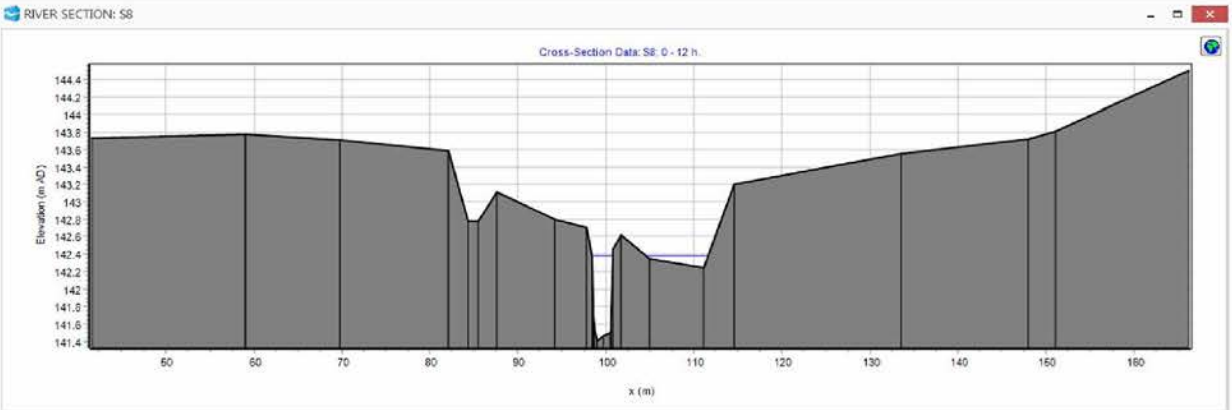
Modelled cross-sections are shown in Figure E-7 from section S3 on Lime Brook west downstream of the road bridge downstream to S18 below the confluence with Lime Brook north. The results show that the watercourse remains predominantly in-bank, excepting sections S3 to S4, where out of bank flows are limited to the left bank, on the opposite side of the watercourse to the site. Where flows are indicated to be above bank crest, for example S8, flood waters do not penetrate more than about 9 m from the channel.

Figure E-7 Lime Brook cross-sections and 100 year + 35% CC peak water levels

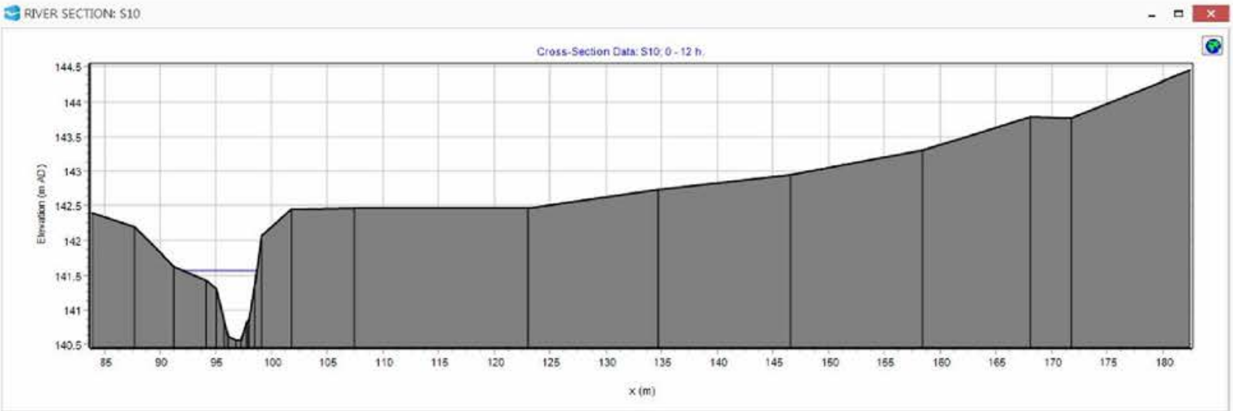


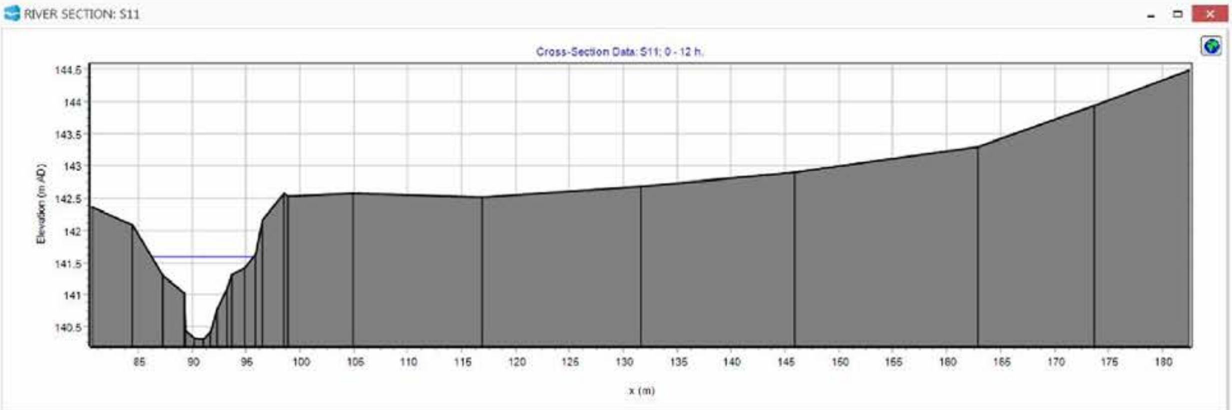

RIVER SECTION: S6
Cross-Section Data: S6; 0 - 12 h.
Elevation (m AD)
x (m)

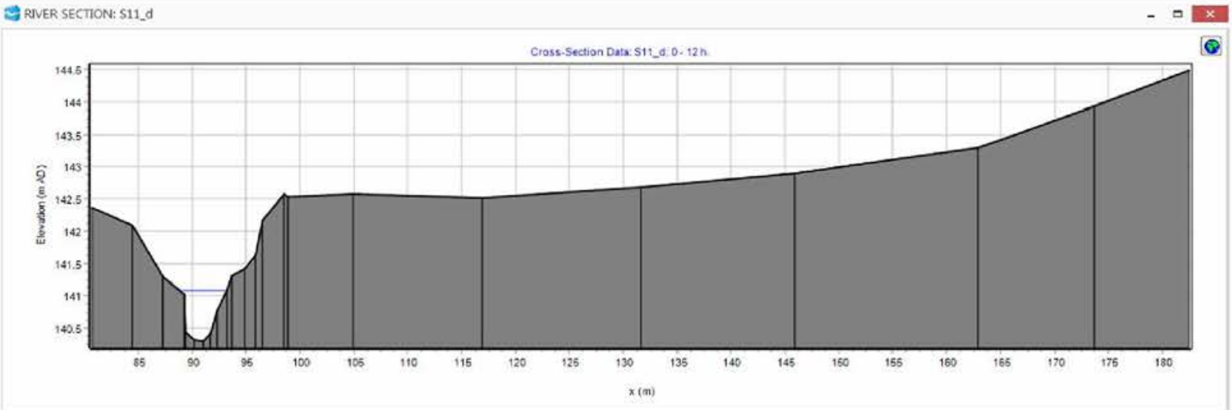

RIVER SECTION: S7
Cross-Section Data: S7; 0 - 12 h.
Elevation (m AD)
x (m)

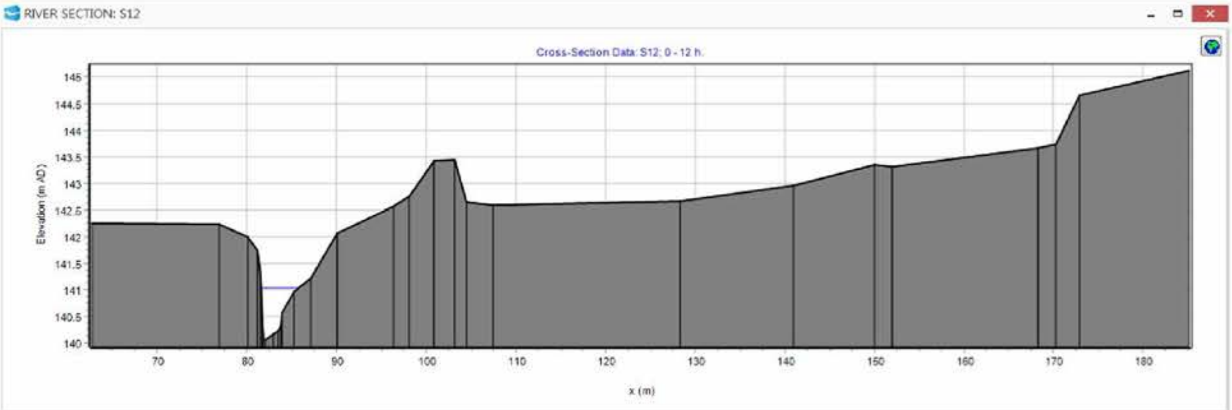

RIVER SECTION: S8
Cross-Section Data: S8; 0 - 12 h.
Elevation (m AD)
x (m)


RIVER SECTION: S9
Cross-Section Data: S9; 0 - 12 h.
Elevation (m AD)
x (m)

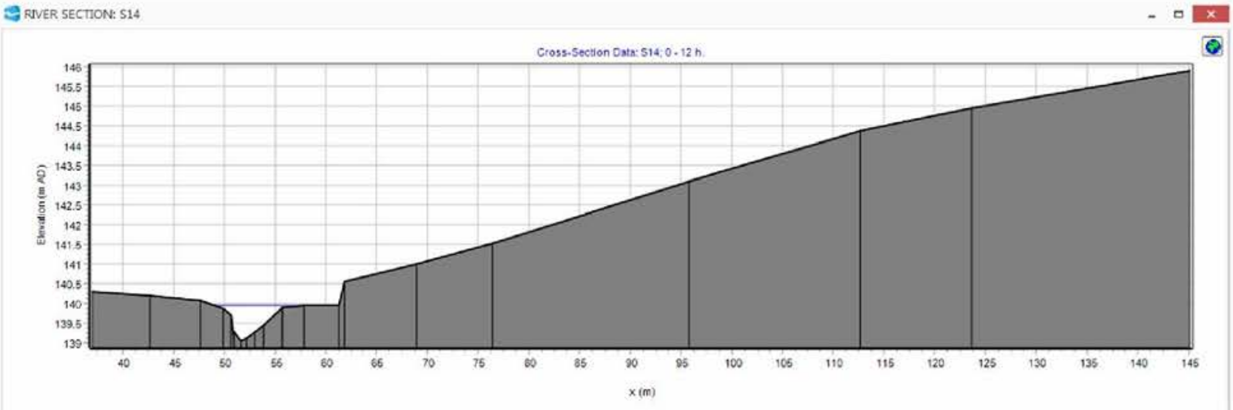

RIVER SECTION: S10
Cross-Section Data: S10; 0 - 12 h.
Elevation (m AD)
x (m)

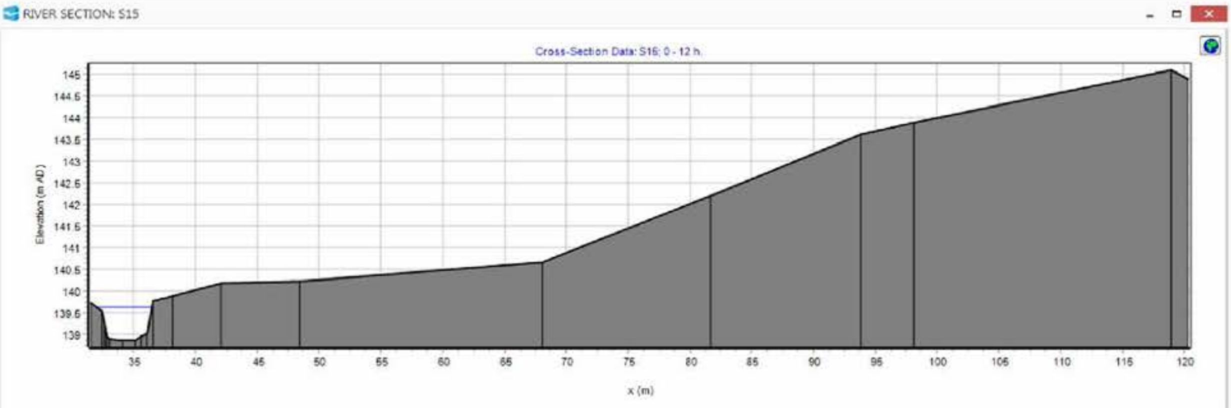

RIVER SECTION: S11
Cross-Section Data: S11; 0 - 12 h.
Elevation (m AD)
x (m)

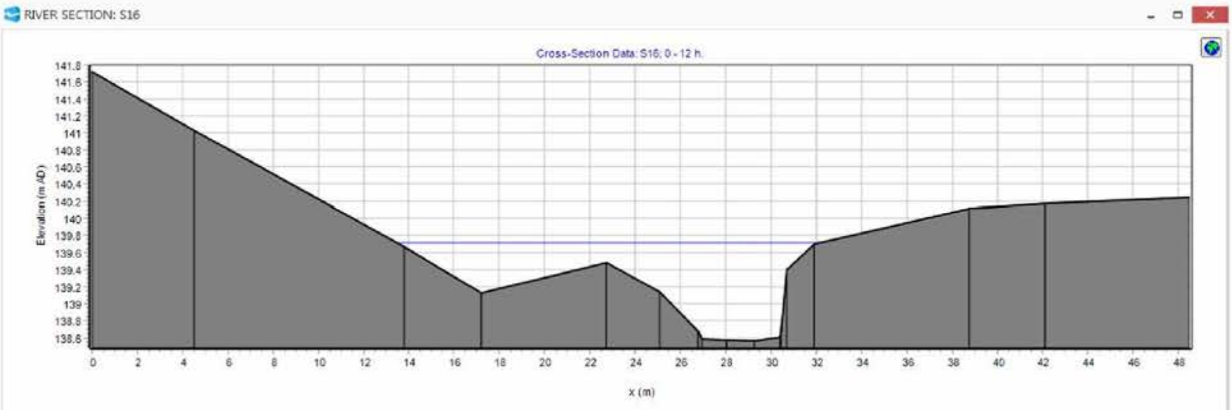

RIVER SECTION: S11_d
Cross-Section Data: S11_d; 0 - 12 h.
Elevation (m AD)
x (m)

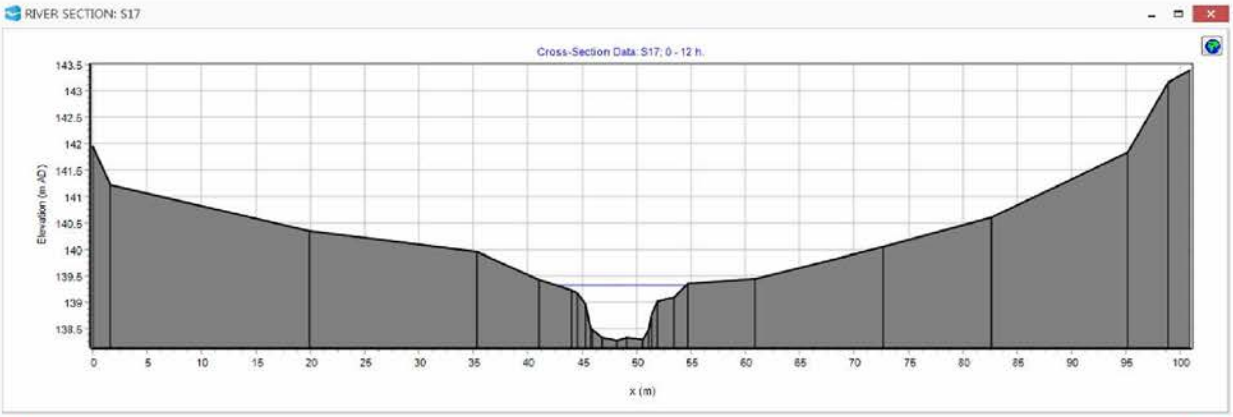

RIVER SECTION: S12
Cross-Section Data: S12; 0 - 12 h.
Elevation (m AD)
x (m)

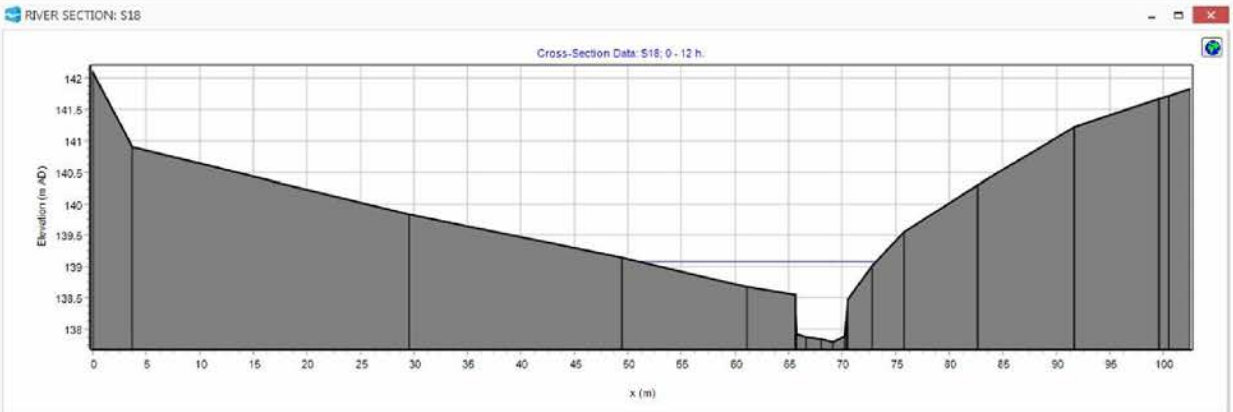

RIVER SECTION: S13
Cross-Section Data: S13; 0 - 12 h.
Elevation (m AD)
x (m)


RIVER SECTION: S14
Cross-Section Data: S14; 0 - 12 h.
Elevation (m AD)
x (m)


RIVER SECTION: S15
Elevation (m AD)
x (m)


RIVER SECTION: S16
Cross-Section Data: S16; 0 - 12 h.
Elevation (m AD)
x (m)





E.3.2 Sensitivity Analysis Results

The peak water levels produced by the sensitivity test (section E2.7) scenario - 100 year (1.0% AEP) storm plus 35% allowance for climate change, globally increase *Manning's n* by 20% - are shown in Table E-7.

Table E-7 Sensitivity Test (*Manning's n* +20%) simulated peak water levels

cross-section	100yr+35%CC	100yr+35%CC Manning's n +20%	
	Elevation (mAOD)	Elevation (mAOD)	Δ (m)
S3	143.580	143.614	0.034
S3_d	**143.521**	**143.570**	**0.049**
S4	**143.258**	**143.350**	**0.092**
S5	**143.189**	**143.267**	**0.078**
S6	**143.141**	**143.216**	**0.075**
S7	**143.037**	**143.115**	**0.078**
S8	**142.379**	**142.464**	**0.085**
S9	**141.758**	**141.824**	**0.066**
S10	**141.573**	**141.576**	**0.003**
S11	**141.587**	**141.589**	**0.002**
S11_d	**141.086**	**141.136**	**0.050**
S12	**141.035**	**141.081**	**0.046**
S13	**140.262**	**140.320**	**0.058**
S14	**139.955**	**140.065**	**0.110**
S15	139.636	139.730	0.094
S16	139.719	139.808	0.089

S16_d	139.719	139.808	0.089
S17	139.329	139.429	0.100
S18	139.083	139.197	0.114

The following observations may be made for the cross-sections that fall within the site boundary (i.e. S3_d to S14).

- The 100yr+35%CC plus 20% increase in *Manning's n* (roughness) peak levels are on average 61 mm higher than the 100yr+35%CC peak levels.
- The 100yr+35%CC plus 20% increase in *Manning's n* peak levels are predominantly in-bank, with the only significant out-of-bank flooding indicated at the downstream limit of the site (S14) which shows an increase in water level of 110 mm.
- The 100yr+35%CC plus 20% increase in *Manning's n* peak levels are on average 51 mm lower than, *and never greater than*, the 1,000yr levels.

The relatively small differences between the 100yr+35%CC results and the 100yr+35%CC plus 20% increase in *Manning's n* results suggests that the site is well modelled notwithstanding the limitations of the data. Increased *Manning n* roughness values provides precautionary modelling scenario showing Lime Brook west barely goes out of bank. Even when the watercourse does flood, waters are retained close to the channel and would not pose any flood risk to any development.

E.3.3 Blockage Results

The peak water levels produced by the blockage analysis (section E4) scenario - 100 year (1.0% AEP) storm plus 35% allowance for climate change, upstream bridge on Lime Brook west (S3) configured with 50% blockage - are shown in Table E-8.

Table E-8 Blockage test (S3 50% blockage) simulated peak water levels

cross-section	100yr+35%CC	100yr+35%CC S3 50% Blockage	
	Elevation (mAOD)	Elevation (mAOD)	Δ (m)
S3	143.580	143.612	0.032
S3_d	**143.521**	**143.521**	**0.000**
S4	**143.258**	**143.258**	**0.000**
S5	**143.189**	**143.188**	**-0.001**
S6	**143.141**	**143.141**	**0.000**
S7	**143.037**	**143.037**	**0.000**
S8	**142.379**	**142.379**	**0.000**
S9	**141.758**	**141.758**	**0.000**
S10	**141.573**	**141.573**	**0.000**
S11	**141.587**	**141.587**	**0.000**
S11_d	**141.086**	**141.086**	**0.000**
S12	**141.035**	**141.035**	**0.000**
S13	**140.262**	**140.262**	**0.000**
S14	**139.955**	**139.955**	**0.000**
S15	139.636	139.636	0.000
S16	139.719	139.719	0.000
S16_d	139.719	139.719	0.000
S17	139.329	139.329	0.000
S18	139.083	139.083	0.000

The following observations may be made for the cross-sections that fall within the site boundary (i.e. S3_d to S14).

- The 100yr+35%CC plus 50% blockage of S3 peak levels are effectively the same as the 100yr+35%CC peak levels throughout the site (S3 to S14), i.e. the watercourse remains predominantly in-bank. The exception is cross-section S3 immediately upstream of the bridge, where out of bank flows are limited to the left bank, on the opposite side of the watercourse to the site.
- The peak water level at the cross-section immediately upstream of the blocked bridge orifice (S3) is 32 mm above the 100yr+35%CC peak levels. The peak water level of the blocked bridge orifice (S3) is 143.612 mAOD, and the soffit level of the bridge is 143.400 mAOD, confirming that the opening would surcharge under the blockage scenario.

Flow over the bridge would be impeded by the 9.1 m wide parapet, but Figure E-8 shows that the road adjacent to the left bank is considerably lower than the right bank and falls to 143.39 m AOD. The view of the bridge from upstream suggests that surcharged flood waters would likely leave the watercourse upstream of the bridge, travel along the road and pass to the left of the cottage, before re-entering the channel downstream.

Figure E-8 View downstream from left bank towards upstream face of bridge at S3

E.3.4 Flood Extent Mapping

The flood extent maps for the 100 year plus 35% climate change and 1,000 year scenarios are shown in Figure E-9 and Figure E-10 respectively.

The maps were developed as follows.

- A QGIS project was developed based on 1 m cell LiDAR (elevation) data. A 0.25 m contour layer was extracted from the LiDAR data and added to the project. Further georeferenced images of the OS map at the site, the site survey plan, and the proposed development layout were added to the project.
- Referencing the results produced by the 1D *Flood Modeller* model simulations, a vector point layer was created to mark each surveyed cross-section at the intersection of peak water levels with the terrain (i.e. the limit of the flood extent), based on the distance of the intersection from the centre of the channel.
- The elevation of the intersection was verified against the underlying LiDAR data, and generally the correspondence was found to be good. On occasion the elevation of the intersection obtained from the 1D *Flood Modeller* results was found to be higher than the elevation of the underlying LiDAR, and in this case, adopting a precautionary approach, the intersection was moved further from the channel into the flood plain to the location of *Flood Modeller* data and LiDAR data correspondence.
- A vector polygon layer was created, and polygons schematised to interpolate between the flood extent points, based on the LiDAR contour layer. The resultant polygons thus represent the flood extents and are coloured dark blue for the 100yr+CC extent, and light blue for the 1,000yr extent, in accordance with Environment Agency flood zone mapping.

The flood extent maps enable the following conclusions to the drawn.

- The site is predominantly in Flood Zone 1, beyond the limits of the 1,000 year flood extent, and therefore available for development.
- A slightly greater extent of the site is in Flood Zones 1 or 2, beyond the limit of the 100 year plus 35% climate change flood extent, corresponding to the *Higher Central* allowance for climate change.
- Both the 100 year plus 35% climate change and 1,000 year flood extents are confined to the vicinity of the watercourse.

Figure E-9 100 year plus 35% climate change flood extent

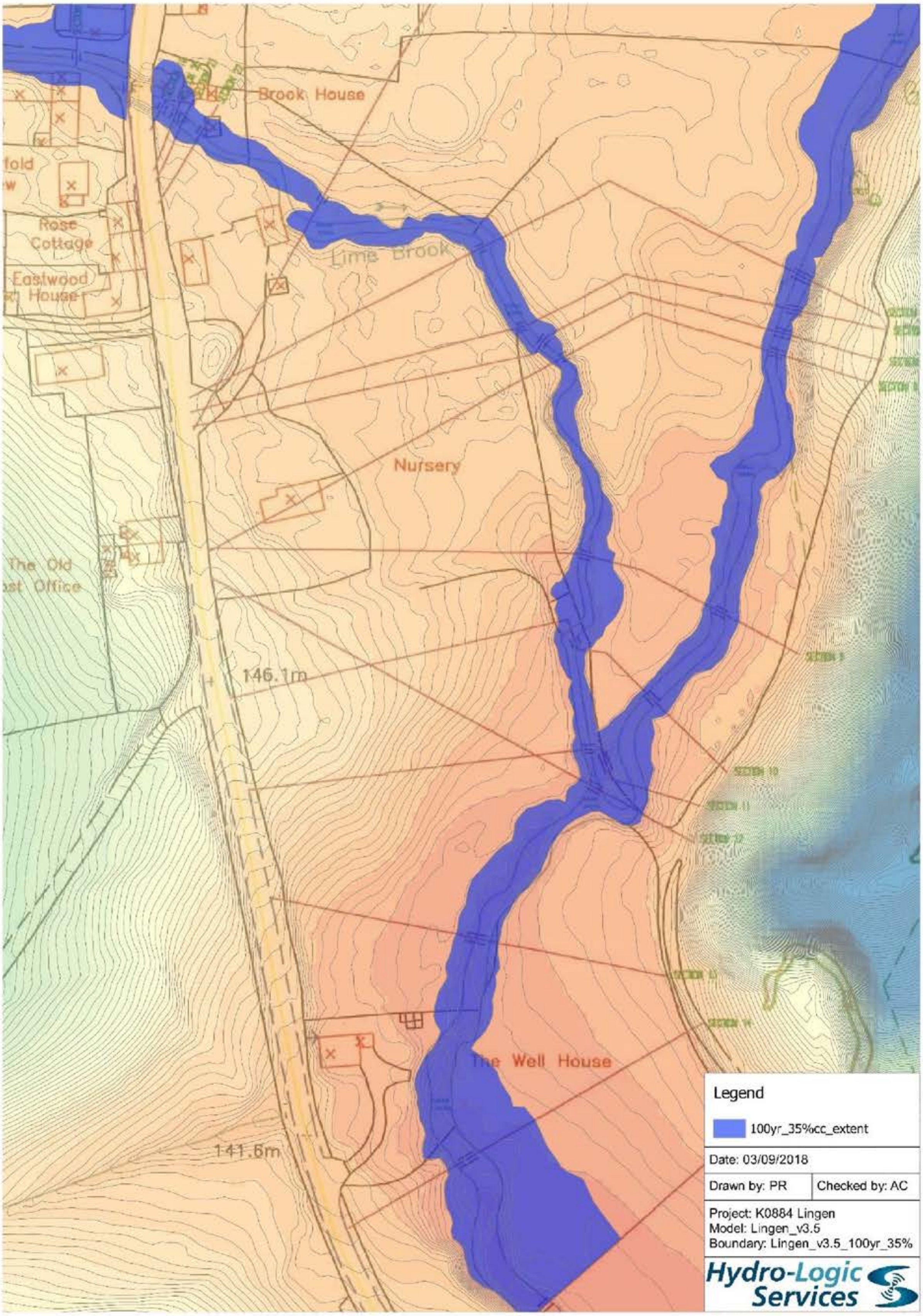
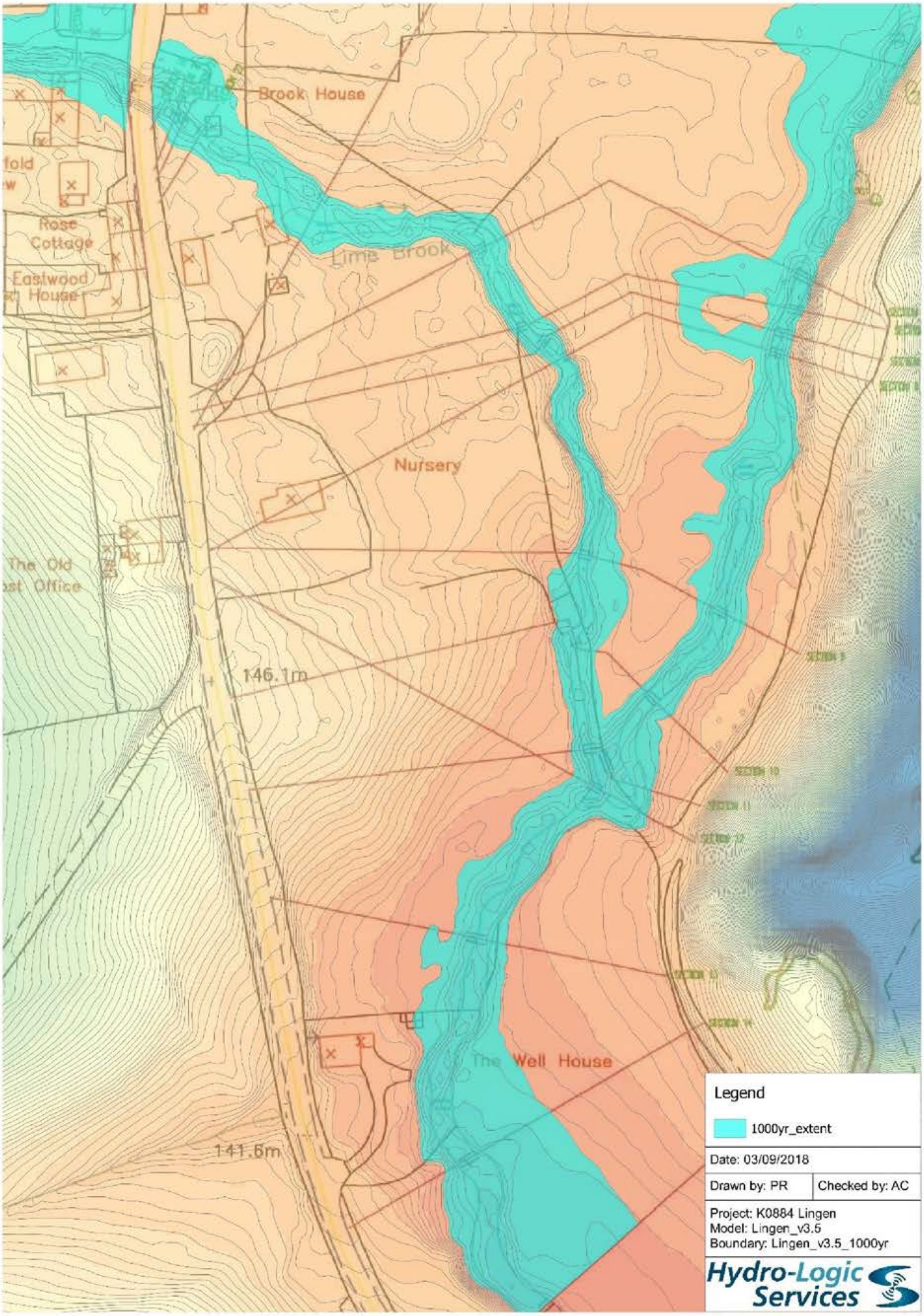


Figure E-10 1,000 year flood extent



E.4 SUMMARY AND CONCLUSIONS

1) A *Flood Modeller* 1D hydraulic model has been developed using survey data for reaches of Lime Brook west and Lime Brook north bounding Lingen Nursery, Lingen, Shropshire.
2) Hydrological analysis was undertaken to establish peak flows in Lime Brook west and Lime Brook north for the 100 year and 1,000 year storms.
3) Simulations of the following scenarios were undertaken:
   - 100 year plus 35% climate change (corresponding to the *Higher Central* allowance);
   - 100 year plus 70% climate change (corresponding to the *Upper End* allowance);
   - 1,000 year storm.
4) Resultant peak water levels are summarised and show that flooding at the site from all three scenarios is limited to the vicinity of the Lime Brook west channel.
5) The 100 year plus 35% climate change and 1,000 flood year extents have been mapped. The maps and confirm that the site is predominantly in Flood Zone 1 and is thus largely appropriate for development of the type proposed.
6) A sensitivity test was undertaken whereby the 100 year plus 35% climate change scenario was modified such that the estimated values for *Manning's n* roughness were increased by 20%. Peak water levels are shown to be greater than the baseline 100 year plus 35% climate change scenario, but less than the 1,000 year scenario.
7) A blockage test was undertaken whereby the 100 year plus 35% climate change scenario was modified such that the upstream bridge on Lime Brook west was subject to 50% blockage. Peak water levels are shown to be equal to the baseline 100 year plus 35% climate change scenario, excepting directly upstream of the blocked bridge. Under such circumstances out-of-bank flood flows are indicated to over the left bank.

APPENDIX F Foul Drainage strategy

F.1 General considerations

As per Policy SD3 of Herefordshire Core Strategy, foul and surface water should be managed separately. As a first approach, there was an attempt to find a local wastewater network but there seems to be no sewer networks in the vicinity of the site. However, Policy SD4 of Herefordshire Core Strategy states that connection to a package sewage treatment plant followed by soakage into the soil may be considered. However, due to shallow ground water depth, infiltration to ground was not considered as a viable option.

As an alternative measure it is proposed that foul water flows from the dwelling units are collected by pipework and routed through a package sewage treatment plant. Two different options have been recommended for the treatment of foul water. Option A: considers implementation of a Biokube downstream of a STP to meet an average concentration of Phosphates of 1.2mg/l followed by reed bed, while option B considers implementation of a Klargester BioDisc – that allows to attain an average concentration of phosphates of 2mg/L – followed by a reed bed to further decrease the concentration of phosphates. The need to decrease the concentration of total phosphates down to a concentration of 1mg/L is associated with the fact that the site is located within the River Wye catchment, which is deemed to be a eutrophic sensitive river.

To further improve the impacts of phosphates in the eutrophic-sensitive environment, it is proposed that the foul water flows from the old building, falling within the site blue boundary, are directed to the proposed drainage system serving the new development. Currently, the old building drains the foul water flows into a septic tank and the improvement of this system would result in the improvement of phosphate concentrations.

The future occupiers of dwellings would also be encouraged to opt for phosphorus-free detergents, therefore addressing the high phosphate concentration problem at a source level.

In both cases, the treated outflow from the STP would be discharged to the watercourse running along the eastern boundary of the site to a point where there is a permanent flow throughout the whole year.

F.2 Outline of foul drainage strategy

The foul water from the proposed dwellings will include a single sewage treatment plant either an STP followed by a BioKube (Option A) or a Klargester BioDisc followed by a reed bed (Option B). The design of this sewage treatment plant is based on the following assumptions:

- Once collected, the foul water flows are to be treated and discharged to Lime brook, downstream of the confluence between both tributaries;
- The sewage treatment plant should be sited at least 5m from any habitable parts of buildings;
- The property owners would be responsible for the management and maintenance of the foul drainage system serving their properties.
- According to Flows and Loads – 4 Code of Practice by British Water, the system should be designed for a total of 36PE – see Table F.1. This figure includes a 20% reduction applied to drainage system serving between 26P and 50P.

Table F.1 – Design criteria for the Sewage Treatment Plant

Reference	Bedrooms	People*
Plot 1	2	3
Plot 2	2	4
Plot 3	3	5
Plot 4	4	6
Plot 5	3	5
Plot 6	3	5
Plot 7	3	5
Plot 8	3	5
Old building	4	6
Total	-	44
Total after 20% reduction	**-**	**36**

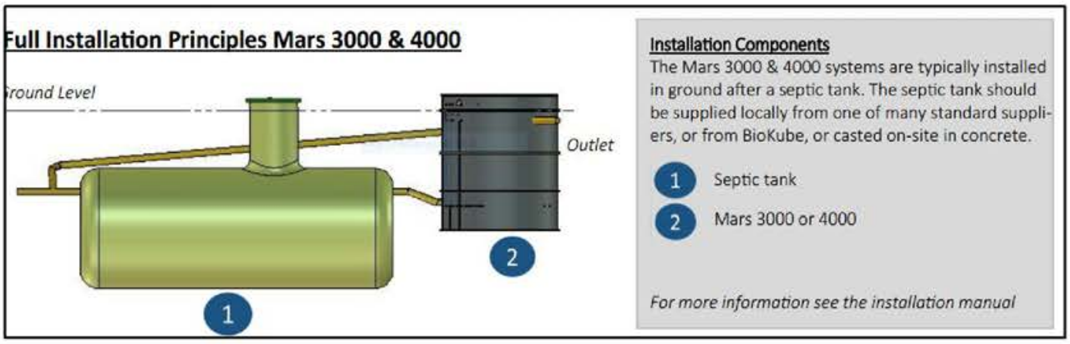
* According to *Flows and Loads – 4 Code of Practice by British Water*

F.3 Option A – Foul drainage management via STP followed by BioKube and reed bed

As mentioned before, two different approaches have been recommended for the foul drainage management system. Option A considers implementation of a Biokube Mars treatment plant. Biokube Mars systems are small STP most commonly used for single households or smaller residential groups treating up to 30m^3 wastewater a day. It will produce effluent qualities of < 10mg/l BOD, ammonia < 5mg/l and an average phosphate level 1.2 mg/l. Although in other cases Natural England has advised that a maximum concentration of phosphates of 1mg/L should be attained, 1.2 mg/L was shown to be the lowest concentration achieved by the technology in the market. The manufacturer provides a number of modules based on the number of people to serve. Module 3000-4C is capable of serving 40 people, hence proposed for the proposed development. The Mars systems must be installed downstream of a sewage treatment plant. A typical arrangement of the system is shown if Figure F2. The dimension and pipe placement for the unit is shown in Figure F3. All the dimensions are shown in mm.

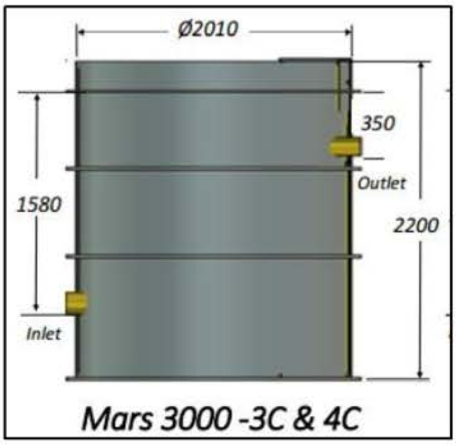
In order to further polish the treated effluent and decrease the concentration of phosphates even more, a reed bed will be located downstream of the BioKube. The reed bed system will be further discussed in the next section.

Figure F2: Typical layout Bio-kube Installation



Source: Installation manual Biokube

Figure F3: Dimension and pipe placement



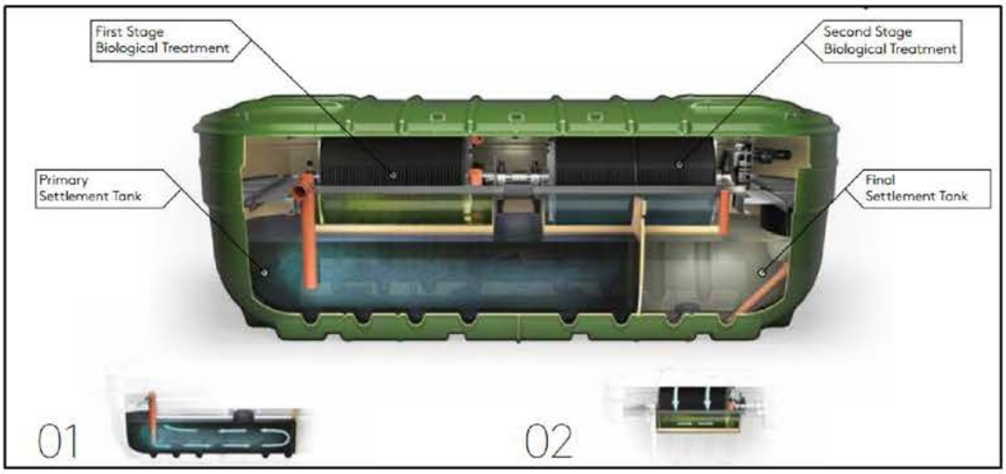
Source: Installation manual Biokube

The outflow from the Biokube will be discharged to the watercourse. The proposed foul water drainage system is shown in Figure F6.

F.4 – Option B – Foul drainage management via Klargester BioDisc followed by reed bed

Option B considers the implementation of a Klargester BioDisc followed by a set of reed bed modules. BioDisk ® is a high-performance package treatment plant which, in normal domestic situations, will produce effluent qualities of 8mg/l BOD, 13mg/l SS, 4mg/l ammonia and 2 mg/l phosphate. This system is capable of treating foul water 100% compliance with industry requirement, including national and international regulations such as BS EN12255 and EN12566-3 (up to 50 PE). The system includes a primary settlement tank and two stage biological treatment and finally another settlement tank. A diagram of the plant is shown in Figure F4.

Figure F4: Klagester BioDisk Model BM



Source: Installation manual Klargester Kingspan

The dimension of the BioDisc unit are 10.42m in length, 2.45m in width and 3.23m in height. Since the legal requirement of phosphate concentration is 1.5mg/l, a tertiary will be provided by implementation reed bed. The reed bed system will typically improve the BOD and SS effluent discharge quality by 50%. Phosphates discharged in the effluent will also be reduced but maintenance of the reed bed is mandatory beyond 6-9 months. The outflow from the STP will be discharged to a reed bed for tertiary treatment for further removal of phosphate level.

The reed bed system comprises of horizontal modules constructed from Glass Reinforced Plastic (GRP), filled with granular material, which together with the reeds provides the hydraulic flow path and environment to achieve the improved effluent quality. Each GRP module has a dimension of 2.50m in length, 80mmm in width and 800mm in depth. Each module should have at least ten plants. A 12-population unit uses 4 modules. For a population of 36 people a total of 12 modules are required. A typical image of the reed beds is shown in Figure F5.

The proposed foul drainage layout for Klargester bio-disc option is shown in Figure 6.

Figure F5: Klargester Reed Bed Tertiary Filter

Source : Kingspan Manufacturers manual

Figure F6: Proposed foul drainage layout option A

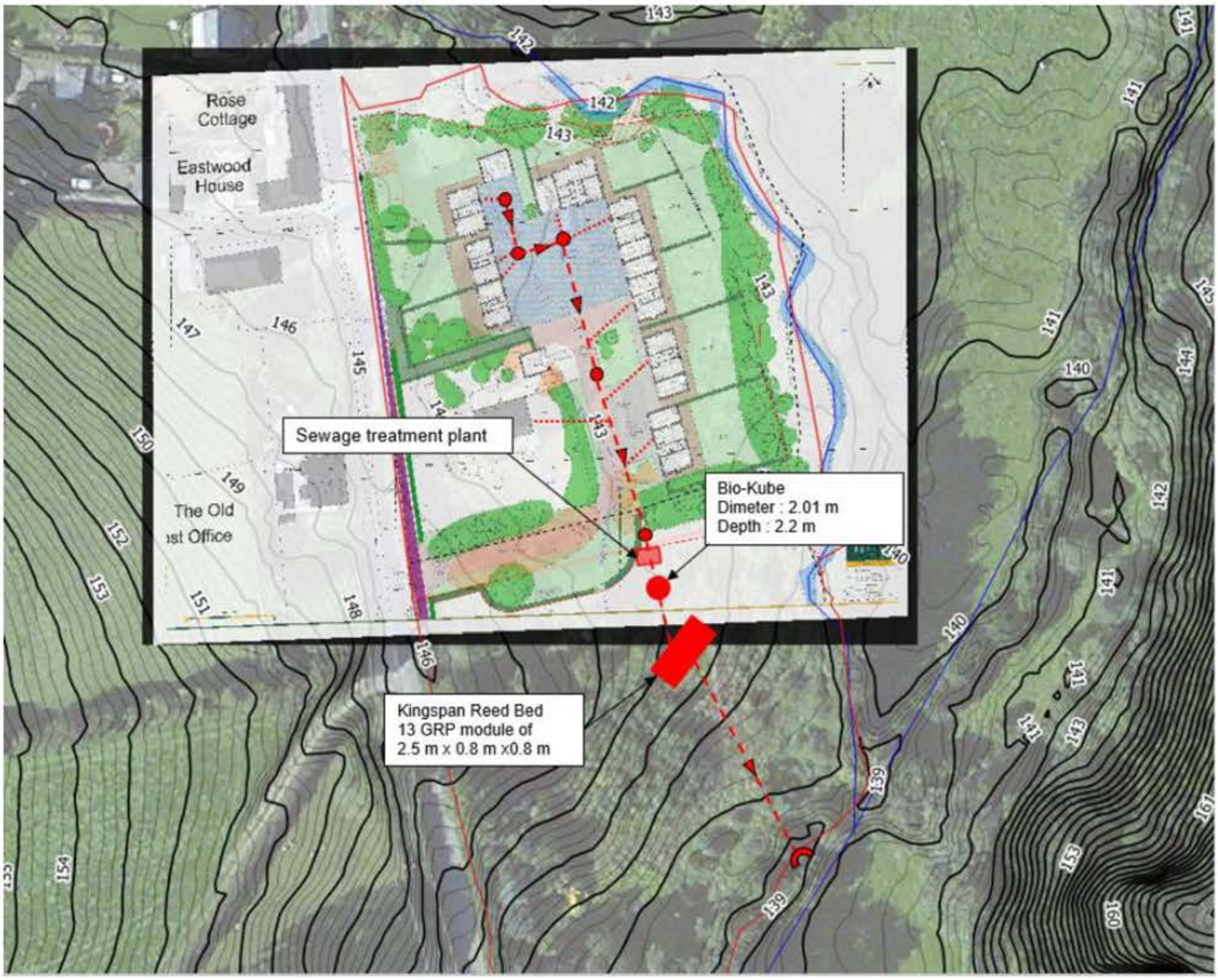
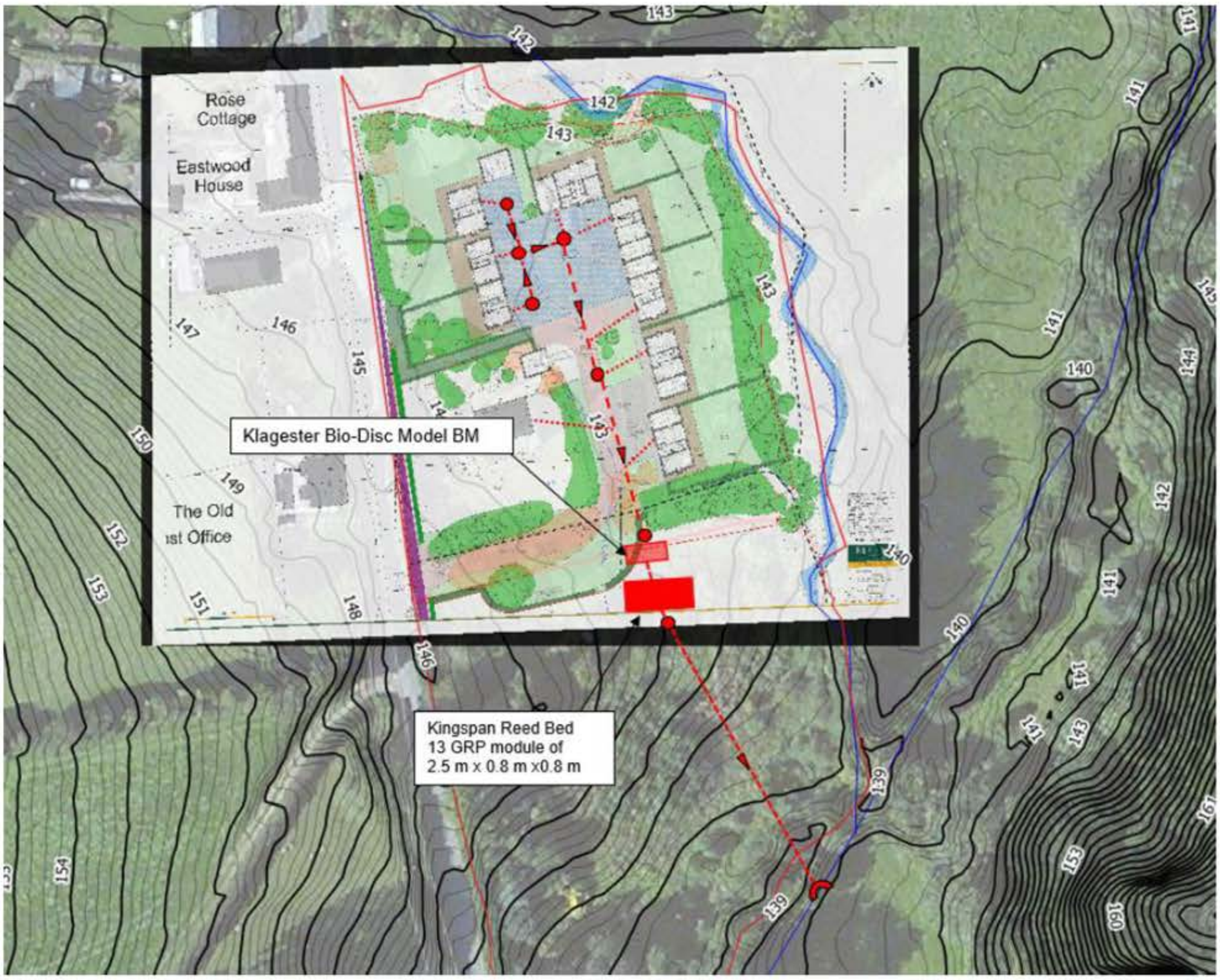


Figure F7: Proposed foul drainage layout option B



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