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Ansty Garden Community: Flood Risk Assessment and Outline Drainage Strategy

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Document Control

Title

Ansty Garden Community: Flood Risk Assessment and Outline Drainage Strategy

Client

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1. Introduction

1.1. Instruction

Aqua Terra Consultants Ltd (Aqua Terra) was instructed by Fairfax Acquisition Ltd (the Client) to provide a Flood Risk Assessment (FRA) and Outline Sustainable Drainage (SuDS) Strategy for a proposed residential led mixed use development on a parcel of land near Ansty, Mid-Sussex (the Site). Instruction to proceed was provided by email on the 19th July 2022.

1.2. Background

The Client is seeking to obtain planning permission for a residential led development at the Site. The development will contain up to 1,450 dwellings (including 30% affordable housing), up to 90 residential care units (C2 class), a primary school, new SEND school, sports facilities including all weather hockey pitches and tennis centre, allotments, retail, community and employment uses together with ancillary and associated development including new and enhanced pedestrian/cycle routes, open spaces, and landscaping. In addition to this, the planning application includes a greenfield area to the north; this area will be utilised as a Parkland Reserve (no development in this area). Further details of the proposed development are provided in Section 4.

This report constitutes a FRA and Outline Sustainable Drainage Strategy for the proposed development which is required for the planning application. This report focuses on the proposed housing development area ("the Site"); the proposed Parkland Reserve ("Beechy Bottom Parkland Reserve"), which will not be developed and is considered a water compatible land use (see Figure 3-1), is covered by a separate flood risk note (ref: P25035_R3, May 2025).

1.3. Scope

The scope of this assessment is as follows:

- Preparation of a FRA, written in line with the National Planning Policy Framework (NPPF) and supporting Planning Practice Guidance (PPG) to satisfy the Mid Sussex District Council (MSDC) and the Lead Local Flood Authority (LLFA, West Sussex County Council) that all potential flood risks to and from the proposed development have been considered and that the proposed development is appropriate, as defined in the NPPF;
- Consideration of appropriate Site-specific flood risk mitigation measures; and,
- Development of an outline SuDS strategy to mitigate the potential increase in runoff and deterioration in water quality released from the Site, as well as providing amenity and biodiversity benefits.

1.4. Data Sources

The main sources of data utilised in this assessment are summarised below:

- The proposed development plans as provided by the Client;
- LiDAR Digital Terrain Model (DTM) data obtained through data.gov.uk;
- Topographical survey of the Site (provided by the Client);
- Ordnance Survey data;
- Data collected during a Site walkover;
- Environment Agency (EA) flood risk data;
- The Mid Sussex Level 1 Strategic Flood Risk Assessment (Mid Sussex District Council, 2024);
- The West Sussex LLFA Policy for the Management of Surface Water (West Sussex County Council, 2018);

- A guide for master planning sustainable drainage into developments in southeast of England (AECOM, 2013);
- Soilsclapes soil mapping;
- British Geological Survey (BGS) mapping and borehole logs;
- Site specific infiltration test data collected in June 2023; and,
- Southern Water sewer asset location plans

1.5. Limitations

This report is written strictly for the benefit of the Client and bound by the conditions presented in Appendix A.

2. Revisions to report

2.1. August 2025 feedback

This report has been revised/updated based upon feedback received from the Lead Local Flood Authority in August 2025. The following table itemises all comments raised and the revisions that have been made to address each point.

Comment	Response
1. In the greenfield runoff rate calculations, region 1 is being used, which does not reflect the hydrological region for the site. Please send FEH point data file to FRM@westsussex.gov.uk to confirm the location of the calculation inputs. Do not send this to the planning officer, to ensure the data remains confidential and in accordance with FEH Web Service terms of use. Please title the email: DM/23/2866 FEH point file. It is essential that the correct parameters are used when calculating greenfield runoff rate, otherwise surface water flood risk could increase elsewhere.	The hydrological region has been corrected, and revised greenfield rates have been updated in the report – see Section . There is no change to the peak Q1 flow rate, which remains at 460 l/s, although the calculated peak Q1 flow rates for each sub-catchment have increased very slightly as a lower Q1 value was used in the calculation previously (it is now based on the 460 l/s value). The FEH data has also been issued to FRM@westsussex.gov.uk .
2. In the calculations, runoff volume calculations for existing and proposed scenarios is required.	The calculated pre and post development runoff volumes have been calculated and presented within Section 8.2 and 8.5.
3. As requested previously, evidence of consideration into the use of infiltration drainage in areas away from watercourse corridors will need to be provided at this outline planning stage. No further testing has been completed and the application is currently relying on testing carried out in summer, not winter.	A revised section covering the infiltration potential around the wider site has been included within Section 3.4.4 on the report.
4. At outline planning stage, we require an outline drainage layout showing proposed location of drainage components, storage volumes, outfall locations and a rough idea of flow rates from each outfall. 3m easements will be required around basins and swales. All outfalls to watercourses must be within the red line boundary, or third party agreements will need to be provided at outline stage.	The drainage layout plans within Appendix G have been enhanced to make the requested information more clearly visible.
5. The watercourses must have a 3m easement to allow riparian maintenance to take place.	This information has been added to the drainage layout figures (Appendix G).
6. The ponds are labelled with 'freeboard depth'. We believe this does not reflect freeboard as this should be 300mm following	The freeboard in basins has been clearly defined/displayed as per the SuDS Manual (remaining depth between the max design water

Comment	Response
the SuDS Manual. All SuDS should be designed to follow SUDS Manual.	level - for T30 +CCA event - and the bank level). References in the drawings have been corrected where the "freeboard depth" label may have been misleading.
7. As requested previously, further details regarding the use of the pump and emergency procedures following 24 hour failure are required.	Additional measure/procedures have been provided in Section 8.5.1 to give a more robust strategy to mitigate the risk of a pump failure. .
8. The FRA and Drainage Strategy does not cover how the site will remain safe in times of flooding. As mentioned previously, due to the impact crossing heights can have on surrounding development layouts and proposed site levels it is important this is considered for all watercourse crossings at this early masterplan stage. This is to ensure that alterations to facilitate safe access/egress do not impact the proposed development description or likely masterplan layout.	Cross sections of the flow path/watercourse intersections, illustrating the ground elevations, flood levels and deck levels of crossings, have been included within Section 7.2 to illustrate that intersections will remain dry and accessible during a flood event. The exercise was also undertaken for areas in proximity to the Copyhold Gill and issued to the LLFA previously.
9. We have concerns that there will be a loss of floodplain storage as a result of this development. As mentioned previously, evidence that the impact development could have on floodplain storage, for all sources of flooding has been considered and mitigated against will be required at this stage of planning. This is required at outline application stage due to the potential this could have on developable area on the site and the impact that could have on the development's masterplan.	There is no planned development within the fluvial floodplain area (aside from the bridge crossing as discussed in the FRA). Cross sections at intersections between access routes and surface water flow paths or ephemeral streams have been assessed in Section 7.3 of the report. The crossings will be constructed above the peak flood level, with the crossing utilising open-span style bridge, with no construction within the channels (e.g. no culverts or abutments which may result in a loss of floodplain storage or limit channel flows). . For the few areas of mapped SW flood risk which overlap with development areas (the 1 in 100 year +CCA flooding area in the southern development area), these flood risk areas will not be present post-development owing to the introduction of the drainage infrastructure (i.e. the catchment area will be positively drained, with surface water being directed to the SuDS features rather than ponding on the surface). Calculations have been provided which demonstrate the capacity of the proposed SuDS features to receive and manage runoff within the various site sub-catchments, as well as calculated losses in ponding volume vs volumes provided by representative SuDS features (see Section 7.3).

Comment	Response
10. The current masterplan and drainage strategy shows several access roads and footpaths/cycle links located within areas of increased flood risk and surface water flow paths. We require further evidence that existing flow paths will not be impeded by any type of development at this stage of planning.	As per point 8 above, we have provided cross sections of the flow path/watercourse intersections, illustrating the ground elevations, flood levels and deck levels of crossings. These illustrate that that existing flow paths will not be impeded by any type of development.
11. As mentioned previously the calculations need to use FEH2022 rainfall and Cv value 1.	The calculations have been revised to include the updated FEH-22 rainfall data and a Cv value of 1 for the drainage simulations.
12. In the calculations, there is extensive flooding in the 1 in 100 year plus climate change event, which is likely to increase flood risk elsewhere. Without addressing the above, surface water flood risk is likely to increase elsewhere, as it has not been adequately demonstrated that the site can drain.	We have upsized the SuDS features to accommodate the additional inflows above the T30+CCA design storm. The drainage model and drainage layout plans have been updated to reflect this and none of the features now flood under the T100+CCA event.

2.2. September 2025 feedback

Following the implementation set out above, the LLFA confirmed that the majority of these points had now been addressed. The following amendments were also requested.

Comment	Response
1. Only the northern section of the outline drainage layout has been updated as requested.	Southern area now included.
2. A Cv value 0.75 is still being used in the Design Settings. A Time of Entry of 5 minutes should be used instead of 10 minutes (with some additional variation). The Additional Storage should be 0.	The Design Setting Cv has been revised (although this does not affect the simulation results, just the initial pipe sizing). The ToE and additional storage settings have also been amended as requested. The size and freeboard depths of all features have been adjusted to allow for these changes.

2.3. 2024 feedback

It should also be noted that an additional round of comments was received from the MSDC in 2024. These points were addressed within a separate document (P21367_L1 MSDC DM-23-2866).

The drainage strategy included within this report (Section 8) has also been updated considering the updated 2025 national standards for sustainable drainage design.

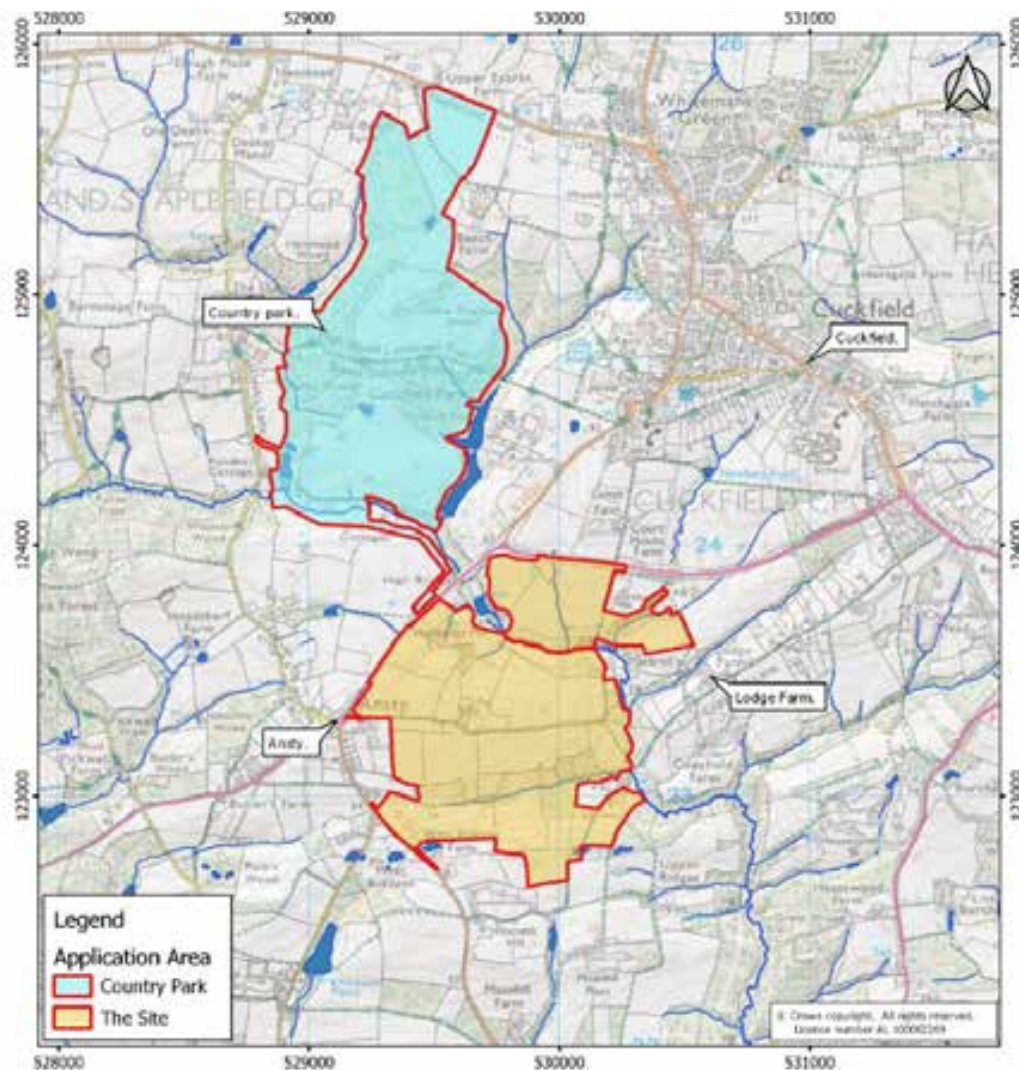
3. Site setting

The following section collates and presents available information pertinent to the Site and its local environs.

3.1. Site location and description

The Site is located to the east of Ansty Village in the District of Mid Sussex, closest postcode is RH17 5AG (see Figure 3-1) The National Grid Reference for the approximate centre of the Site is TQ 29653 23438. The Site covers a total area of approximately 100Ha, and Beechy Bottom Parkland Reserve covers an approximate area of 87Ha.

Figure 3-1 Site location

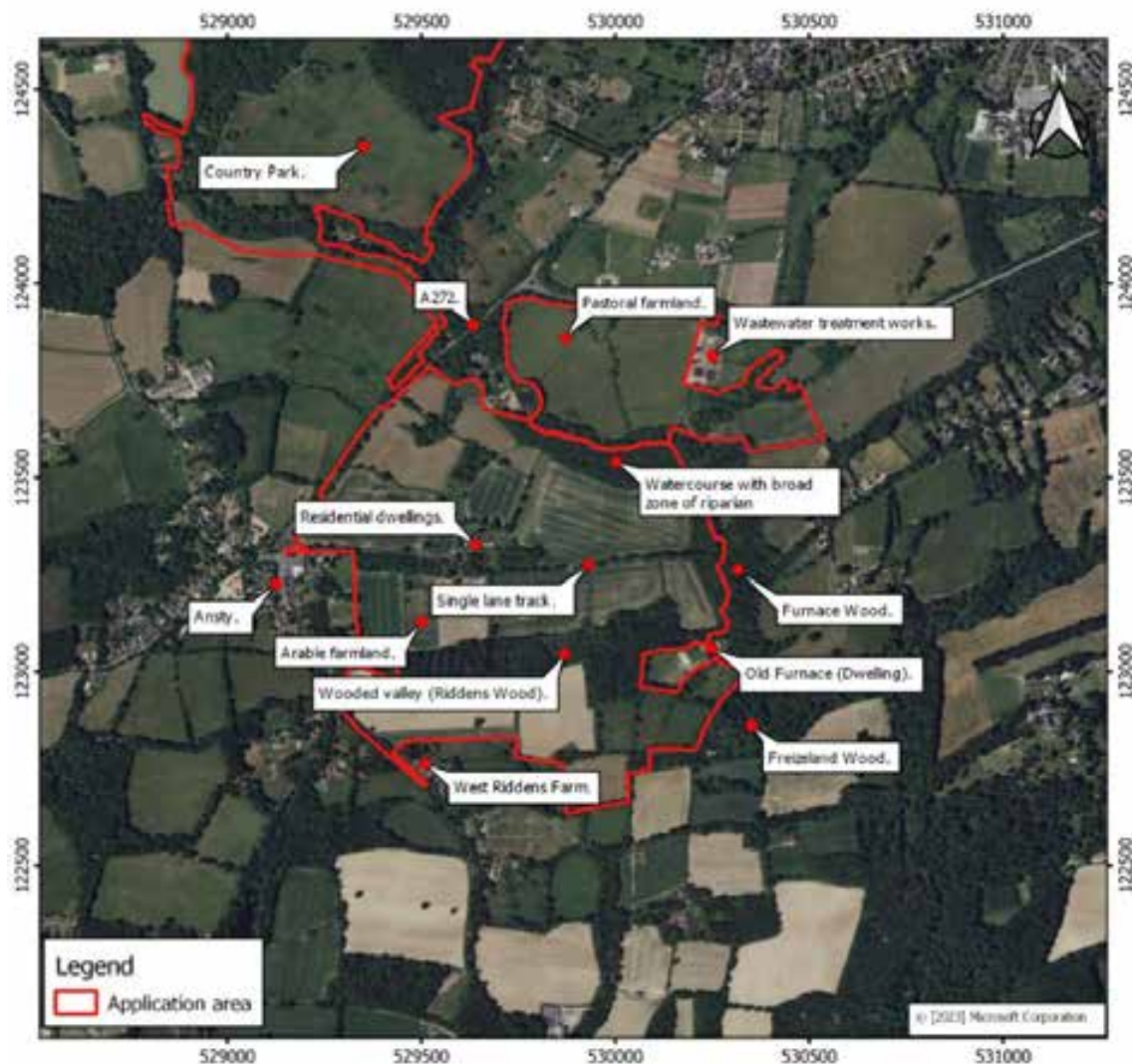


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Figure 3-2 presents an aerial image of the Site showing its current land use and condition. The Site currently comprises agricultural fields separated by hedgerows and ditches. There are also wooded areas in parts of the Site, particularly in the vicinity of the watercourses and overland flowpaths.

The village of Ansty lies at the western extent of the Site, and Cuckfield is located approximately 1km to the northeast. The Site is bounded to the south by more agricultural land. The A272 road bounds the Site to the north. A wastewater treatment works is located adjacent to the northern boundary.

Figure 3-2 Existing Site layout (development area)



The Beechy Bottom Parkland Reserve area to the north is a mixture of mature woodland and fields and Copyhold Gill flows eastwards through the southern end of this area. The Site and the Beechy Bottom Parkland Reserve are separated by the A272.

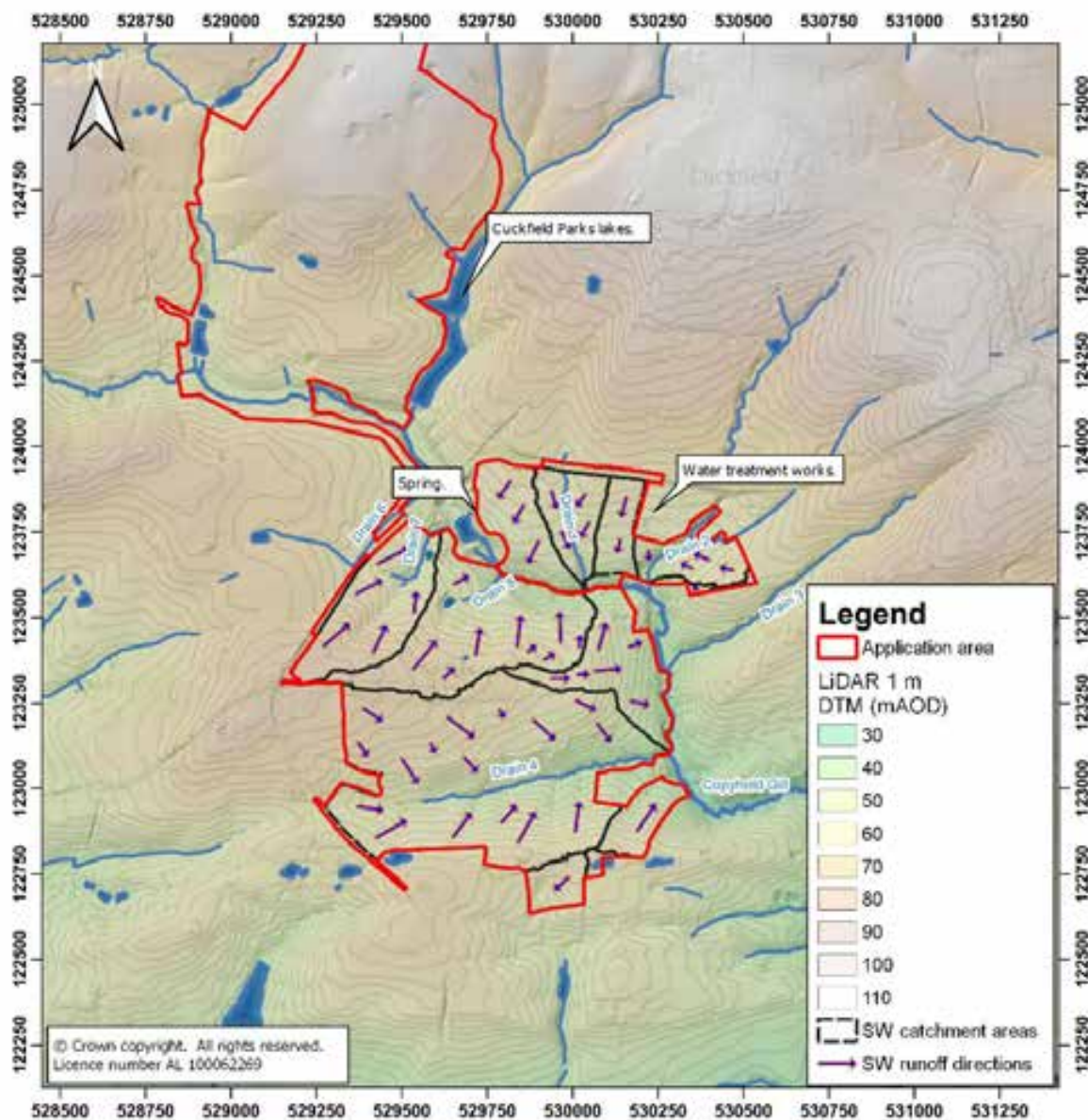
3.2. Topography

Topographical data (from LiDAR) is provided in Figure 3-3. Ground levels at the Site range between approximately 32.85m and 78.37m above Ordnance Datum (m aOD). The Copyhold Gill valley

separates the northern and southern parts of the Site. The areas allocated for development in the proposed plans are located on the elevated parts of the Site, above the valley bottom.

The topography of Beechy Bottom Parkland Reserve falls from an elevated point in the centre (approximately 104m aOD) towards the valleys along the eastern, western and southern boundaries.

Figure 3-3 Site topography and current runoff regime in development area



Contains Open Street Map data © OpenStreetMap

3.3. Site visit and data collection

Site visits were undertaken by Yellow Sub Geo Ltd in January and May 2023. The weather was dry and partly sunny on the day although in January the ground conditions remained very wet and soft following a period of prolonged rainfall. The purpose was to undertake a site walkover and gain an

understanding of the general Site conditions and potential development constraints and/or opportunities.

3.4. Geology and hydrogeology

3.4.1. Published geology and soils

The following information has been compiled Soilscales and British Geological Survey (BGS) 1:50,000 scale mapping. The geological sequence underlying the Site is as follows (see Figure 3-4 for spatial distribution of referenced formations/ features).

3.4.1.1. Soil

Soilscales provides high level information on natural soil characteristics across the UK. Soilscales classifies the soil type at the Site as: 'Slightly acid loamy and clayey soils with impeded drainage' (Cranfield Soil and AgriFood Institute, 2023).

3.4.1.2. Superficial deposits

Superficial deposits are largely absent over the Site area, with the exception of a small area of Head Deposits (silt, sand and gravel) on the western boundary.

3.4.1.3. Solid geology

Much of the Site is underlain by the Upper Tunbridge Wells Sand (interbedded sandstone and siltstone). The northern and southern parts of the Site are also underlain by the Lower Tunbridge Wells Sand (sandstone, siltstone and mudstone), the Lower Grinstead Clay (mudstone) and the Cuckfield Stone Bed (calcareous sandstone)

3.4.2. Published BGS borehole records

Several BGS boreholes are noted in the area (see Figure 3-4). Borehole TQ32SW22 is located at Copyhold Farm, just to the east of the Site, which describes the geology as being a series of clays down to 16m below ground level (m bgl). No groundwater is noted in this log.

BGS borehole TQ22SE25 is located along the northern Site boundary. This log notes clay to a depth of 3.80m bgl over mudstone. Water was encountered at 3.8m bgl, which rose to 1.9m bgl after 15 minutes. This suggests that the groundwater may be confined with the upward migration of groundwater being inhibited by the cohesive overlying material.

BGS borehole TQ22SE14 is located 670m to the southwest of the Site, located on the Upper Tunbridge Wells Sands. This log notes the Weald Clay Formation to a depth of 12m bgl, overlying the Tunbridge Wells Sands (thickness of 33m at this location) and then further Weald Clay Formation below this. Groundwater strikes are not noted in this log owing to the drilling method utilised.

3.4.3. Hydrogeology

The Upper Tunbridge Wells Sand which underlies much of the Site is classified by the EA as a Secondary A aquifer. These are described by the EA as permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as minor aquifers.

The Upper Tunbridge Wells Sand at the Site is classed as being at high vulnerability. The Site is not located within a groundwater source protection zone (SPZ).

3.4.4. Soakaway test results/infiltration potential

On-Site infiltration testing was undertaken in June 2023. The proposed soakaway test locations were chosen to coincide with the proposed attenuation basins supporting the proposed development. Trial pits were typically excavated to a depth of 3m with infiltration testing between 1m and 3m bgl in accordance with methodology set out in BRE365. Full details of the site work undertaken including engineering logs and the results are included in Appendix D.

All test pits failed due to low permeability and waterlogging, consistent with mapped soils (“slightly acid loamy and clayey soils with impeded drainage”) and underlying clay strata. As the testing was undertaken during summer, when soil moisture levels are typically at their lowest, the results are considered representative of best-case conditions. Additional winter testing is not considered necessary. This conclusion is further supported by site geology and hydrogeology data (Upper Tunbridge Wells Sand overlain by cohesive strata).

It is therefore concluded that infiltration is not a viable means of surface water disposal for the proposed development.

3.5. Hydrology

The valley of Copyhold Gill separates the northern and southern parts of the Site and is also present in the south of Beechy Bottom Parkland Reserve. Copyhold Gill is a relatively minor watercourse with a catchment area of around 8.7km² at this location – it is designated as an Ordinary Watercourse.

The Site area includes several small unnamed surface watercourses/ drains and overland flow paths, which drain to Copyhold Gill. Where these features do not have a name, they have been ascribed a numerical ID (see Figure 3-3) for the purposes of this assessment, to avoid confusion.

The Copyhold Gill (see photos in Figure 3-5) is constrained in a steep-sided valley at this location, as is also evident from the fluvial flood risk data presented later.

3.6. Climate

The Standard Average Annual Rainfall (SAAR) for the Site area is 813mm per annum (mm/a).

3.7. Current drainage arrangements

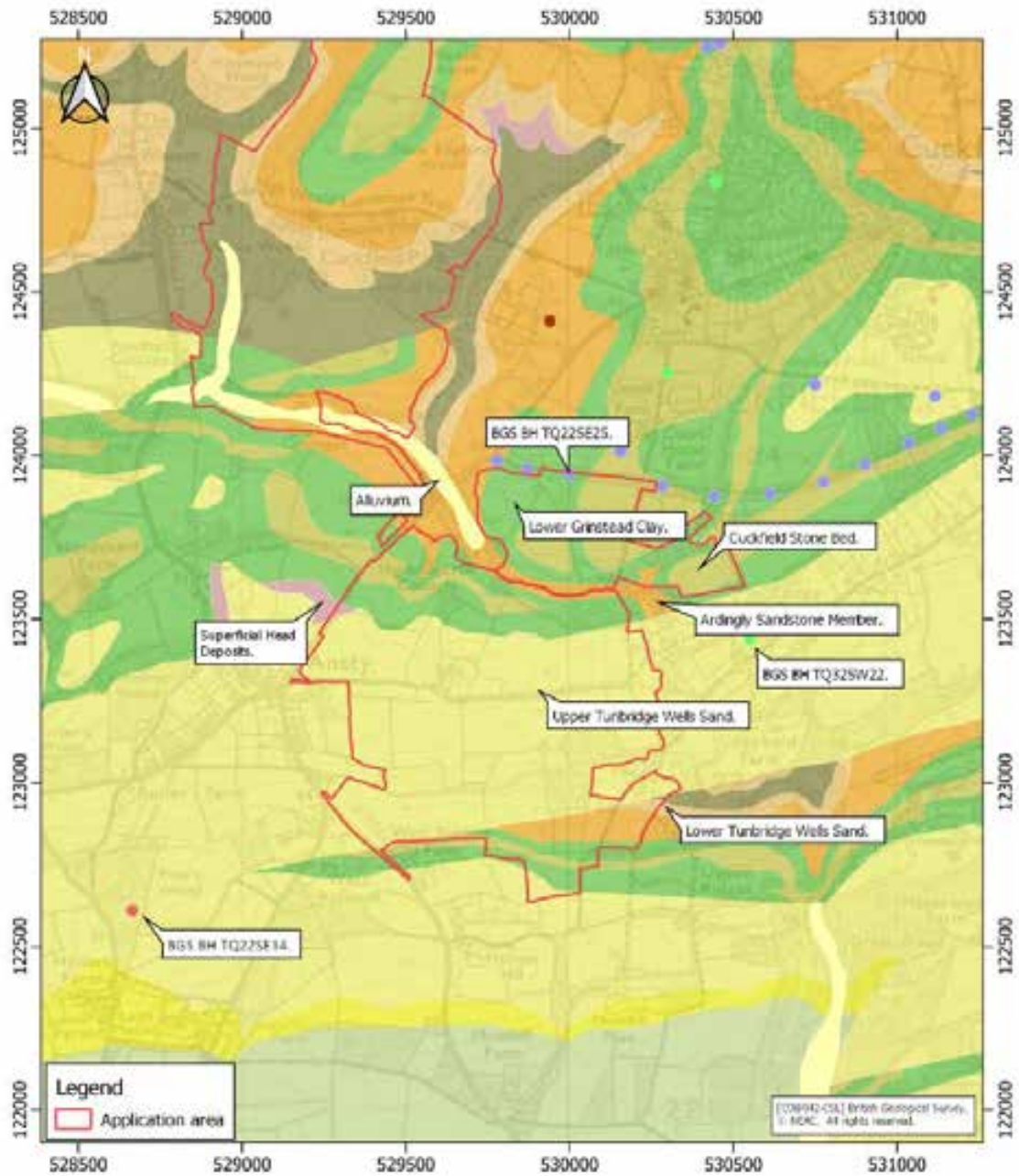
The Site is not currently served by a formal drainage system and rainfall runoff primarily drains to Copyhold Gill (either directly or via one of the smaller tributaries). Figure 3-3 presents the runoff directions and surface water catchment areas for the Site under current conditions. The majority of the southern part of the Site drains to an unnamed drain/ stream (named “Drain 4” in this assessment) which flows through Ridden’s Wood to the confluence with Copyhold Gill. The extreme southern end of the Site drains in a southerly direction to an area of farmland.

Public sewer asset plans for the Site and surrounding local area have been sourced from Southern Water and are presented in Appendix D. There is a foul sewer flowing south into the existing sewage treatment works along the northern boundary, as well as foul sewer assets serving the dwellings in Ansty to the west of the Site. There are no surface water or combined sewers in the vicinity. The dwellings within the centre of the Site (The Barn House, The Grainloft and Old Place) are assumed to be served by private drainage. This likely drains surface water runoff to the adjacent surface water bodies and foul sewerage to closed/ septic systems.

Some photos are included in Figure 3-6 of the on-Site drains and surface water flowpaths. These are incorporated into the Outline SuDS scheme presented in Section 8 which aims to preserve the pre-existing flowpaths and surface water features.

Given the cohesive soils noted during the site visits, the waterlogged conditions and failed soakaway tests, it appears that the majority of rainfall in this area runs off to Copyhold Gill.

Figure 3-4 Superficial deposits and bedrock (1:50,000)



Contains Open Street Map data © OpenStreetMap



Figure 3-5 Site photos collected March 2023. Clockwise from top left: Ground cover in centre of the Site, surface water ponding on-site, on-site pond; Copyhold Gill



Figure 3-6 (Left) Drain 1 in the north (right) Drain 8

4. Proposed development

4.1. The Site

The illustrative masterplan for the Site is provided in Appendix C. The Proposed Development will create a new Garden Community and separate Beechy Bottom Parkland Reserve, comprising of the erection of up to 1,450 homes (including 30% affordable), up to 90 residential care units (C2 class), a primary school, new special educational needs and disabilities (SEND) school, sports facilities including all weather hockey pitches and tennis centre, allotments, retail, community and employment uses together with ancillary and associated development including new and enhanced pedestrian/cycle routes, open spaces, and landscaping

To support the outline planning application, development zones/ parameters have been identified, rather than defined individual plots, gardens and buildings.

The Site will be accessed via three points, one in the south (B2036), the west (the A272 near Ansty) and the north (the A272). The principle internal access road will be surfaced with impermeable hardstanding, with minor access roads and driveways surfaced with permeable material.

Areas of mature vegetation and overland flow routes (such as the Ridden's Wood valley) will be retained/ enhanced. Additional surface water features will be installed downgradient of the development areas to manage surface water runoff from the various sub-catchments of the developed Site as close to the source as possible.

4.2. Beechy Bottom Parkland Reserve

Beechy Bottom Parkland Reserve occupies approximately 87Ha of land to the north-west of the Site, to the west of Cuckfield and north of Ansty. It comprises approximately 80Ha of agricultural land and 7Ha of private recreation grassland. The proposed development will comprise change of use of farmland and woodland to parkland reserve to include public access and instigation of long-term management and rewilding regime, including establishment of pedestrian and cycle tracks, with new pedestrian and cycle access points off Cuckfield Road to the south and Staplefield Road to the north. Proposals to include the addition of two wooden viewing platforms. Sports pitches at Beech Farm Field to remain in sports use.

As noted in Section 1.2 Beechy Bottom Parkland Reserve is covered separately and is not discussed further herein.

5. Flood risk to the proposed development

5.1. Fluvial and tidal

The EA Flood Map for Planning for the Site area is presented in Figure 5-1. The definitions of each flood zone are presented within Table 5-1.

The majority of the Site is in Flood Zone 1, except for areas along the banks of Copyhold Gill and Drain 4, which are classed as being within Flood Zones 2 and 3. All of the proposed developed areas are in Flood Zone 1. All areas in Flood Zones 2 and 3 will have 'Water Compatible' land uses. Figure 5-1 EA fluvial flood risk data

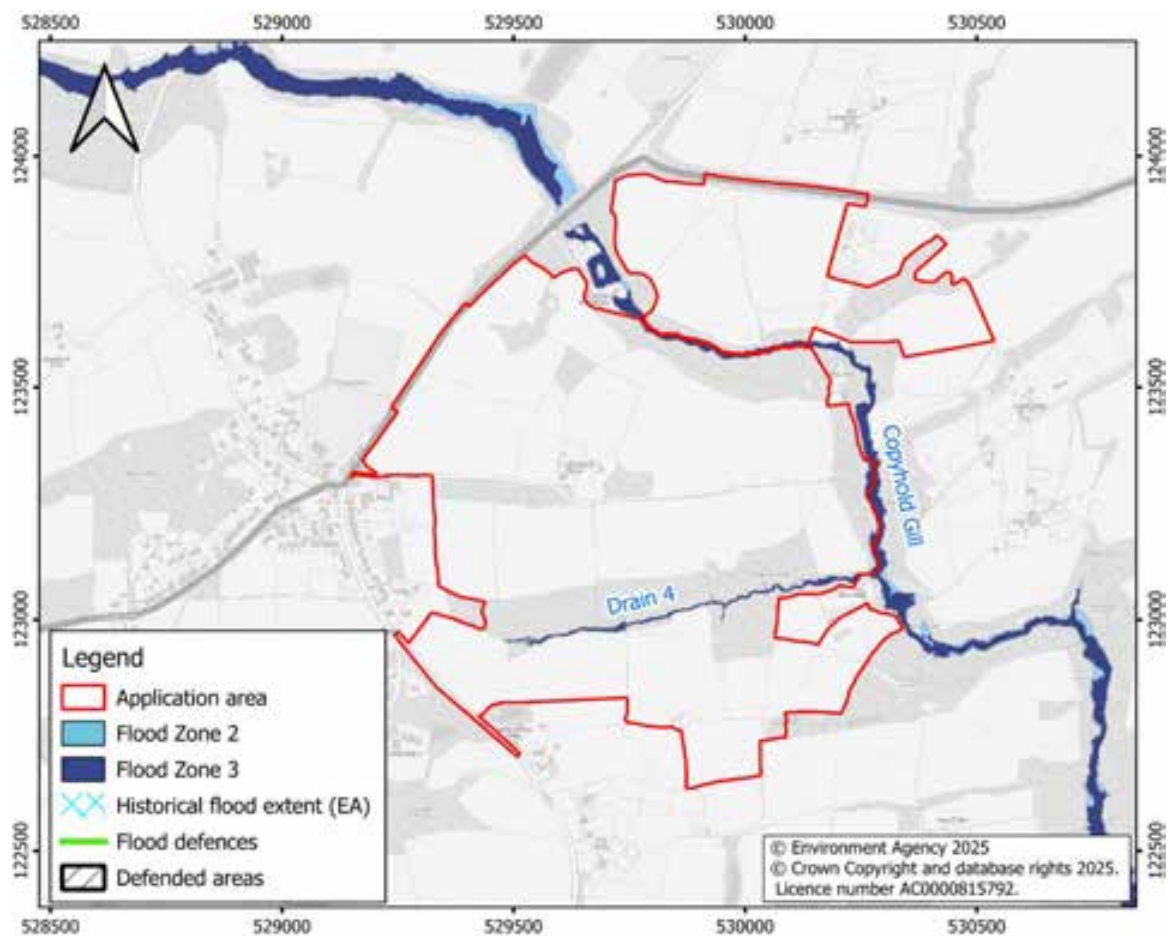


Table 5-1 EA Flood Zone Definitions

Flood zone	Definition
Zone 1 Low Probability	Land having a less than 1 in 1,000 annual probability of river or sea flooding. (Shown as 'clear' on the Flood Map – all land outside Zones 2 and 3)
Zone 2 Medium Probability	Land having between a 1 in 100 and 1 in 1,000 annual probability of river flooding; or land having between a 1 in 200 and 1 in 1,000 annual probability of sea flooding. (Land shown in light blue on the Flood Map)
Zone 3a High Probability	Land having a 1 in 100 or greater annual probability of river flooding; or Land having a 1 in 200 or greater annual probability of sea flooding. (Land shown in dark blue on the Flood Map)
Zone 3b The Functional Floodplain	This zone comprises land where water has to flow or be stored in times of flood. Local planning authorities should identify in their Strategic Flood Risk Assessments areas of functional floodplain and its boundaries accordingly, in agreement with the Environment Agency. (Not separately distinguished from Zone 3a on the Flood Map). Mid Sussex Level 1 SFRA defines this as having a 1 in 20 or greater annual probability of flooding.

Source: Environment Agency: Flood risk and coastal change guidance, accessed 07/11/2023

5.1.1. Fluvial flood model data

The EA were contacted to request fluvial flood model data. None currently exists for Copyhold Gill.

5.2. Flood defences

The Site is not protected by formal flood defences according to EA data and Site observations. The watercourse is constrained in a well-defined valley at this location.

5.3. Surface water

Surface water (pluvial) flooding is usually associated with extreme rainfall events but may also occur when rain falls on land that is already saturated or has a low permeability. Rainfall that is unable to infiltrate into the ground generates overland flow which can lead to flooding or 'ponding' in localised topographical depressions before the runoff is able to enter local drainage systems and watercourses.

A map of EA's Risk of Flooding from Surface Water (RoFSW) data is shown in Figure 5-2 and the risk of surface water flooding reaching or exceeding depth of 0.2 m and 0.6 m are shown in Figure 5-3 and Figure 5-4, respectively. This data includes the 'Central' climate change allowance for the 2050s epoch (2040-2060).

Most of the Site is at negligible risk of surface water flooding. There are multiple runoff flowpaths with elevated (Low to High) flood risk along the bottom of the valleys of Copyhold Gill and its tributaries, most notably Drains 1, 2, 4 and 7. There are also multiple small areas of elevated surface flood risk in the central, southern and northern Site area caused by localised topographical depressions.

The majority of the elevated flood risk areas are modelled to reach flood depths of at least 0.2 m, with central ponds and the lower reaches of Drain 4 reaching / exceeding flood depth of 0.6 m.

There are no identified critical drainage areas (CDA) within proximity of the Site according to the SFRA (Mid Sussex District Council, 2024).

Figure 5-2 EA flood risk from surface water (with climate change allowance)

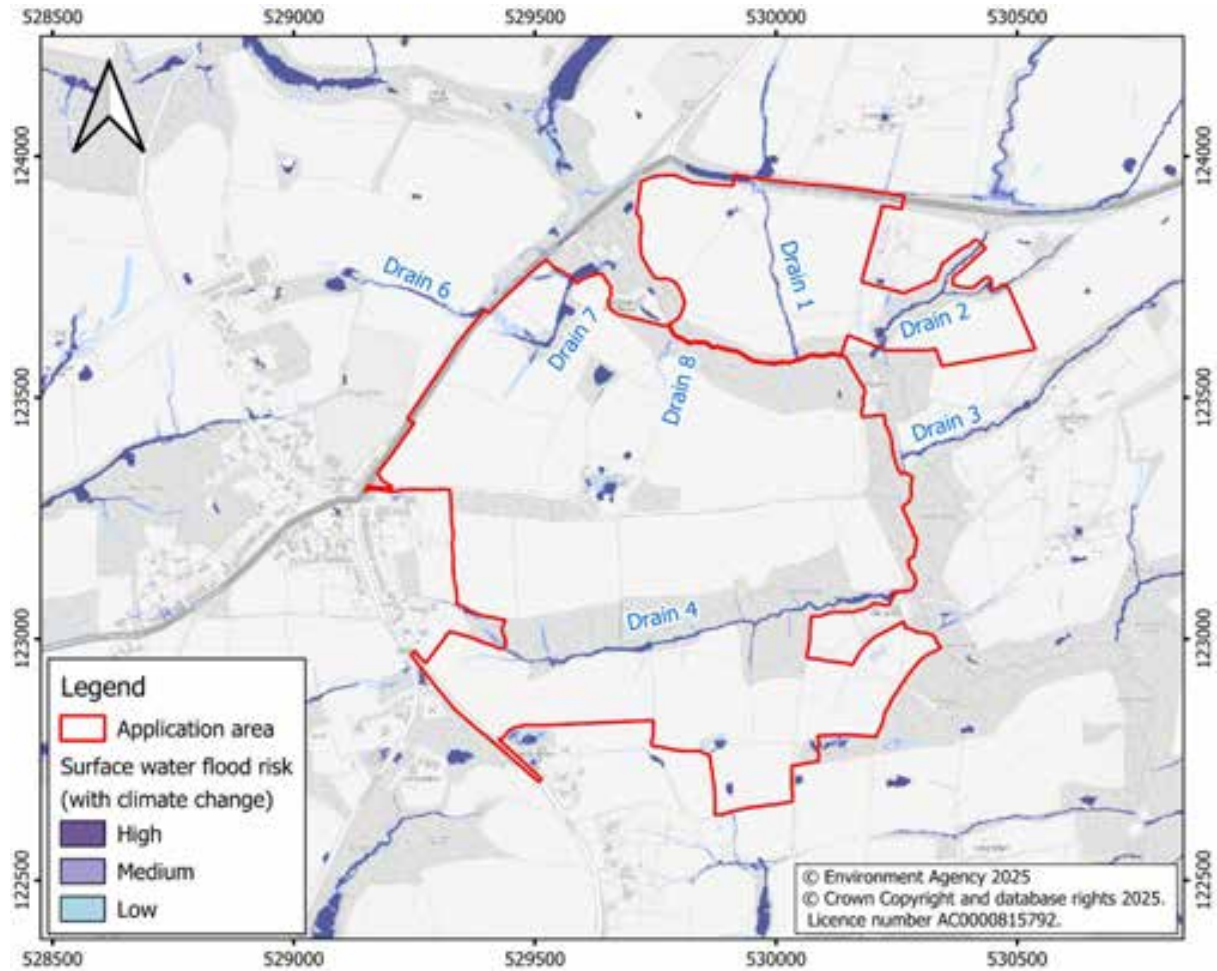


Figure 5-3 EA surface water flood depth risk (0.2m; with climate change allowance)

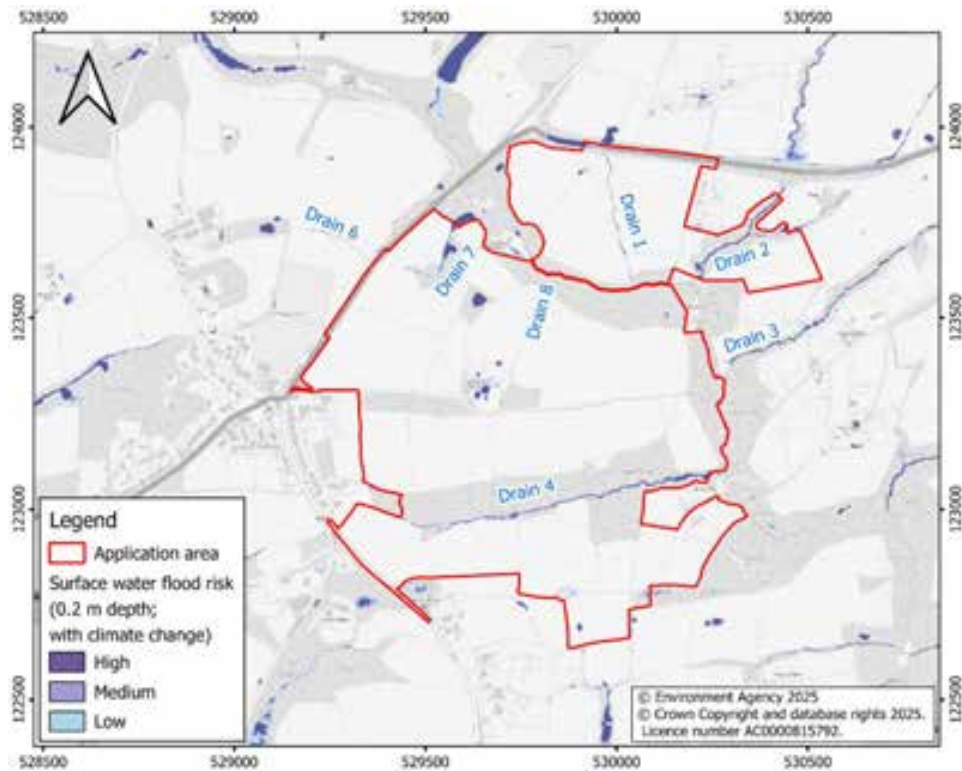


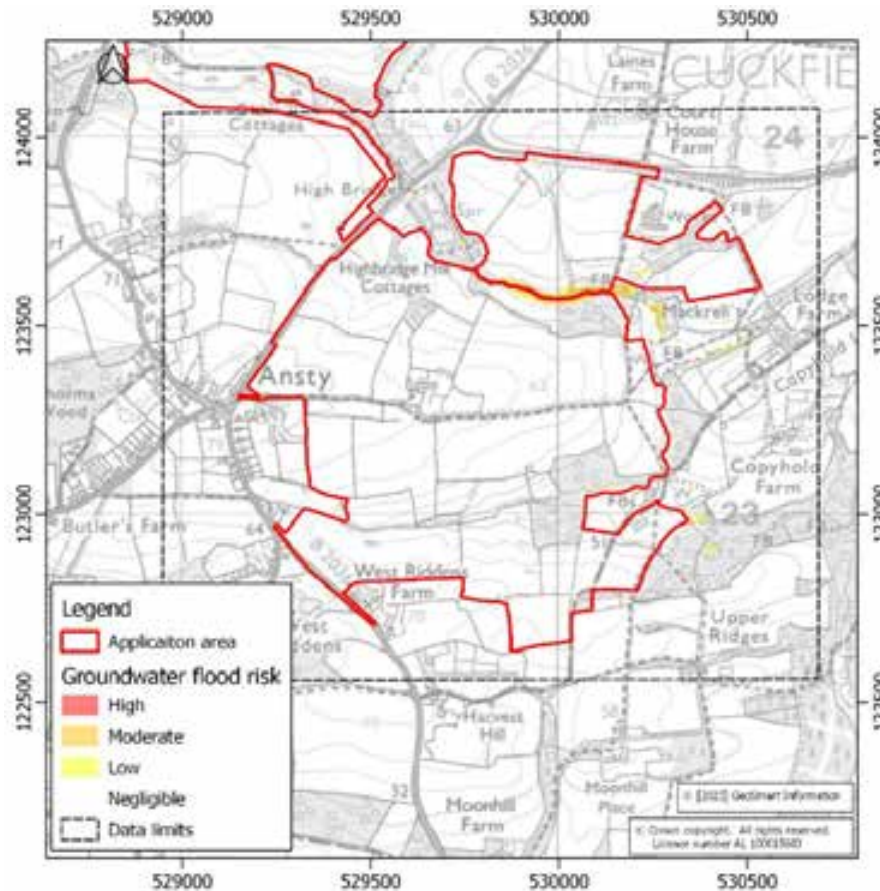
Figure 5-4 EA surface water flood depth risk (0.6m; with climate change allowance)

5.4. Groundwater

Groundwater flooding occurs when the water table rises above the surface elevation (or the floor of sub-surface structures).

Commercial groundwater flood risk data has been purchased for this assessment and is presented below in Figure 5-5. This illustrates that the majority of the Site is classified as being at negligible risk of groundwater flooding. Some areas of elevated groundwater flood risk (Low and Medium risk) are noted in the valley bottom of Copyhold Gill and Drain 2, where there will likely be some emergence of groundwater into the watercourses. The areas of elevated groundwater flood risk do not overlap with any of the proposed development zones.

Figure 5-5 Flood Risk from Groundwater Ambient Risk Analytics



5.5. Sewer flooding

Sewer flooding can occur during periods of intense rainfall and/ or if a sewer becomes blocked with debris. The Site is not currently served by a sewer system but is adjacent to some residential streets which would be.

The Mid-Sussex SFRA does not identify the Ansty area as having issues with sewer flooding, however there have been 11 incidents reported over the last 10 years in post code district RH17 5, which includes Ansty and Cuckfield. Burgess Hill is also noted as having previously experienced regular sewer flooding (Mid Sussex District Council, 2024).

5.6. Catastrophic failures

This section considers catastrophic failures of water bearing infrastructure in the area of interest.

The risk of reservoir flooding is related to the failure of a large water storage reservoir. The maximum expected extent of flooding from reservoir failures does not encroach on the Site area according to EA data.

5.7. Historical flooding

The EA database of historical flooding contains no evidence of flooding having occurred in this area. The Mid Sussex SFRA contains no references of historical flooding occurring at the Site area. Water levels in Copyhold Gill and its tributaries will have reached extreme, flood levels in the past, during

storm conditions. Given the steep sided valleys and the relatively large range of ground elevations across the Site, this is unlikely to have caused flooding of the wider Site area – as illustrated in the Flood Zone 2 and surface water flood risk extents.

6. Suitability of the proposed development

The National Planning Policy Framework requires that “the sequential test is used in areas known to be at risk now or in the future from any form of flooding, except in situations where a site-specific flood risk assessment demonstrates that no built development within the site boundary, including access or escape routes, land raising or other potential vulnerable elements, would be located on an area that would be at risk of flooding from any source, now and in the future”.

The Planning Practice Guidance note on Flood risk and coastal change provides a Flood risk vulnerability and flood zone ‘incompatibility’ table (see Table 6-1) indicating where proposed development should be allowed with respect to the fluvial and tidal flood zones. The Site is mostly located within Flood Zone 1 with the developable area entirely in Flood Zone 1 (see Figure 5-1), and therefore should be acceptable for all types of development without the requirement of an Exception test. This assessment however only accounts for the fluvial and tidal flood risk to the Site, and not the surface water flood risk which is known to exist across the Site.

Table 6-1 Flood risk vulnerability and Flood Zone compatibility

Flood risk vulnerability classification	Essential Infrastructure	Water Compatible	Highly Vulnerable	More Vulnerable	Less Vulnerable
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	✓	Exception test required	✓	✓
Zone 3a	Exception test required	✓	✗	Exception test required	✓
Zone 3b	Exception test required	✓	✗	✗	✗

Source: PPG for Flood Risk and Coastal Change

6.1. Sequential development

Where flood risk is present to varying degrees across a Site, layouts should be tailored to ensure the most sensitive parts of a development are located in parts of the Site least at risk of flooding.

For the Site, all development is located within Flood Zone 1. Areas at high risk of surface flooding are predominantly confined to existing woodland areas and areas of soft landscaping. The remaining areas at risk of surface water flooding are likely to change as the Site is re-profiled, particularly due to their drainage catchment being entirely, or almost entirely contained within the Site boundary, and due to runoff being collected by the proposed Sustainable Urban Drainage Scheme for the Site (detailed below).

As the Site as a whole includes areas within Flood Zone 3 and surface water flow paths, a Sequential Test has been completed (ref: P25035_R1, May 2025).

6.2. Flood risk from the proposed development

The NPPF stipulates that all new developments must be “safe, without increasing flood risk elsewhere”. As such the following stipulations are provided in the EA guidance for managing rainfall runoff:

Stormwater runoff rates and volumes discharged from urban developments should approximate to the Site greenfield response over a range of storm frequencies of occurrence (return periods); and,

Runoff for extreme events should be managed on-site. This requires:

- The peak rate of stormwater run-off to be limited.
- The volume of run-off to be limited.
- The pollution load to receiving waters from stormwater runoff to be minimised.
- The assessment of overland flows and temporary flood storage across the Site.

A Sustainable Drainage Strategy will be drafted for the Site, which is designed in such a way as to prevent an increase in runoff from the Site under a range of design storm scenarios. This includes suitable allowances for future increases in rainfall intensity caused by climate change.

In particular the Sustainable Drainage Strategy references the surface water flow paths crossing the Site, and how that will be captured and incorporated into the drainage design to reduce flood risk to the Site and from the Site.

7. Flood risk mitigation measures

7.1. Key considerations

To meet the PPG requirements, the proposed development will be considered appropriate in this location provided the following conditions are met:

- Remains safe in times of flooding whilst taking climate change into account;
- Does not result in a net loss of floodplain storage;
- Does not impede existing water flow pathways; and,
- Does not increase the volume and rate of surface water runoff leaving a site over its intended design lifetime.

Each of these requirements is discussed in relation to the proposed development in Sections 7.2 to 7.5 below.

7.2. Remain safe in times of flooding

The development areas are in Flood Zone 1 and will be at negligible risk of flooding from rivers (and the sea), surface water and groundwater.

There are eight proposed crossings over Copyhold Gill and other on-site surface water flow paths (see Figure 6 1). Each crossing will be an open-span bridge, with the soffit level elevated at least 300mm above the peak 1-in-100 year +CCA flood water level. Figure 6 2 presents cross-sections at each location, showing the local topography, modelled flood water levels (extracted from LiDAR data at modelled flood extents), and proposed crossing. (soffit) elevations. The cross-sections indicate that safe crossings during times of flooding can be achieved without altering local topography or reducing watercourse capacity.

Figure 7-1 Location of watercourse/ flow path intersection cross-sections

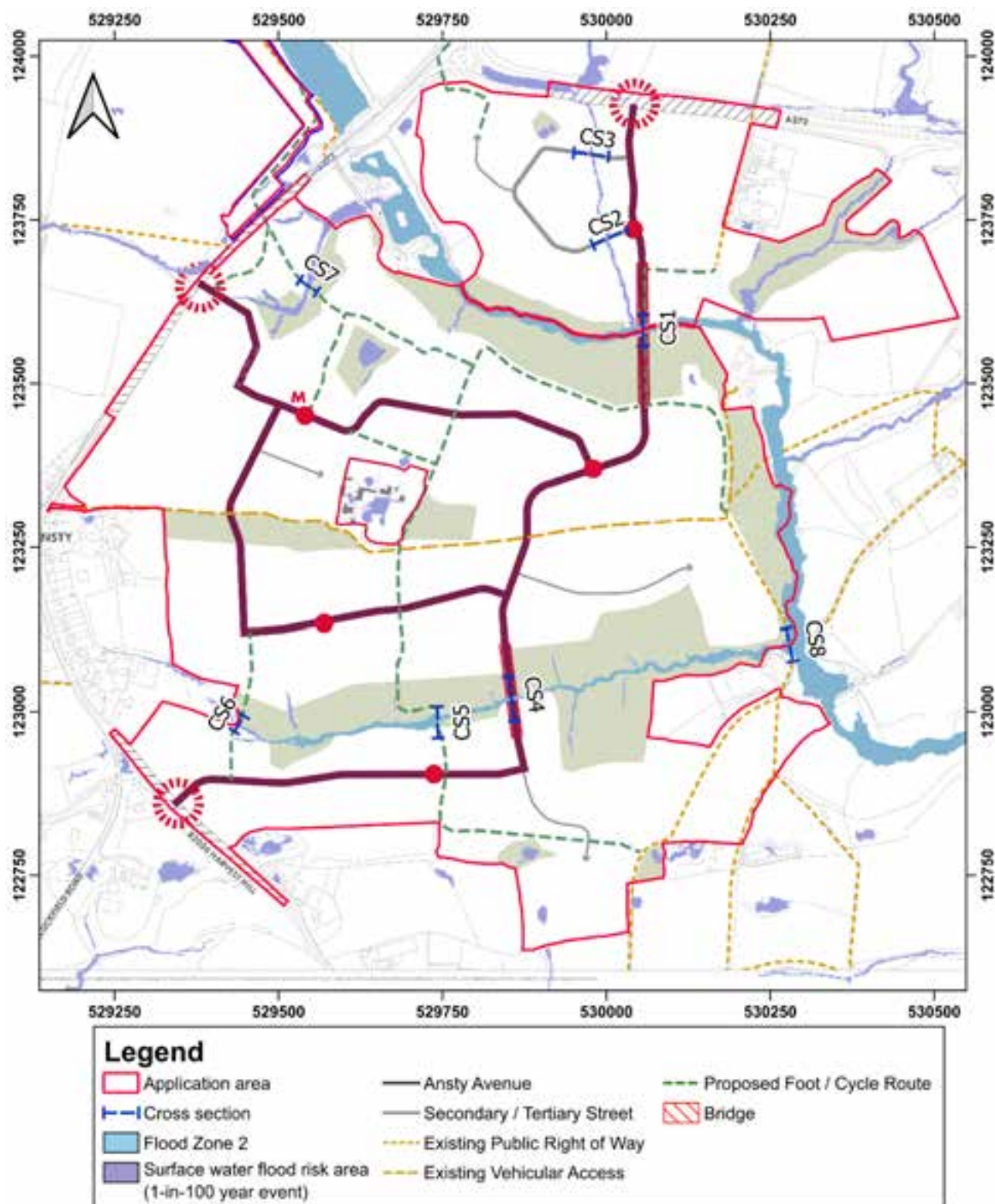
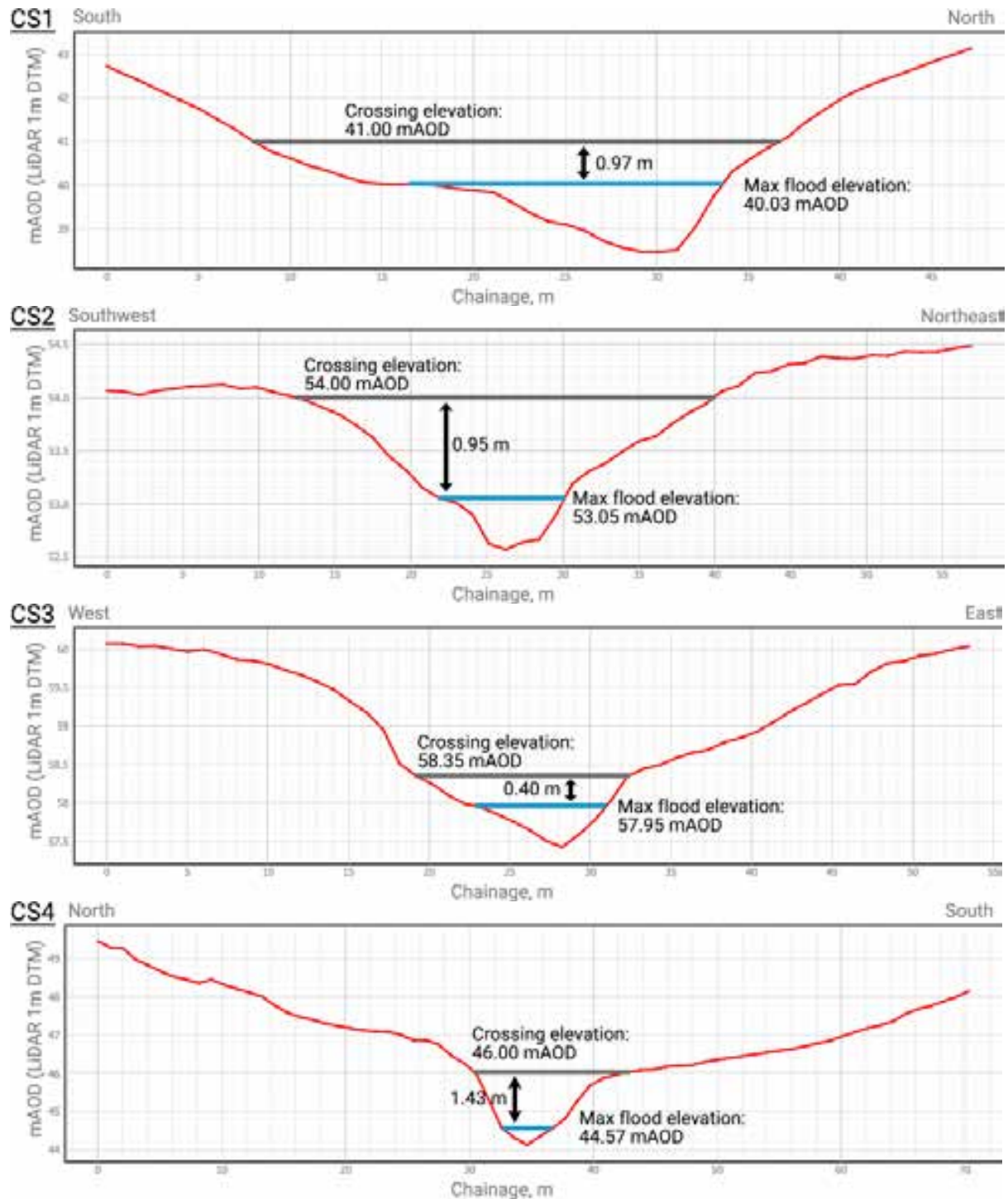
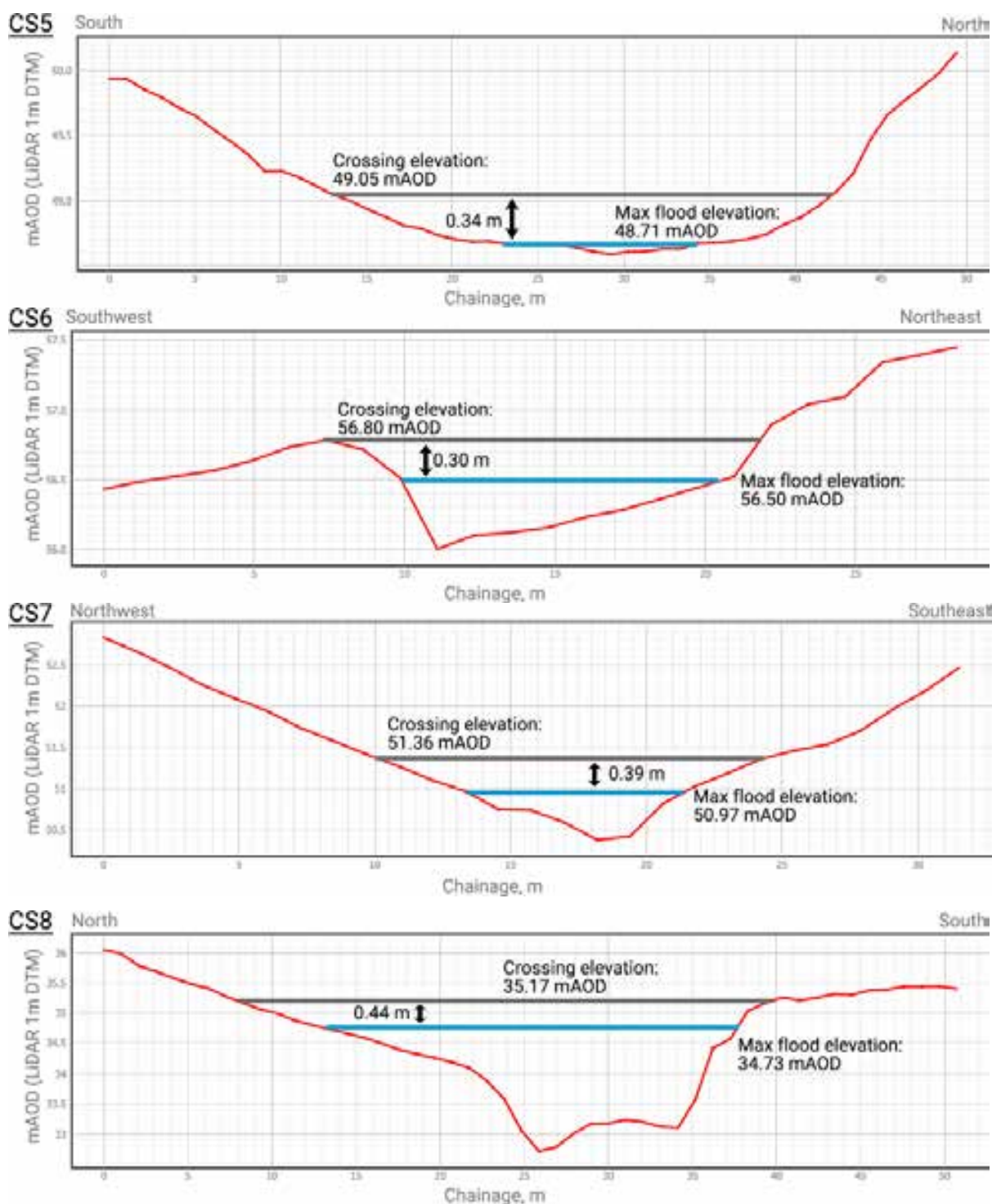


Figure 7-2 Watercourse crossing (soffit) and flood risk elevation cross-sections





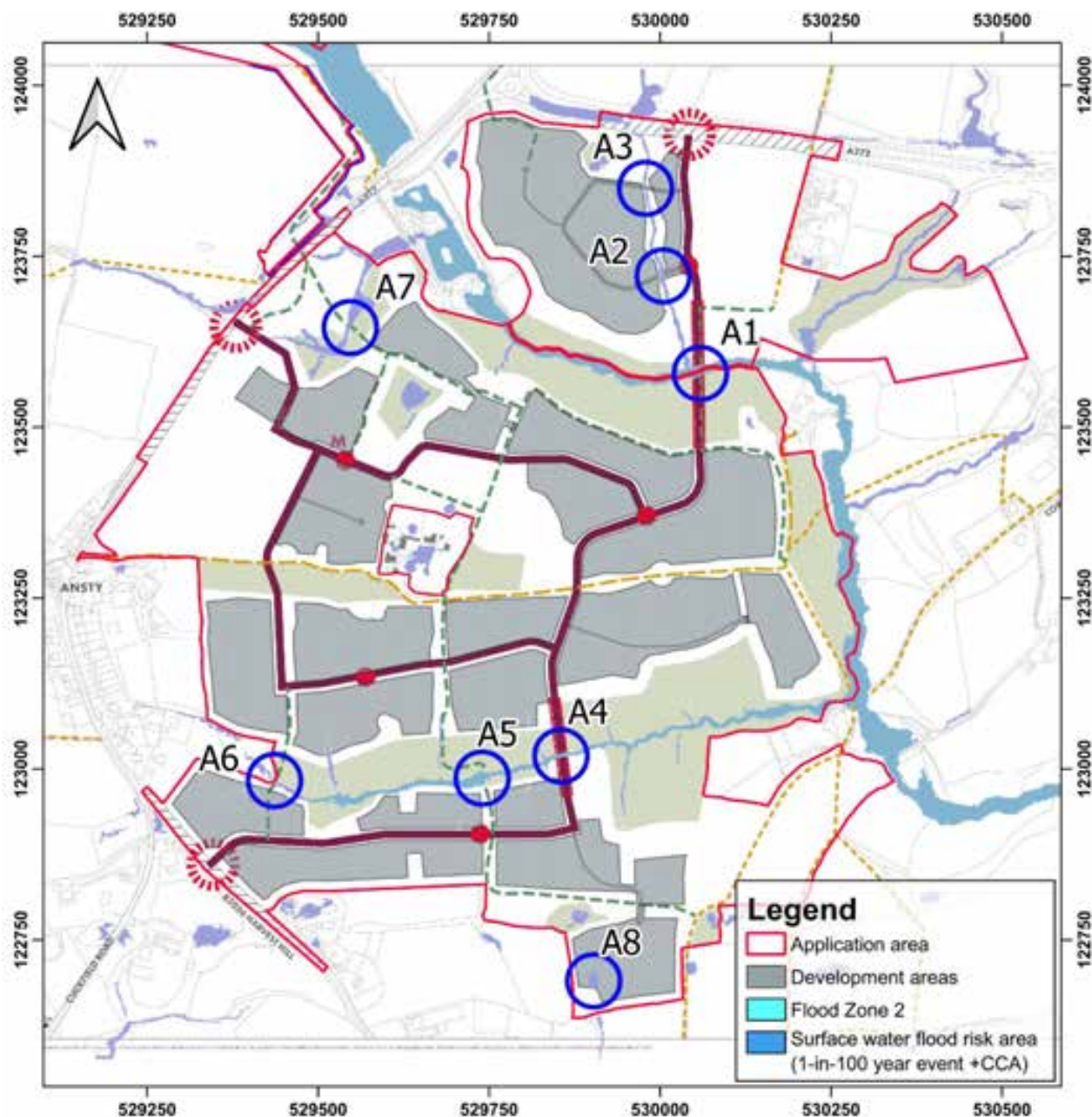
7.3. No net loss of floodplain storage

There is no development proposed within a fluvial floodplain. There are eight areas where the proposed development overlaps with areas with areas of elevated surface water flood risk (see Figure 7-3), based upon the peak modelled extent of the T100+CCA event. Seven of these areas (A1-A7) are associated with crossings over flow paths (minor valley features), which will not be impacted. As discussed in Section 7.2, the crossings will be designed to avoid any potential losses in floodplain

storage and to provide sufficient flow capacity for extreme storm flows to pass underneath. This will be achieved by installing clear-span bridges over the channel, with the channels below left intact and undisturbed. The soffit levels of the bridges will be set at least 300 mm above the expected maximum flood level at each location.

Overlap area A8 is associated with a small (c. 295 m²) area of elevated surface water flood risk caused by localised topographical ponding; this equates to 59m³ when considering a uniform depth of 0.20 m (a conservative assumption based on the EA SW flood depth data). The ponding in this location will not be present post-development as the catchment area will be served with drainage infrastructure. The loss in the flood storage capacity will be more than compensated with the introduction of basin P13 (capacity 630 m³ as well as the storage provided by the pipes, manholes). The overall extent of surface water flooding on-site will reduce relative to the current/baseline scenario owing to the influence of the sustainable drainage network.

Figure 7-3 Flood risk and development area overlap areas



7.4. No impediments to flood water flows

An overland flowpath is noted in the north-western part of the Site (along drains 6 and 7). These drain an area to the west of the Site to Copyhold Gill through an area of woodland. This will be retained post-development, and does not extend over the proposed development footprint. There is a smaller 'low risk' flow path draining southwest to northeast in the same area, and joining drain 7 within the wooded area, which does overlap the proposed development footprint. This flow path is derived from Site runoff and will be managed by the drainage infrastructure post-development.

Several crossing are proposed over Drain 4 and one over Copyhold Gill. A suitable design will be required to ensure that these do not impede storm flows such that local flood risk is increased.

7.5. No increase in the volume and rate of surface water runoff

The Sustainable Drainage Strategy discussed in Section 8 would ensure that runoff rates and volumes are not increased as a result of the proposed development.

8. Outline Sustainable Drainage (SuDS) Strategy

8.1. Introduction

The following sections describe the outline SuDS Strategy for the proposed development with due regard to DEFRA's 2025 Non-Statutory Technical Standards for SuDS (DEFRA, 2025) and the policy for surface water drainage on major developments in West Sussex (West Sussex County Council, 2018), which recommends the following hierarchy for the disposal of surface water from new developments:

- 1 Discharge to ground via infiltration techniques (most preferred);
- 2 Discharge to a surface water body;
- 3 Discharge to a surface water sewer; local highway drain or another drainage system; and,
- 4 Discharge to a combined sewer (least preferred).

The 2018 WSCC Policy for the Management of Surface Water provides individual SuDS policies by which planning applications are determined against. These policies are summarised in Table 8-1.

Table 8-1 Individual SuDS policies included in the WSCC's 2018 Policy for the Management of Surface Water

Policy	Summary
SuDS Policy 1	Follow the drainage hierarchy
SuDS Policy 2	Manage Flood Risk Through Design
SuDS Policy 3	Mimic Natural Flows and Drainage Flow Paths
SuDS Policy 4	Seek to Reduce Existing Flood Risk
SuDS Policy 5	Maximise Resilience
SuDS Policy 6	Design to be Maintainable
SuDS Policy 7	Safeguard Water Quality
SuDS Policy 8	Design for Amenity and Multi-Functionality
SuDS Policy 9	Enhance Biodiversity
SuDS Policy 10	Link to Wider Landscape Objectives

The policies presented within WSCC's Policy for the Management of Surface Water are broadly consistent with the National Planning Policy Framework (NPPF), National Planning Practice Guidance and Defra. National Standards for Sustainable Drainage. We understand that WSCC also produced an additional guidance document in 2023 for the implementation of sustainable drainage strategies. The 2023 document could not be located at the time of this report, but this drainage strategy aligns with the 2025 standards, which WSCC have stated should be adhered to.

The proposed residential development will be located on previously undeveloped, 'greenfield' land. A significant proportion of the Site would comprise impermeable areas following its development (for example, roofs and some of the main access roads). Without appropriate management, this would result in a significant increase in both the volume and rate of surface runoff generated by the proposed development, which could lead to an increase in surface water flood risk elsewhere (i.e. downstream). Surface runoff from the developed Site will, however, be sustainably managed using SuDS, as described in the following sections.

SuDS aim to mimic the natural drainage characteristics of a site prior to its development by controlling surface water runoff as close to where the rain falls as possible e.g. through interception and re-use, evaporation and infiltration into the ground. Furthermore, SuDS provide opportunities to remove pollutants from runoff and also provide amenity and biodiversity benefits.

No SuDS are proposed in the Beechy Bottom Parkland Reserve area, given that no development is planned in this area. Therefore this Section focuses on the proposed housing development area (the Site).

8.2. Greenfield runoff and permissible discharge rates

The Institute of Hydrology Report 124 (IH124) method in the 'Rural Runoff' calculator within the industry standard software Causeway Flow+ was utilised to estimate the greenfield runoff rates for the existing Site as a whole (see Appendix F). It would seem appropriate to consider the bulk runoff rates at this point given that the whole Site is thought to ultimately discharge to the Copyhold Gill, either directly or via the smaller on-Site tributaries. This total value has been scaled down for each of the post-development sub-catchment areas in the following Outline Drainage Strategy, to define suitable flow control rates in each location.

The 'QBAR' (i.e. 1 in 2.3 year return period) greenfield runoff rate for the existing Site was determined along with runoff rates for other relevant return period storms using the regional growth curve (see Table 5). The greenfield runoff volumes were calculated using the SPR value of 47% and are also included in Table 8-2 (not that these have been updated in this report to represent hydrological region 7).

Table 8-2 Greenfield runoff

Storm return period(yrs.)	Peak greenfield runoff rate(l/s)	Total greenfield runoff volume
1	460.0	11,844
2.3 (Qbar)	541.2	14,135
30	1298.9	25,596
100	1726.4	33,033

The greenfield QBAR runoff rate or 2 l/s/ha (whichever is greater) is generally set as the 'permissible discharge rate' for new developments. However, the WSCC's Policy for the Management of Surface Water states that:

"Discharge to a watercourse or surface water sewer must be restricted to the estimated mean greenfield runoff rate (Q1) by means of a controlled outflow..."

Q1 is defined as the greenfield 1 in 1 year flow rate; in this case the Q1 rate would be 460.0 l/s for the Site as a whole (but has been scaled down for the various sub-catchments of the Site as discussed below).

8.3. Climate change

The potential increase in rainfall intensity needs to be considered when designing drainage strategies. The recommended allowances for rainfall intensity in the Adur and Ouse Management Catchment are included in Table 8-3.

Table 8-3 Climate change allowances for rainfall in the Adur and Ouse Management Catchment

Epoch	Central allowance	Upper end allowance
1 in 30 year (3.3%)		
2050s	20%	35%
2070s	20%	40%
1 in 100 (1%)		
2050s	20%	45%
2070s	25%	45%

The EA guidance for climate change allowances in flood risk assessments (Environment Agency, 2022) recommends designing development so that, with the upper end allowances for the 1 in 100 year event:

- “there is no increase in flood risk elsewhere; and
- your development will be safe from surface water flooding”.

A design lifespan of 100 years has been assumed and the upper end allowances are applicable in runoff/ drainage calculations for the proposed development.

8.4. Runoff destination and proposed SuDS design

On-Site infiltration testing has demonstrated that infiltration-based SuDS are not viable in this location (see Section 3.4.4).

Southern Water asset location plans (see Appendix E) demonstrate that there are no surface water or combined sewers in the vicinity of the Site.

Discharges to surface watercourses (Copyhold Gill and associated tributaries) with suitable attenuation in surface blue/ green infrastructure is considered to be feasible in this instance, given the extensive size and undeveloped nature of the Site. An Outline Sustainable Drainage Strategy has been drafted primarily based on this method of water disposal. This includes swales and basins within each sub-catchment to provide the attenuation required prior to off-site discharge at a controlled rate (<Q1). As the layout plans for the Site progress, the required attenuation will be distributed over a greater number of features which will comprise a “SuDS train” within each sub-catchment, but the level of detail currently provided is considered to be consummate with the current stage of the design and planning process.

Rainwater harvesting (i.e. the use of water butts or more sophisticated tank systems) could be implemented at the Site. These systems collect water from clean surfaces (such as rooftops) for (generally non-potable) use on-site. Rainwater harvesting is particularly useful at sites with a low infiltration potential and limited space for attenuation features. It also has wider sustainability benefits with regards to lowering the water supply demand.

GIS software was used to calculate the total area within the sub-catchments draining to each SuDS feature (see Figure 8-1). The impermeable surface area within each catchment has been estimated based on the housing density in each development zone (see Table 8-4). The final impermeable areas used in the calculations are included in Table 8-5 below – no urban creep has been included in these calculations as the impermeable surface areas utilised are already considered to be conservatively high; this can be added to the calculations at the detailed design phase, once the actual impermeable surface area is known.

The permissible discharge rates for each catchment have been included for each of the sub catchments. This has been scaled down from the total 1 in 1 year peak flow rate for the whole Site

(see Table 5 for the respective catchment areas). This value was used to set the flow control limits from the principal SuDS features

Figure 8-1 Catchment areas and proposed SuDS features

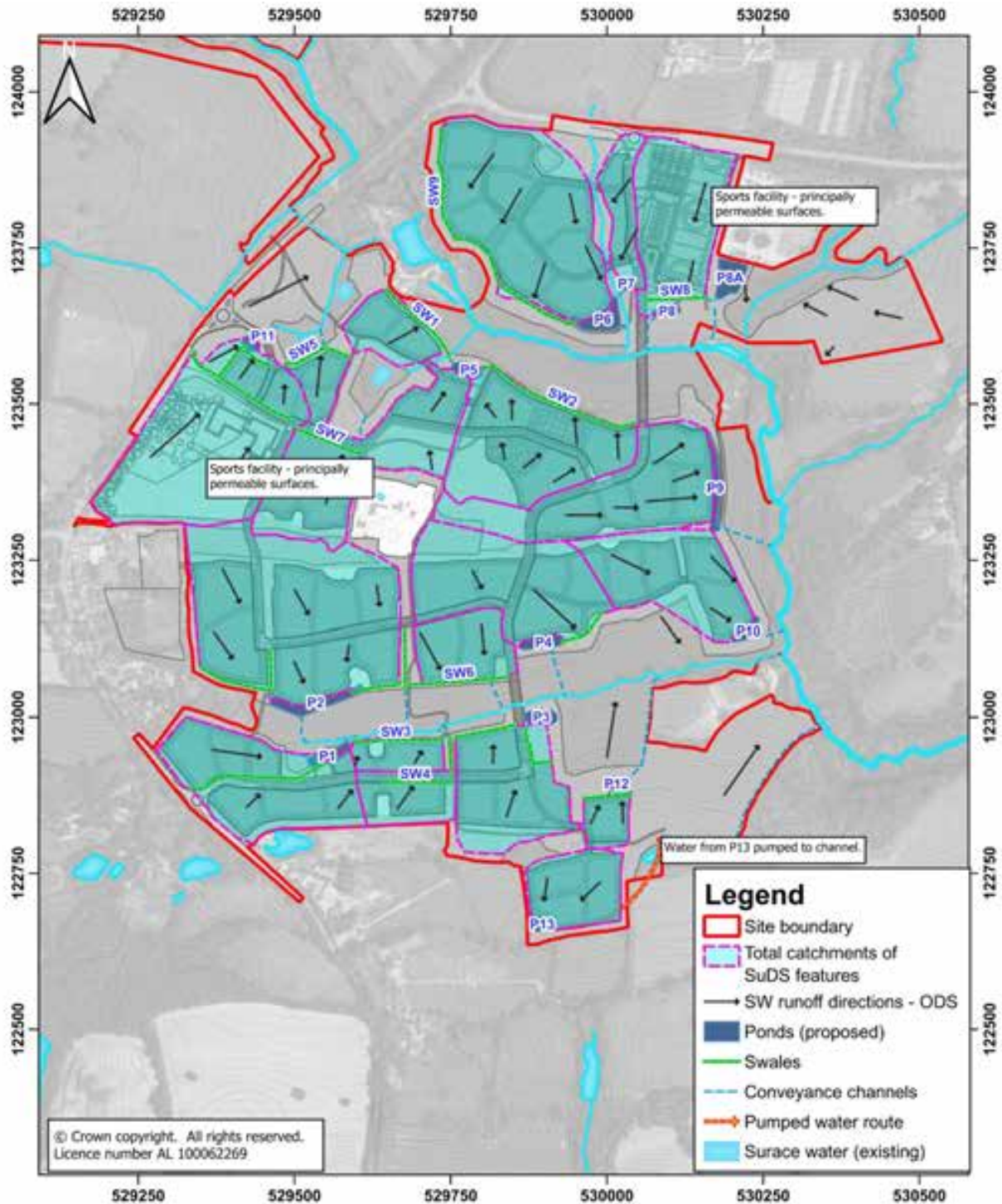


Table 8-4 Impermeable areas assumed for development areas

Housing density (DPH)	% Impermeable
30	50%
32.5	53%
35	55%
40	57%
45	65%
50	70%
80	75%

Table 8-5 Impermeable areas and greenfield rates of post development SuDS catchments

SuDS feature	Total catchment area (ha.)	Development area (ha.)	Mean density (DPH)	% impermeable	Imp. area (ha.)	Peak Q1 flow rate (l/s)	Notes
P1	3.978	3.187	32.5	53%	1.689	17.99	
P2	7.955	5.255	45	65%	3.416	35.97	
P3	3.119	2.512	40	57%	1.432	14.10	
P4	4.262	2.671	45	65%	1.736	19.27	
P5	2.014	1.668	32.5	53%	0.884	9.11	
P6	6.072	5.454	35	55%	3.000	27.45	
P7	1.114	0.619	35	55%	0.340	5.04	
P8A	0	0	n/a	n/a	1.000	0.00	No catchment within development. Nominal imp. Area added for now to represent inflows.
P8	0	n/a	Permeable	25%	0.000	0.00	Receives outflow from SW8 (and upstream catchment).
P9	3.000	3.763	32.5	53%	1.994	13.56	
P10	3.168	2.304	32.5	53%	1.221	14.32	
P11	6.487	0.867	45	65%	0.564	29.33	
P12	0.629	0.49	32.5	53%	0.260	2.84	
P13	1.683	1.539	35	55%	0.846	7.61	
SW1	1.208	0.855	32.5	53%	0.453	5.46	
SW2	4.362	0.742	35	55%	0.408	19.72	Drains onwards to P5.
SW3	0.756	0.648	45	65%	0.421	3.42	
SW4	1.286	0.632	30	50%	0.316	5.81	

SuDS feature	Total catchment area (ha.)	Development area (ha.)	Mean density (DPH)	% impermeable	Imp. area (ha.)	Peak Q1 flow rate (l/s)	Notes
SW5	0.745	0.634	50	70%	0.444	3.37	
SW6	1.558	1.355	45	65%	0.881	7.04	
SW7	0.000	0.000	50	70%	0.000	0.00	Received outflows from perm paving.
SW8	3.39	n/a	Permeable	25%	0.848	15.33	Coverings largely permeable. Nominal imp. area value of 25% used for calculations. Drains to P8 (main flow control for catchment installed on P8 feature).
SW9	0.000		32.5	53%	0.000	0.00	Drains to P9 (main flow control for catchment installed on P9 feature).
Perm paving	2.221	3.488	50	70%	2.442	10.04	Drains to SW7

The Outline SuDS layout is included in Appendix G along with the relevant dimensions of each SuDS feature and other supporting information/ details. Additional details will be added as the design of the site layout progresses. Basins are included with a large freeboard depth at present (which will likely be revised down in future as the scheme is developed). These would have a shallow permanent waterbody at the base for ecological benefit.

The Site is not expected to require land raising/ lowering to accommodate a gravity driven drainage system; there is a suitable gradient over most of the Site to allow for a gravity driven drainage network.

8.5. Performance calculations

The initial design of the principal attenuation SuDS features has been undertaken using the Causeway "Flow+" v 15 industry standard software. Simulations were run for the 1 in 30 year + 40% and the 1 in 100 year + 45% events (allowances given for climate change as defined above) using the FEH22 rainfall input data. The global Time of Entry variable was reduced to 5 minutes and the additional storage variable was set to zero following an additional round of comments from the LLFA in September 2025. The sizes of all SuDS features were revised to accommodate these changes.

Hydrological descriptors for the Site were obtained from the Flood Estimation Handbook (FEH) website (UK Centre for Ecology & Hydrology, 2023). These are shown in Table 8-6 below (and were provided to the LLFA via email as requested in their recent feedback).

Table 8-6 FEH Hydrological descriptors

Catchment Descriptor	Abbreviation	Value
Base Flow Index associated with each HOST soil class	BFIHOST19	0.459
Proportion of time when soil moisture deficit was equal to, or below, 6mm during 1961-90	PROPWET	0.36 (i.e. 36% of the time)
Average Annual Rainfall (1961 – 1990)	SAAR	813 mm

The main attenuation features (basins and swales) within each catchment have been included as storage features within the model. The attenuation capacity provided by other grassed conveyance swales and source control features were not included at the request of the LLFA (previously included as a generalised additional storage capacity). The capacity of the main SuDS features included in the model may be reduced in due course as more SuDS features are included across the Site.

Note that the Site has a fairly steep gradient in places and therefore, some check dams within the swales will be required – perhaps a cascade sequence of smaller features where appropriate). This will maximise the storage capacity available, increase residency time within the features and reduce scour risk. This fine tuning of the scheme can be achieved during the detailed design phase and for now, the swales have been modelled as single features with a single invert/ bank levels to demonstrate the broad general feasibility of the scheme (i.e. that there is sufficient space to provide the required attenuation prior to discharging to a watercourse at the greenfield Q1 rate).

The infiltration rate for all features was set to zero, based on the results of the on-Site test results (Section 3.4.4).

Hydro-brake® flow control features have been included to limit the outflow from the various SuDS features. These may be substituted for alternative features at the detailed design phase if appropriate.

Appendix G contains the output from the drainage simulations. This confirms that, based on the parameters described above, the proposed drainage scheme will be able to attenuate and discharge all runoff generated during the 1 in 30-year storm + 40% event of the 1 in 100-year storm +45% event (following upsizing to remove any flooding following feedback from the LLFA). The sum of the peak outflow rates and total volumes from the SuDS features are presented in Table 8-7, along with the greenfield equivalent value and the calculated change for each scenario (a reduction in each case).

Table 8-7 Summary of drainage simulation results

Storm return period (yrs.)	Greenfield runoff rate (l/s)	Post-dev discharge rate (l/s)	Change (l/s)	Greenfield runoff volume (m ³)	Post-dev discharge vol (m ³)	Change (m ³)
1	460	226.1	-233.9	11,844	4,886	-6,958
30	1298.9	261.2	-1037.7	25,596	8,518	-17,078
100	1726.4	261.3	-1465.1	33,033	10,852	-22,181

A freeboard depth of at least 300 mm is retained in all SuDS features under the design storm scenario (T30 +CCA) and no flooding is calculated under the T100+CCA event from any feature. Details of exceedance flow routes (under a very extreme scenario) are provided in Section 8.6. The SuDS features have been positioned in the downgradient part of the development areas in each case. Suitable overflow features will be installed within the banks of the features to control overflows under very extreme events.

As the detailed layout plan evolves, it will be possible to include further SuDS techniques within the development layout in order to enhance the 'SuDS Management Train'. Techniques such as rainwater capture and re-use and bio-retention areas will be considered during the development of the detailed layout to maximise water efficiency, water quality, biodiversity, health and wellbeing, and amenity benefits.

8.5.1. Pump failure mitigation

As part of the Outline SuDS layout (Appendix G), it is proposed to install a pump to transfer flows from the southern section of the Site (P13) to a north-eastern conveyance channel with a controlled peak rate of 7.5 l/s. Pumping is required as, due to the local topography and surface runoff directions, no valid receptor for this part of the Site was identified and infiltration was deemed unviable (see Section 3.4.4). Pump failure analysis is provided in Technical Note 3 (Yellow Sub Geo, 2025).

The following measures are recommended to mitigate the risk of pump failure:

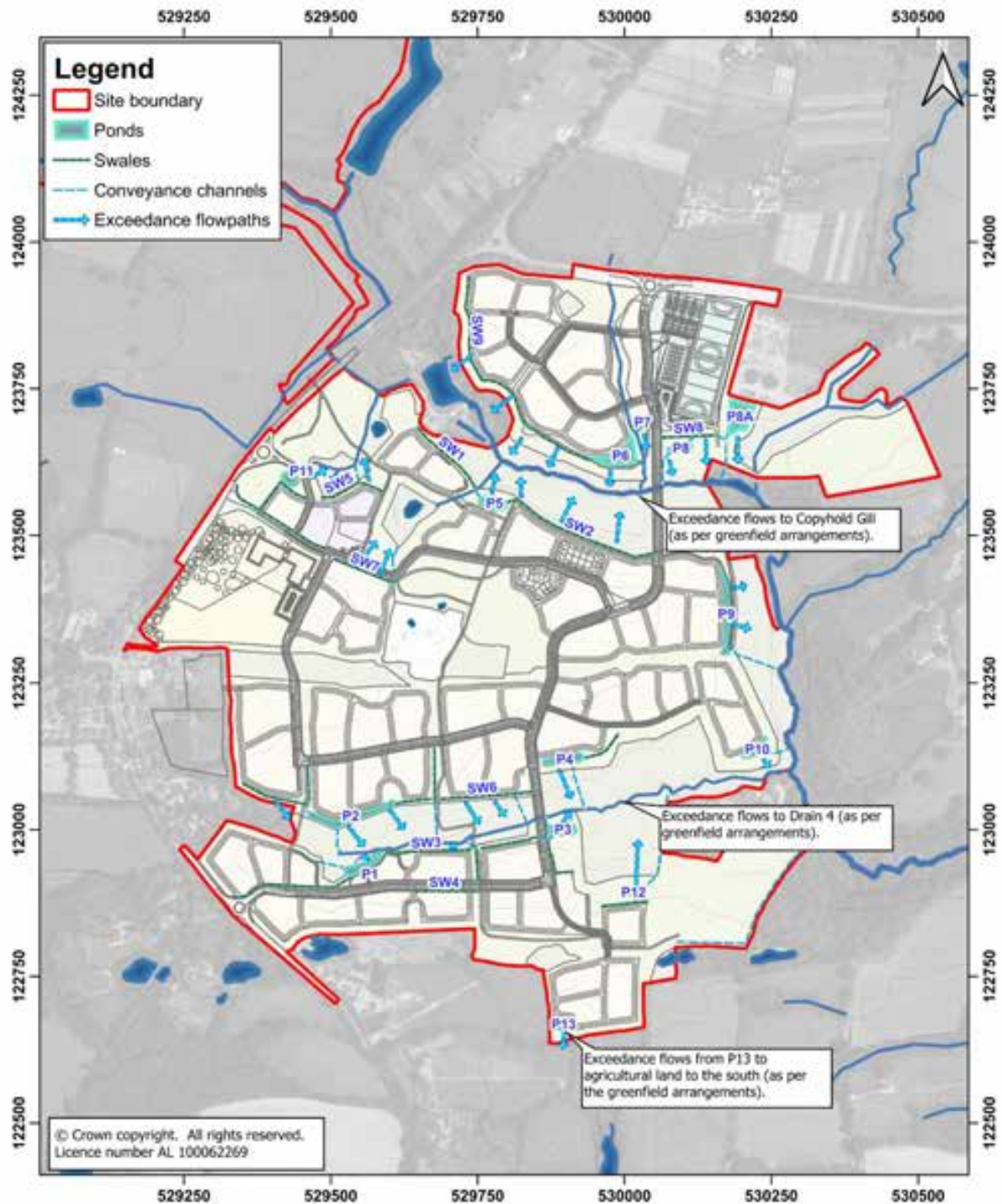
- **Routine maintenance:** Regular inspection, testing, and servicing of the pumps will form part of the overall SuDS maintenance requirements for the site (see Table 7.9).
- **Full redundancy:** A complete back-up system (including pump, inlet, outlet, and pipework) will be installed to ensure continuity of operation in the event of primary pump failure.
- **Automatic alarms:** The pump system should be fitted with automatic alarms to provide immediate notification of pump failure or abnormal operating conditions.
- **Emergency procedure:** In the event of a prolonged outage (>24 hrs), arrangements should be in place for temporary water removal by tanker to prevent exceedance of on-site storage capacity.

8.6. Exceedance routes

Flooding of some features is expected under the extreme 1 in 100 year +45% scenario (as shown in Appendix H). Under these extreme events, exceedance flows from the SuDS features will be designed to follow the existing preferential surface water flow pathways towards Copyhold Gill and its tributaries (or to the south in the instance of Pond 13). This is illustrated in Figure 8-2. A more detailed

analysis of exceedance flows can be undertaken once the drainage network has been finalised and modelled.

Figure 8-2 Exceedance flow routes from SuDS features



8.7. Water quality

SuDS techniques can be used to effectively manage the quality of surface water flowing across a site. Different methods can be used to intercept pollutants and allow them to degrade or be stored in-situ without impacting the quality of water further downstream. Frequent and short duration rainfall events are those that are most loaded with potential contaminants (silts, fines, heavy metals and various organic and inorganic contaminants). Therefore, the first 5mm to 10 mm of rainfall (i.e. the 'first flush') should be adequately treated using SuDS.

The proposed development will include residential dwellings, low traffic roads and driveways. The CIRIA SuDS manual categorises runoff from residential dwellings as presenting a very low water quality hazard and runoff from low usage roads and residential driveways as presenting a low hazard rating (see Table 8-8).

Table 8-8 Water quality hazard ratings (CIRIA, 2015)

Land use	Hazard level
Residential Roof drainage	Very Low
Residential, amenity uses including low usage car parking spaces and roads, other roof drainage.	Low
Commercial uses including car parking spaces and roads (excluding low usage roads, trunk roads and motorways).	Medium
Sites with heavy pollution (e.g. haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemical and fuels (other than domestic fuel oil) are delivered, handled, stored used or manufactured, industrial sites.	High
Trunk roads and motorways	High

The CIRIA SuDS manual (CIRIA, 2015) advocates a qualitative approach to designing a SuDS scheme for a site with a low hazard rating. This should provide adequate controls on pollutants contained in runoff water.

As the proposed development is residential in nature with a low hazard rating, hazard indices of 0.5 for Total Suspended Solids (TSS), 0.4 for Metals and 0.4 for Hydrocarbons are considered applicable.

The following measures in Table 8-9, are examples which are suitable for inclusion in a drainage strategy for a residential development to mitigate a potential increase in pollutant load within on-site and off-site runoff – note text in bold are measures included in this SuDS Strategy. Removal indices are included for each feature type relative to the specific pollutant.

Table 8-9 Mitigation indices for SuDS components

Component Type	TSS	Metals	Hydrocarbons
Filter drain	0.4	0.4	0.4
Swale	0.5	0.6	0.6
Permeable paving	0.7	0.6	0.7
Detention basin	0.5	0.5	0.6
Pond	0.7	0.7	0.5

The inclusion of swales and ponds within the SuDS Strategy for the post-development catchments will provide adequate treatment to mitigate the low hazard associated with runoff from the development.

Additional SuDS measures (such as source control features) will be included in the layout as the design progresses. These features will add to a SuDS train for the management of runoff from each development area catchment.

Sediment traps (i.e. sumps within the inspection chambers of the final manhole upstream of each feature) will be used to facilitate the maintenance of these SuDS features and reduce the build-up of potentially polluted material.

All runoff from roads will pass through at least one water treatment feature prior to discharging to a watercourse (to be included at the detailed design phase).

8.8. SuDS maintenance

Inspection and long-term maintenance of SuDS components ensures efficient operation and prevents failure. Surface SuDS components can be managed using landscape maintenance techniques. Table 8-10 describes the management and maintenance requirements for the SuDS features included. These requirements will be implemented following the completion of the proposed development, and will be undertaken either by the Lead Local Flood Authority, a private management company or by the local water company, subject to discussions regarding this responsibility.

Table 8-10 Management and maintenance requirements for SuDS features

SuDS Device	Maintenance requirements	Maintenance frequency
Swales	- Litter/ trash removal - Cut grass - Inlet/ outlet cleaning - Sediment monitoring and silt removal.	- Monthly - Monthly in summer - Quarterly - Annually or every three years
Ponds	- Litter/ trash removal - Inlet/ outlet cleaning - Vegetation management - Sediment monitoring and silt removal.	- Monthly - Quarterly - Quarterly - Annually or every 3 yrs.
Pump	- Visual inspection and controls check - Automatic alarm system test - Silt/ debris removal from pump chamber - Pump, seals, and electrical connections servicing	- Monthly - Quarterly - Quarterly - Annually

8.9. Further SuDS considerations

As the layout plan evolves, it will be possible to include further SuDS techniques within the development layout in order to improve the 'SuDS Management Train'. Techniques such as rainwater capture and re-use, and bio-retention areas will be considered during the development of the detailed layout plan to maximise water efficiency, water quality, biodiversity, health and wellbeing, and amenity benefits.

8.10. Biodiversity and amenity

SuDS schemes present opportunities to enhance habitat for wildlife on-site and this often improves the biodiversity of the surrounding areas. Ponds, constructed wetlands and other surface water features are landscape assets that have amenity value and improve the aesthetics of a site more than conventional drainage systems. The use of grassed swales and ponds (with mature vegetation around the banks) will enhance the biodiversity and amenity value of the Site post-development. Ecological diversity should be enhanced by the use of native planting within each feature.

Surface water flowpaths will be preserved along with areas of open green space to provide naturalised zones within the Site boundary which will provide amenity and ecology benefits.

9. Conclusions

The Site is primarily within Flood Zone 1, with a linear area of Zone 3 present along Copyhold Gill in the north of the Site and Drain 4 in the south of the Site. The proposed development areas are all within Flood Zone 1. As noted in Section 5, there is a low risk of flooding from fluvial, surface water, groundwater, sewer and catastrophic sources to the proposed development areas.

The proposed development will be residential in nature, with a vulnerability classification of 'More Vulnerable' with regards to flood risk. All types of development are permissible within Flood Zone 1.

Given the apparent low flood risk present at the Site, no specific mitigation measures are proposed other than the implementation of a Sustainable Drainage Strategy to mitigate any potential increases in off-site flood risk and the suitable design of infrastructure which intersects water features and surface water flowpaths.

This report provides an Outline SuDS Strategy for the Site. The principal components of the scheme are ponds and large grassed swales located downgradient of the development areas. The details of the drainage network will evolve along with the layout plans, but the key findings of the present strategy and calculations are that there is a suitable means for water disposal in the area and there is ample space for the required attenuation and treatment of runoff prior to its disposal.

A preliminary assessment of the performance of the initial SuDS system under the 1 in 30 year + 40% climate change storm was undertaken and shows that the proposed SuDS are capable of accommodating the required stormwater runoff volumes.

Appropriate management and maintenance arrangements for the proposed SuDS scheme will be in place throughout the lifetime of the proposed development.

10. References

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Appendix A Report conditions

Report Conditions

This report has been prepared by Aqua Terra Consultants Ltd. (Aqua Terra) in its professional capacity as soil and groundwater specialists, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client and is provided by Aqua Terra solely for the internal use of its client.

The advice and opinions in this report should be read and relied on only in the context of the report, taking account of the terms of reference agreed with the client. The findings are based on the information made available to Aqua Terra at the date of the report (and will have been assumed to be correct) and on current UK standards, codes, technology, and practices as at that time. They do not purport to include any manner of legal advice or opinion. New information or changes in conditions and regulatory requirements may occur in future, which will change the conclusions presented here.

Where necessary and appropriate, the report represents and relies on published information from third party, publicly and commercially available sources which is used in good faith of its accuracy and efficacy. Aqua Terra cannot accept responsibility for the work of others.

Site investigation results necessarily rely on tests and observations within exploratory holes only. The inherent variation in ground conditions mean that the results may not be representative of ground conditions between exploratory holes. Aqua Terra take no responsibility for variation in ground conditions between exploratory positions.

This report is confidential to the client. The client may submit the report to regulatory bodies, where appropriate. Should the client wish to release this report to any other third party for that party's reliance, Aqua Terra may, by prior written agreement, agree to such release, if it is acknowledged that Aqua Terra accepts no responsibility of any nature to any third party to whom this report or any part thereof is made known. Aqua Terra accepts no responsibility for any loss or damage incurred as a result, and the third party does not acquire any rights whatsoever, contractual, or otherwise, against Aqua Terra except as expressly agreed with Aqua Terra in writing. Aqua Terra reserves the right to withhold and/ or negotiate the transference of reliance on this report, subject to legal and commercial review.

Appendix C Site development plans



- LEGEND
- ANSTY GARDEN COMMUNITY BOUNDARY
 - PARKLAND RESERVE BOUNDARY
 - DEVELOPMENT BLOCK
 - ANCIENT WOODLAND
 - EXISTING WOODLAND
 - PROPOSED TREES AND WOODLAND
 - PROPOSED SUDS
 - SITE ACCESS
 - TREE-LINED SPIKE STREET
 - TREE-LINED SECONDARY LOOP
 - TERTIARY STREET
 - FEATURE NODE WITH BUS STOP
 - LOCAL CENTRE
 - RETIREMENT LIVING/CARE HOME
 - 3FE PRIMARY & SEND SCHOOLS
 - EXISTING ACCESS TRACK TO MACKERELL'S FARM COTTAGE RETAINED
 - RETAINED WOODLAND WITH BUFFER
 - RETAINED ANCIENT WOODLAND WITH BUFFER
 - RETAINED HEDGEROW
 - PARKLAND RESERVE
 - RETAINED PROW
 - NEW FOOT/CYCLE LINK
 - SPORTS FACILITY, INCLUDING HOCKEY, OUTDOOR TENNIS, INDOOR TENNIS AND PADL COURTS
 - PUBLIC OPEN SPACE
 - FORMAL CHILDREN'S PLAY
 - ALLOTMENTS
 - BRIDGE ACROSS WOODED VALLEY
 - MOBILITY HUB

10	Site boundary updated to incorporate new access	ID	24.05.25
09	Updated eastern access location	ID	24.05.25
08	Updated footpaths	ID	17.10.24
07	Updated footpaths	ID	15.10.24
06	Updated access and minor amendments	ID	06.10.24
05	Parkland reserve in blue boundary	ID	06.09.24
04	Update to sports facility, northern blocks and northern bus stops	ID	14.05.24
03	Updates to bus stops	ID	15.04.24
02	Update to sports centre	ID	05.04.24
01	Update to street widths and layout	ID	31.03.24
REV.	DESCRIPTION	APP.	DATE

fabrik

PROJECT TITLE
LAND AT ANSTY, HAYWARDS HEATH

DRAWN TITLE
CONCEPT MASTERPLAN

ISSUED BY
London

DATE
MAY 2025

SCALE
1:2,500

STATUS
FINAL

DWG. NO.
D3012-FAB-00-XX-DR-Y-009

T: 020 7620 1453

DRAWN
ID

CHECKED
ID

APPROVED
BS

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Appendix D Soakaway testing report



Ansty Garden Community: Soakaway Testing

P21367_R4
August 2023





Document Control

Title

Ansty Garden Community: Soakaway Testing

Client

Fairfax Acquisitions Ltd.
Buncton Barn,
Buncton Lane,
Bolney,
West Sussex,
RH17 5RE



Reference

P21367_R4

Status

Final

Document Reference	Issue Date	Comments	Written by	Approved by
P21367_R4	August 2023	Final	RLW	JEM



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P21367_R4_D02 Exploratory hole location plan



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Appendix B: Service plans

Appendix C: Engineering logs

Appendix D: Infiltration test results

Appendix E: Photos



1. Introduction

1.1. Instruction

Yellow Sub Geo Ltd (Yellow Sub) was instructed by Fairfax Acquisitions Ltd (the Client) to undertake infiltration testing for a parcel of land at Ansty Farm, Haywards Heath (the Site). Instruction to proceed in accordance with Yellow Sub proposal (Ref: P21367_P4) was confirmed by email dated 8th November 2022.

1.2. Site location

The Site is located to the east of Ansty Village in the District of Mid Sussex. A Site location plan is presented as drawing P21367_R4_D01. The Site address is as follows:

Ansty Farm,
Haywards Heath,
West Sussex,
RH17 5AG

The National Grid Reference (NGR) for the approximate centre of the Site is TQ 29653 23438. The Site covers a total area of c. 99 ha.

1.3. Brief

The Client is seeking to obtain outline planning permission for a proposed residential development. To support the application a sustainable drainage strategy is required which this report supports by demonstrating whether infiltration drainage may be viable at the Site.

1.4. Scope

The scope of work was to undertake infiltration testing across the Site in accordance with BRE365 (2016), to inform the proposed development drainage strategy. Yellow Sub have also been commissioned to prepare a flood risk assessment and sustainable drainage strategy which the data herein informs and is presented separately (see P21367_R5).

1.5. Limitations

This report is written strictly for the benefit of the Client and bound by the conditions presented in Appendix A.



2. Fieldwork

Fieldwork was undertaken over 4No. days between the 26th and 29th June 2023, with the location of each exploratory hole agreed with the Client and designed to minimise Site damage whilst providing suitable coverage. Drawing P21367_R4_DO2 details the locations of the exploratory holes, and the ground conditions encountered are discussed in Section 3. The works undertaken are summarised in the following table:

Table 2-1 Fieldwork undertaken

Work element	Comments/ rationale
Utilities and service clearance	Prior to the Site works a utilities report was obtained and each position was checked for services by trained and competent Yellow Sub staff using the plans (see Appendix B) and non-intrusive CAT and Genny techniques.
Trial pits	Twelve trial pit locations (TPO1, TPO2A, TPO2B, TPO3, TPO4A, TPO4B, TPO4C, TPO4D, TPO4E, TPO5, TPO8, TP10) were machine excavated (using a JCB) to depths of between 1.6m and 3m below ground level (m bgl). TPO3, TPO4A, TPO4B, TPO4C, TPO4D and TPO4E were all terminated within Upper Tunbridge Wells Sand. TPO1, TPO2A, TPO2B, TPO5 and TP10 all terminate within the Cuckfield Stone Bed and TPO8 within the Lower/Upper Grinstead Clay. In all trial pits, the faces and base were cut to create as square a shape as possible, the dimensions of each pit were then measured and recorded
Logging of strata	All strata were logged by competent Yellow Sub staff in accordance with BS5930. The engineering logs are presented in Appendix C.
Soakaway testing	Soakaway testing in general accordance with BRE 365 (2016) was carried out in each trial pit using a 1,000 litre IBC, which rapidly inundated the trial pits to the required depth. Water levels were measured at regular intervals across the test period. If tests were successful, repeat tests were run in the same saturated pit as detailed by BRE365. The results of this testing are presented in Appendix D.
Installations	None of the exploratory positions were installed.
Backfill	On completion of excavation and testing, the pits were backfilled with arisings, which were compacted in discrete layers by the JCB in reverse order to excavation. Excess materials were carefully mounded on the backfilled pits to accommodate future settlement.



3. Ground conditions

3.1. Strata Encountered

The strata encountered during the fieldwork are summarised as follows:

- **Topsoil:** Typically 0.1m to 0.5m thick. Encountered as grass covered yellowish brown to brown, sandy CLAY with an occasional sandstone gravel.
- **Upper Tunbridge Wells Sand:** Encountered in TPO3, TPO4A, TPO4B, TPO4C, TPO4D and TPO4E between 0.15 to 2.7m bgl. Typically encountered as a orangish brown to reddish brown slightly sandy CLAY but also as a clayey SAND with TPO4B. Within TPO4A encountered as a yellow medium grained SANDSTONE between 1.5 and 1.6m bgl.
- **Cuckfield Stone Bed:** encountered in TPO1, TPO2A, TPO2B, TPO5 and TP10 between 0.4 and 2.8m bgl. Typically recorded a sandy gravelly CLAY but also encountered as a very clayey, sandy GRAVEL within TPO5 and as a very clayey SAND in TP10. Gravel is typically medium to coarse of iron rich calcareous sandstone concretions.
- **Lower/Upper Grinstead Clay:** Encountered only in TPO8 between 0.1 and 0.4m bgl as a light brown, very clayey SAND and between 0.4 to 3m bgl to as a brown, mottled grey and orange CLAY.

Exploratory logs of each position can be found in Appendix C and a photographic log is shown in Appendix E.

3.2. Progress and Obstructions

No obstructions were encountered in any of the exploratory holes.

3.3. Stability

All pits remained open and stable for the duration of infiltration testing.

3.4. Visual and olfactory evidence of contamination

There was no evidence of significant or gross impact to soils encountered within exploratory positions. In addition, no Made Ground was encountered in any of the locations.

3.5. Groundwater

No seepages or groundwater strikes were noted in any of the exploratory positions.

3.6. Summary of infiltration testing results

A summary of the infiltration testing is presented in Table 3-1 below with full results presented in Appendix D. The infiltration values for each pit have been calculated in accordance with the methodology set out in BRE 365.



Ansty Garden Community: Soakaway Testing
P21367_R4

Table 3-1 Summary of infiltration testing

Trial pit	Test	Depth of pit (m)	Geology of test section	Permeability (m/s)
TP01	1	2.8	Cuckfield Stone Bed	Failed
TP02A	1	2.5	Cuckfield Stone Bed	Failed
TP02B	1	2.9	Cuckfield Stone Bed	Failed
TP03	1	2.5	Upper Tunbridge Wells Sand	Failed
TP04A	1	1.6	Upper Tunbridge Wells Sand	Failed
TP04B	1	2.4	Upper Tunbridge Wells Sand	Failed
TP04C	1	2.5	Upper Tunbridge Wells Sand	Failed
TP04D	1	2.7	Upper Tunbridge Wells Sand	Failed
TP04E	1	1.5	Upper Tunbridge Wells Sand	Failed
TP05	1	1.9	Cuckfield Stone Bed	Failed
TP08	1	2.7	Lower/ Upper Grinstead Clay	Failed
TP10	1	2.8	Cuckfield Stone Bed	Failed

All twelve of the infiltration tests failed to drain sufficiently with no repeat tests undertaken. This indicates that infiltration drainage is unlikely to be feasible at the Site.



4. Conclusions

4.1. Ground conditions encountered

The trial pitting confirmed the published geology at the Site, consisting of up to 0.5m of topsoil underlain by the Cuckfield Stone Bed, Upper Tunbridge Wells Sand and Lower/ Upper Grinstead Clay to the maximum excavated depth of 3m bgl.

No groundwater seepages were noted within the excavations. No visual or olfactory evidence of gross contamination was noted within the excavations and no Made Ground was encountered.

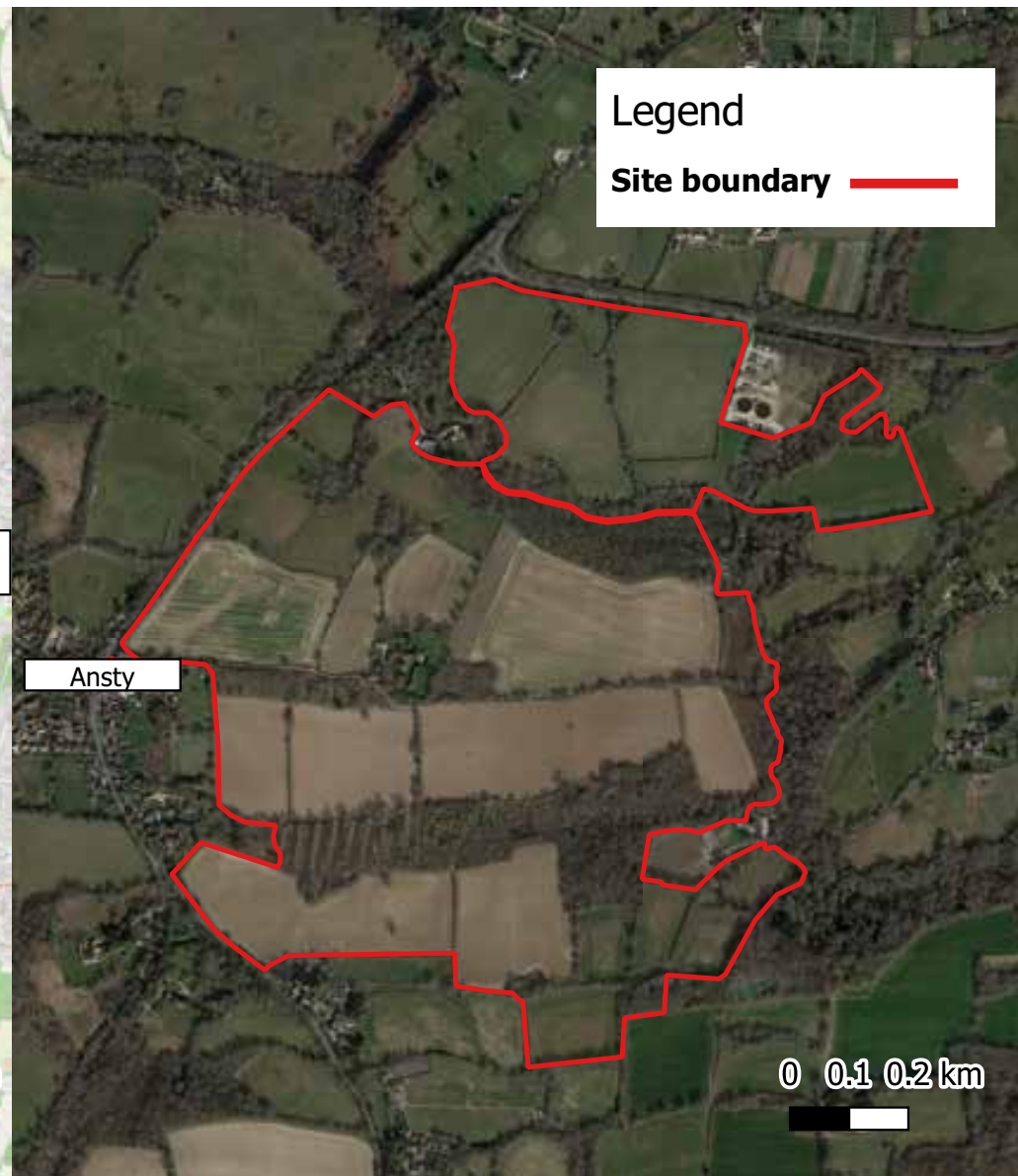
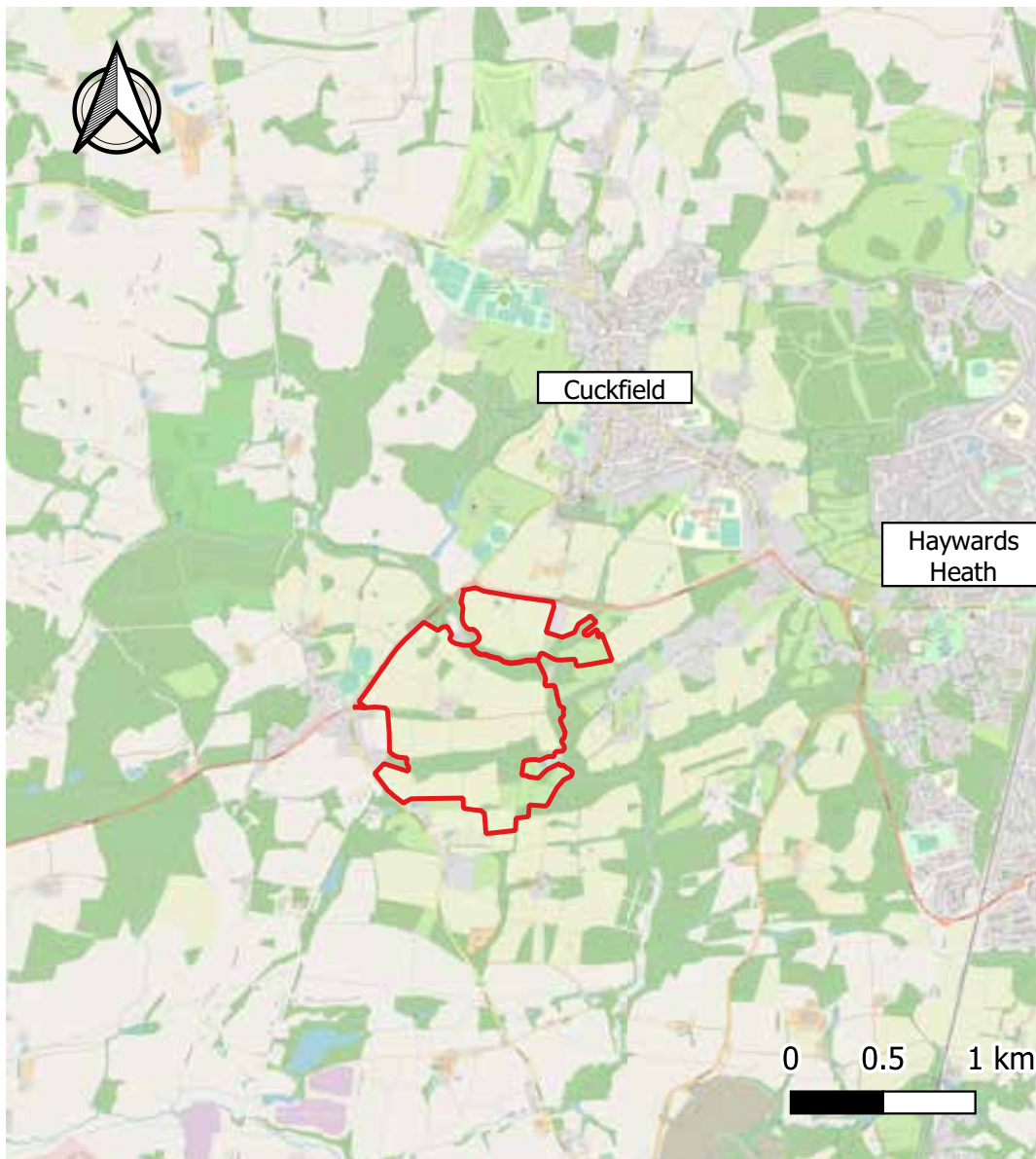
4.2. Infiltration drainage potential

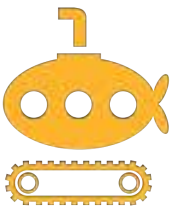
Infiltration rates have been calculated for each trial pit. None of the positions drained away within sufficient timescales. As a result, it is possible to conclude that that soakaway drainage is unlikely to be suitable for the Site.



Drawings





	Figure Title Site location, Ansty Farm	Client	Fairfax Properties Ltd	Date	24/10/2022	Drawn	RLW
		Drawing Number	P21367_R4_D01	Scale	NTS	Checked	JEM
		Project Number	21367	Original	A4	Revision 1	
		Ansty Farm, Mid Sussex					

YELLOW
SUB
GEO



Figure Title

Exploratory positions, Ansty Farm

Client

Fairfax Properties Ltd

Drawing Number

P21367_R4_D02

Project Number

P21367

Date

05/10/23

Scale

1:10,000

Drawn

RLW

Site Location

Ansty Farm, Mid Sussex

Original

A4

Checked

JEM

YELLOW
SUB
GEO





Appendix A Report conditions





Report Conditions

This report has been prepared by Yellow Sub Geo Ltd. (Yellow Sub Geo) in its professional capacity as soil and groundwater specialists, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client and is provided by Yellow Sub Geo solely for the internal use of its client.

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Appendix B Service Plans





Appendix C : Engineering Logs



Trial Pit Log

TP01
Page 1 of 1

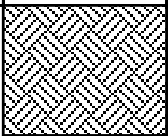
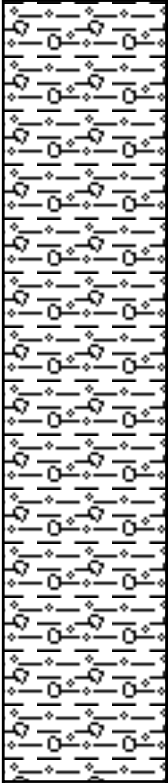
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 27/6/23 - 27/6/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.6 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.40		TOPSOIL: Grass over brown sandy CLAY. Sand is fine to medium. Abundant roots and rootlets		
		2.80		Brown mottled light grey and orange sandy gravelly CLAY. Gravel is medium to coarse sub-angular of iron rich calcareous sandstone concretions	@ 1.5m becoming cobbly of iron rich concretions up to 15cm long axis @ 2.4 becoming very gravelly	0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5

Position cleared with CAT and progressed to 3m bgl whereupon soakaway testing was performed. Position backfilled with arisings.

Trial Pit Log

TP02A
Page 1 of 1

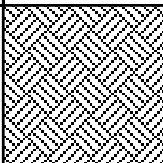
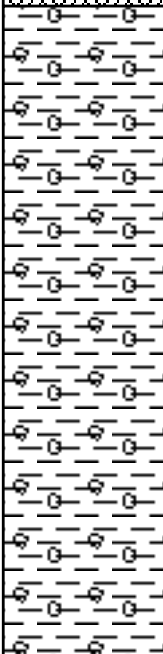
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 27/6/23 - 27/6/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 2 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.50		TOPSOIL: Grass over brown slightly sandy slightly gravelly CLAY sand is fine to medium gravel is coarse sub-angular to angular of sandstone		0.5
		2.50		Brown mottled orange slightly gravelly CLAY gravel is medium to coarse sub-angular of sandstone	@1.0m low cobble content of sandstone @ 1.6m becoming blue grey mottled brown @ 2.0m occasional rootlets, dead decaying	2.5
						3.0
						3.5
						4.0
						4.5

Position cleared with CAT and progressed to 3m bgl whereupon soakaway testing was performed. Position backfilled with arisings.

Trial Pit Log

TP02B
Page 1 of 1

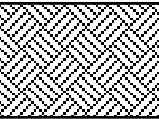
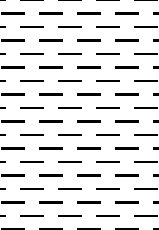
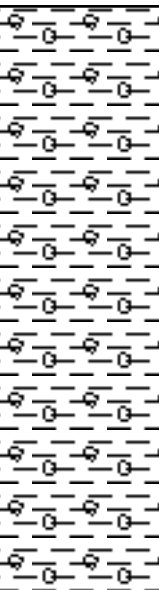
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 27/6/23 - 27/6/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 2 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.35		TOPSOIL: Grass over brown sandy CLAY sand is fine		0.5
		1.10		Yellowish brown mottled light blue grey CLAY		1.0
		2.90		Blue grey mottle orange and ochre sandy, gravelly CLAY with medium cobble content. Gravel is fine to coarse sub-angular of sandstone, sand is fine to coarse		1.5
						2.0
						2.5
						3.0
						3.5
						4.0
						4.5

Position cleared with CAT and progressed to 3m bgl whereupon soakaway testing was performed. Position backfilled with arisings.

Trial Pit Log

TP03
Page 1 of 1

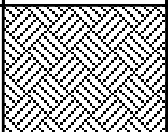
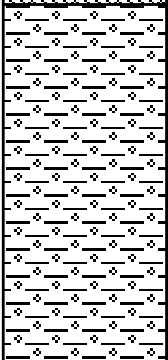
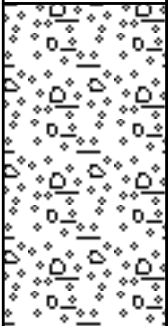
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 27/6/23 - 27/6/23

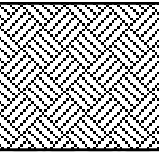
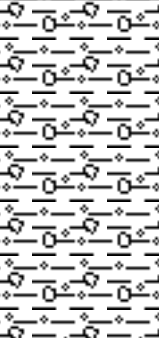
Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.6 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.40		TOPSOIL: Grass over brown slightly sandy CLAY. Sand is fine to medium. Abundant rootlets		
		1.50		Orangeish brown slightly sandy CLAY sand is fine		0.5 1.0
		2.50		Yellowish brown occasionally brown slightly clayey slightly gravelly fine to medium SAND with a low cobble content up to 10cm long axis of lithic fragments. Gravel is fine to coarse sub-angular of lithic fragments of sandstone	@2.0m becoming very gravelly with a high cobble content	1.5 2.0 2.5 3.0 3.5 4.0 4.5

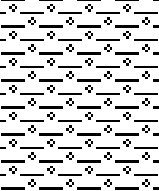
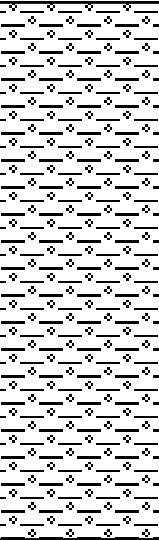
Position cleared with CAT and progressed to 3m bgl whereupon soakaway testing was performed. Position backfilled with arisings.

Project Name: Ansty Farm Project No: P21367
 Location: Haywards Heath, Sussex Co-ords: - Level:
 Hole Type: TP Logged By: JRB Dates: 28/06/23 - 28/06/23
 Client: Fairfax Acquisitions Ltd Consultant: JRB
 Plant Used: Wheeled Excavator Length: 1.9 Width: 0.5

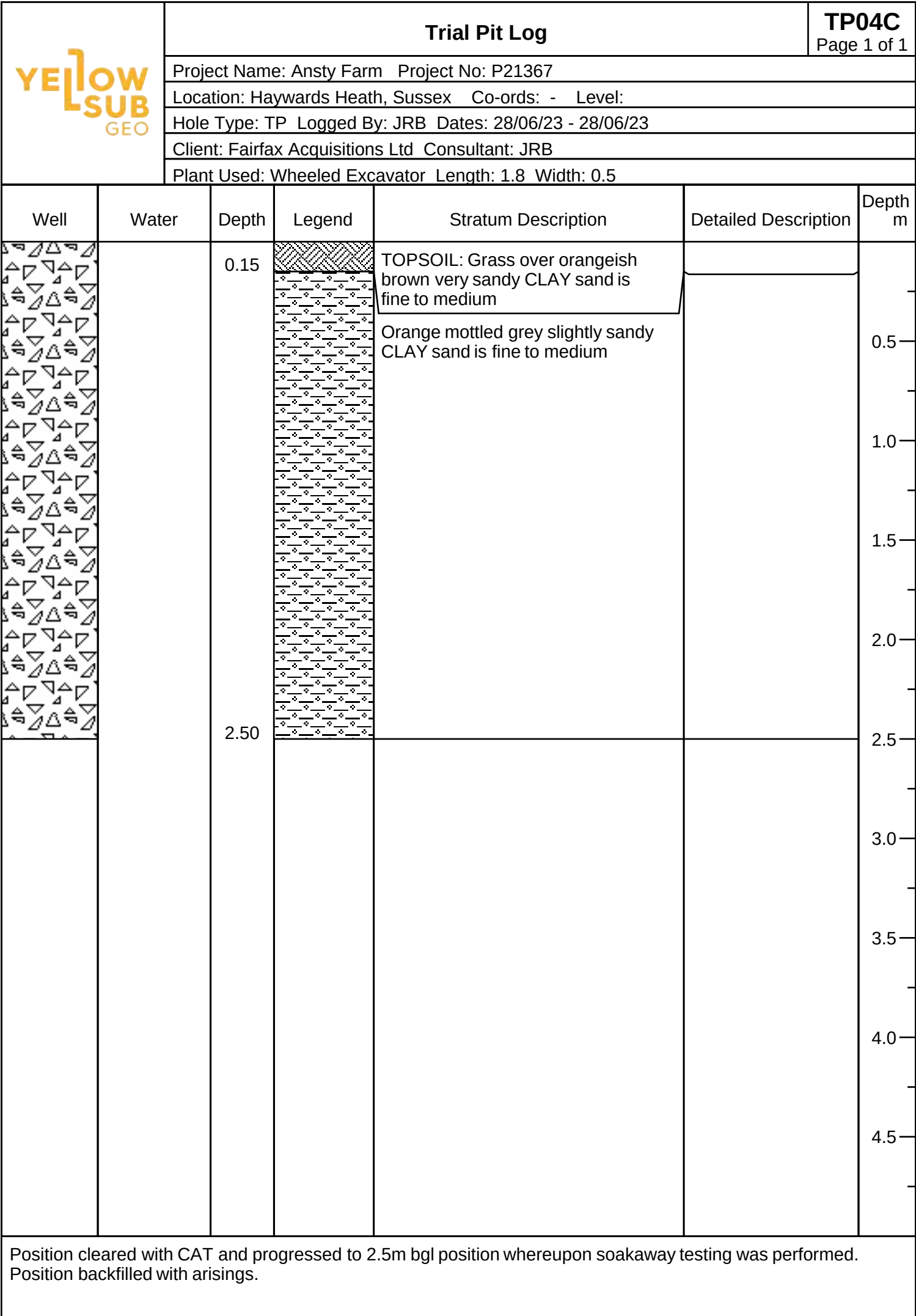
Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.45		TOPSOIL: Grass over orangeish brown very sandy CLAY sand is fine to medium		0.5
		1.50		Light brown mottled grey slightly sandy and slightly gravelly CLAY sand is fine to coarse gravel is fine to coarse sub-angular of iron rich concretions	@1.5m becoming gravelly	1.0
		1.60		Yellow medium grained SANDSTONE recovered as yellow orange gravelly sand		1.5
						2.0
						2.5
						3.0
						3.5
						4.0
						4.5

Position cleared with CAT and progressed to 1.6m bgl position terminated on bedrock whereupon soakaway testing was performed. Position backfilled with arisings.

Project Name: Ansty Farm Project No: P21367
 Location: Haywards Heath, Sussex Co-ords: - Level:
 Hole Type: TP Logged By: JRB Dates: 28/06/23 - 28/06/23
 Client: Fairfax Acquisitions Ltd Consultant: JRB
 Plant Used: Wheeled Excavator Length: 1.7 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.15		TOPSOIL: Grass over orangeish brown very sandy CLAY sand is fine to medium		
		0.75		Reddish brown slightly sandy CLAY sand is fine to medium		0.5
		2.40		Yellow, grey and orange clayey fine to medium predominantly fine SAND. Occasional pockets of (<10cm) of very sandy clay		1.0 1.5 2.0
						2.5 3.0 3.5 4.0 4.5

Position cleared with CAT and progressed to 2.4m bgl position whereupon soakaway testing was performed. Position backfilled with arisings.





Trial Pit Log

TP04D

Page 1 of 1

Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 28/06/23 - 28/06/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.9 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.40		TOPSOIL: Grass over brown slightly sandy slightly gravelly CLAY sand is fine to medium gravel is coarse sub-angular to angular of sandstone		0.5
				Orangeish brown mottled light grey CLAY rare coarse gravel sized iron rich concretions		1.0
		2.70				1.5
						2.0
						2.5
						3.0
						3.5
						4.0
						4.5

Position cleared with CAT and progressed to 2.7m bgl position whereupon soakaway testing was performed.
Position backfilled with arisings.

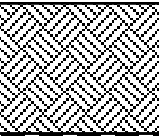
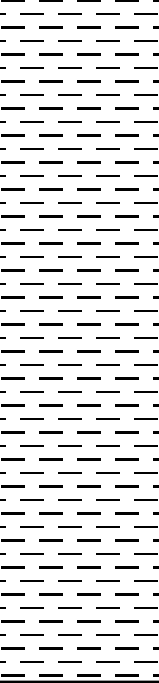
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 29/06/23 - 29/06/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.9 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.40		TOPSOIL: Grass over brown sandy CLAY sand is fine		
				Light brownish orange mottled light blue grey CLAY friable		0.5
		2.50				1.0
						1.5
						2.0
						2.5
						3.0
						3.5
						4.0
						4.5

Position cleared with CAT and progressed to 2.5m bgl position whereupon soakaway testing was performed.
Position backfilled with arisings.

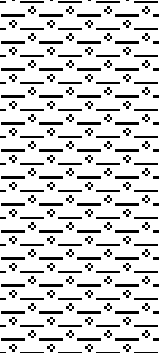
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 29/06/23 - 29/06/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.2 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.10		TOPSOIL: Grass over orangeish brown very sandy CLAY sand is fine to medium abundant rootlets		
		1.20		Reddish brown slightly sandy CLAY with rare pockets <15cm of fine gravel of sub-angular sandstone		0.5 1.0
		1.90		Grey very clayey sandy fine to coarse angular GRAVEL of sandstone sand is fine to coarse		1.5 2.0 2.5 3.0 3.5 4.0 4.5

Position cleared with CAT and progressed to 1.9m bgl position whereupon soakaway testing was performed. Position backfilled with arisings.

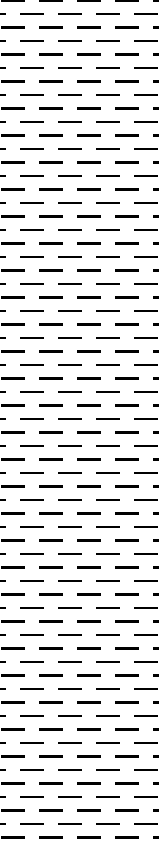
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 26/6/23 - 26/6/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.6 Width: 0.6

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.10		TOPSOIL: Grass over yellowish brown sandy CLAY. Sand is fine to medium. Abundant rootlets		
		0.40		Light brown very clayey fine to medium SAND with occasional rootlets		0.5
		3.00		Brown mottled grey and orange CLAY occasional relic rootlet casts		1.0 1.5 2.0 2.5 3.0
						3.5 4.0 4.5

Position cleared with CAT and progressed to 3m bgl whereupon soakaway testing was performed. Position backfilled with arisings.

Trial Pit Log

TP10
Page 1 of 1

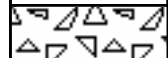
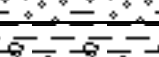
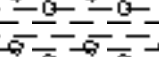
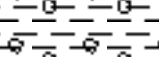
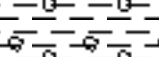
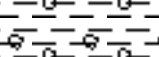
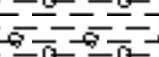
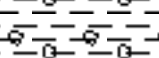
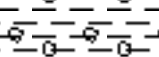
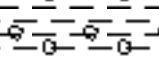
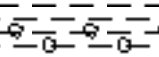
Project Name: Ansty Farm Project No: P21367

Location: Haywards Heath, Sussex Co-ords: - Level:

Hole Type: TP Logged By: JRB Dates: 26/6/23 - 26/6/23

Client: Fairfax Acquisitions Ltd Consultant: JRB

Plant Used: Wheeled Excavator Length: 1.5 Width: 0.5

Well	Water	Depth	Legend	Stratum Description	Detailed Description	Depth m
		0.15		TOPSOIL: Grass over brown sandy		
		0.40		CLAY. Sand is fine to medium. Abundant rootlets		
				Light brown very clayey fine to medium SAND with occasional rootlets		0.5
				Brown mottled light grey and orange slightly gravelly CLAY. Gravel is medium to coarse sub-angular of iron rich calcareous concretions		1.0
						1.5
						2.0
						2.5
						3.0
						3.5
						4.0
						4.5
		2.80				

Position cleared with CAT and progressed to 3m bgl whereupon soakaway testing was performed. Position backfilled with arisings.



Appendix D: Infiltration test results



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP05	Test number:	1	Date of test:	29/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	1.9
Effective depth of Pit, D ⁽¹⁾	(m)	1.27
Length of Pit, L	(m)	1.9
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

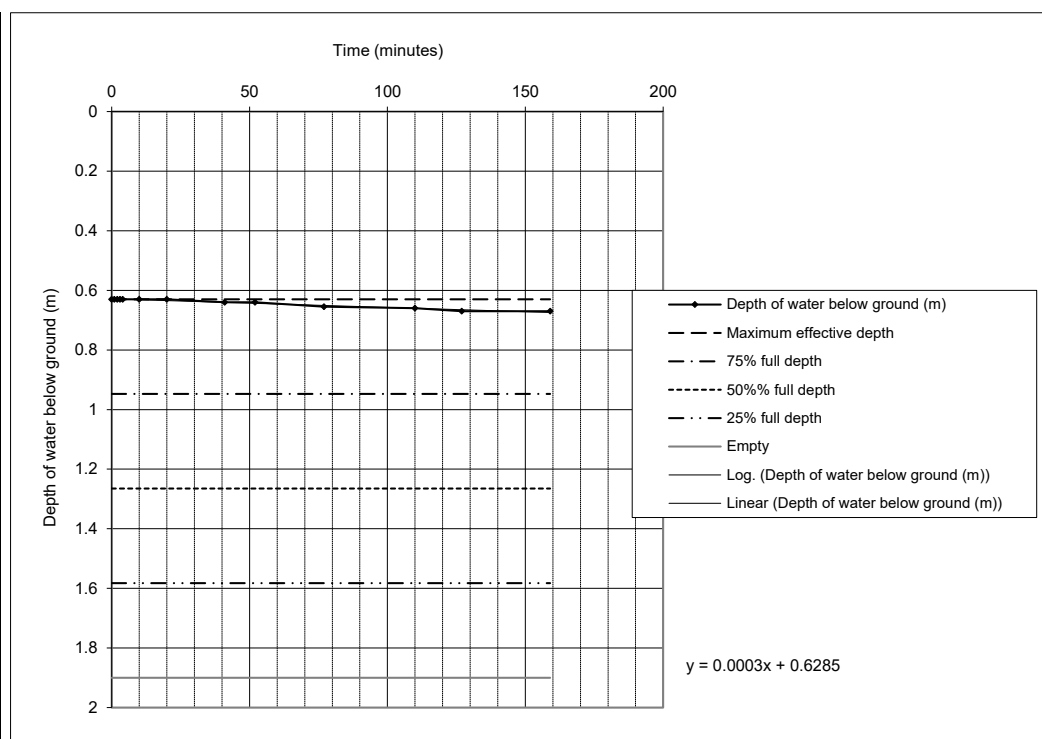
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.60
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.60
Internal surface area of pit up to 50% level, a _{p50} , (m ²)	4.00
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.63
1	0.63
2	0.63
3	0.63
4	0.63
10	0.63
20	0.63
41	0.64
52	0.64
77	0.655
110	0.66
127	0.67
159	0.67



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Extrapolated Time for water to drop to 75% effective depth

Extrapolated Time for water to drop to 25% effective depth

Time taken for water to drop from 75% to 25% (mins)

Time taken for water to drop from 75% to 25% (secs)

Extrapolated using linear trendline equation

Extrapolated using linear trendline equation

0

0

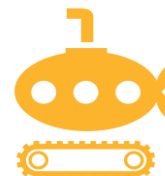
$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.60}{4.00 \times 0}$$

Soil infiltration rate, f

=

Failed

m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP04E	Test number:	1	Date of test:	28/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	1.5
Effective depth of Pit, D ⁽¹⁾	(m)	0.84
Length of Pit, L	(m)	1.9
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

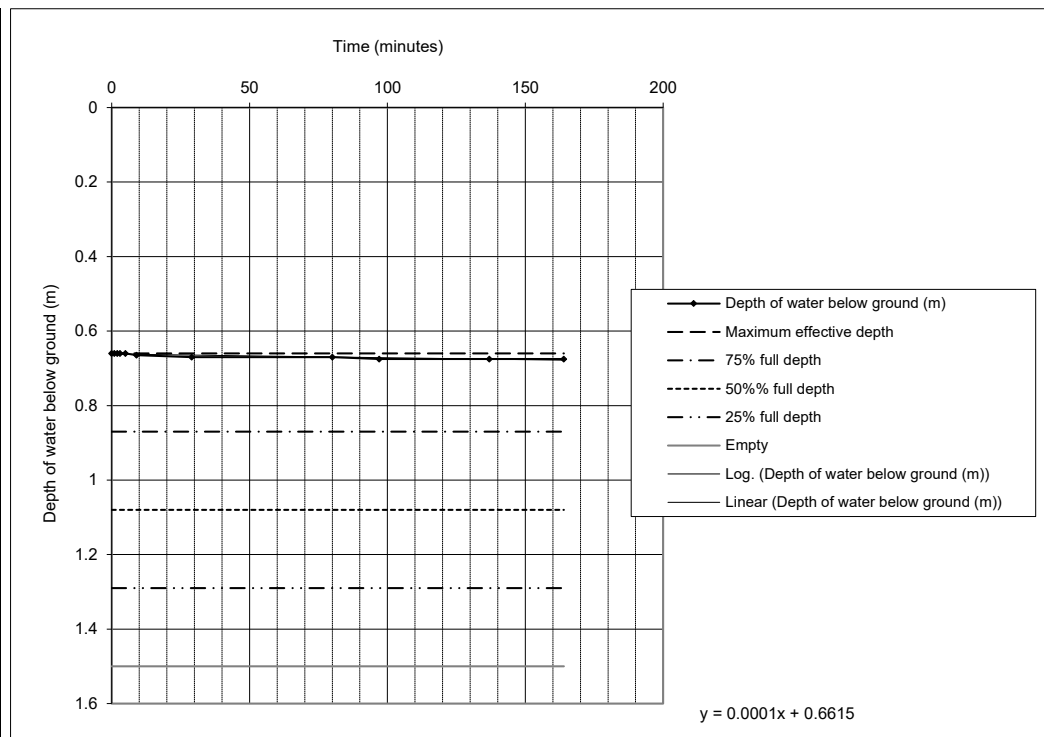
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.40
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.40
Internal surface area of pit up to 50% level, a _{p50} (m ²)	2.97
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.66
1	0.66
2	0.66
3	0.66
5	0.66
9	0.665
29	0.67
80	0.67
97	0.675
137	0.675
164	0.675



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Extrapolated Time for water to drop to 75% effective depth

Extrapolated Time for water to drop to 25% effective depth

Time taken for water to drop from 75% to 25% (mins)

Time taken for water to drop from 75% to 25% (secs)

Extrapolated using linear trendline equation

Extrapolated using linear trendline equation

0

0

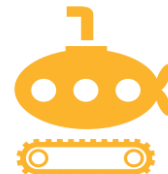
$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.40}{2.97 \times 0}$$

Soil infiltration rate, f

=

Failed

m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP04D	Test number:	1	Date of test:	28/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.7
Effective depth of Pit, D ⁽¹⁾	(m)	2.01
Length of Pit, L	(m)	1.9
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

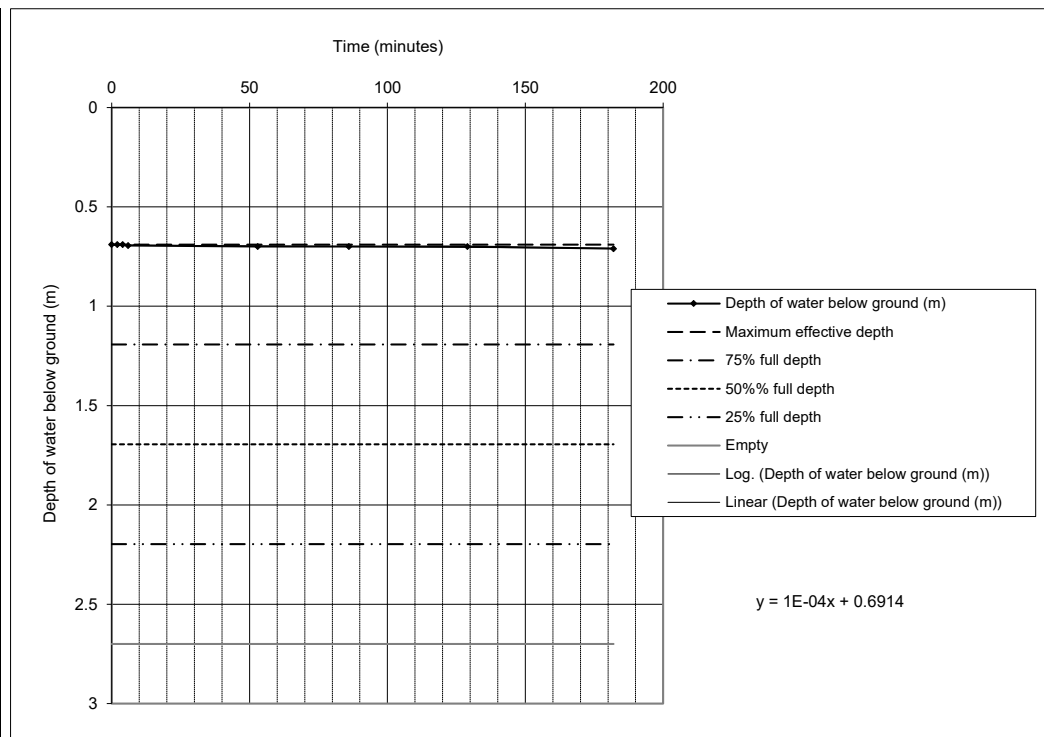
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.95
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.95
Internal surface area of pit up to 50% level, a _{p50} (m ²) = area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	5.77

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.69
2	0.69
4	0.69
6	0.695
53	0.7
86	0.7
129	0.7
182	0.71

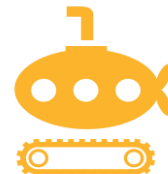


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Extrapolated Time for water to drop to 75% effective depth		Extrapolated using linear trendline equation
Extrapolated Time for water to drop to 25% effective depth		Extrapolated using linear trendline equation
Time taken for water to drop from 75% to 25% (mins)	0	
Time taken for water to drop from 75% to 25% (secs)	0	

$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.95}{5.77 \times 0}$$

Soil infiltration rate, f = Failed m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP04C	Test number:	1	Date of test:	28/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.5
Effective depth of Pit, D ⁽¹⁾	(m)	1.76
Length of Pit, L	(m)	1.8
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

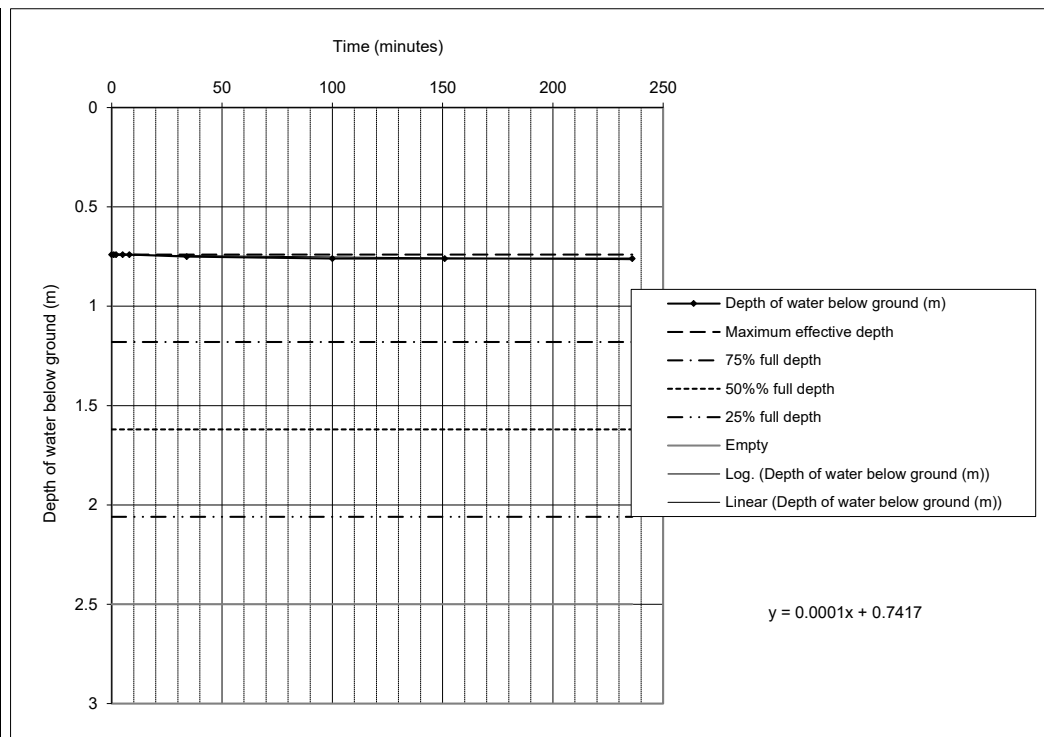
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.79
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.79
Internal surface area of pit up to 50% level, a _{p50} (m ²)	4.95
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.74
1	0.74
2	0.74
5	0.74
8	0.74
34	0.75
100	0.76
151	0.76
236	0.76



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Extrapolated Time for water to drop to 75% effective depth

Extrapolated Time for water to drop to 25% effective depth

Time taken for water to drop from 75% to 25% (mins)

Time taken for water to drop from 75% to 25% (secs)

Extrapolated using linear trendline equation

Extrapolated using linear trendline equation

0

0

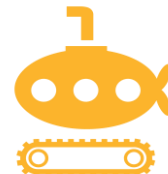
$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.79}{4.95 \times 0}$$

Soil infiltration rate, f

=

Failed

m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP04B	Test number:	1	Date of test:	28/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.4
Effective depth of Pit, D ⁽¹⁾	(m)	1.68
Length of Pit, L	(m)	1.7
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

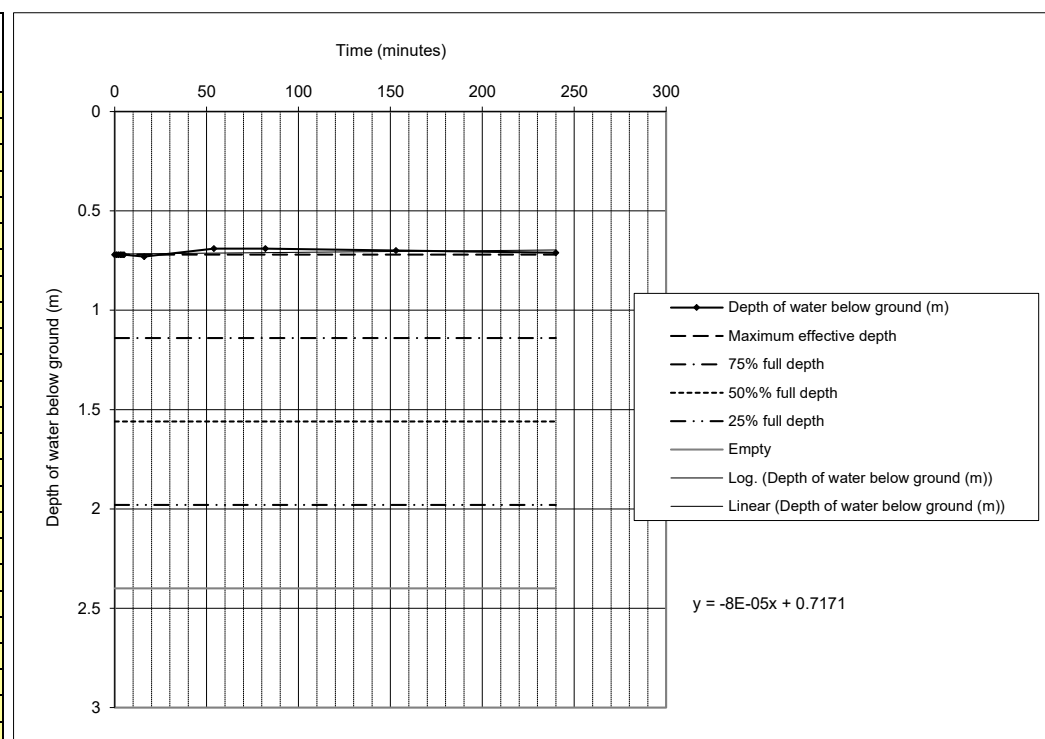
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.71
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.71
Internal surface area of pit up to 50% level, a _{p50} (m ²)	4.55
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.72
1	0.72
2	0.72
3	0.72
4	0.72
5	0.72
16	0.73
54	0.69
82	0.69
153	0.7
240	0.71



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Extrapolated Time for water to drop to 75% effective depth

Extrapolated Time for water to drop to 25% effective depth

Time taken for water to drop from 75% to 25% (mins)

Time taken for water to drop from 75% to 25% (secs)

Extrapolated using linear trendline equation

Extrapolated using linear trendline equation

0

0

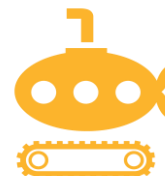
$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.71}{4.55 \times 0}$$

Soil infiltration rate, f

=

Failed

m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP04A	Test number:	1	Date of test:	28/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	1.6
Effective depth of Pit, D ⁽¹⁾	(m)	1.04
Length of Pit, L	(m)	1.9
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

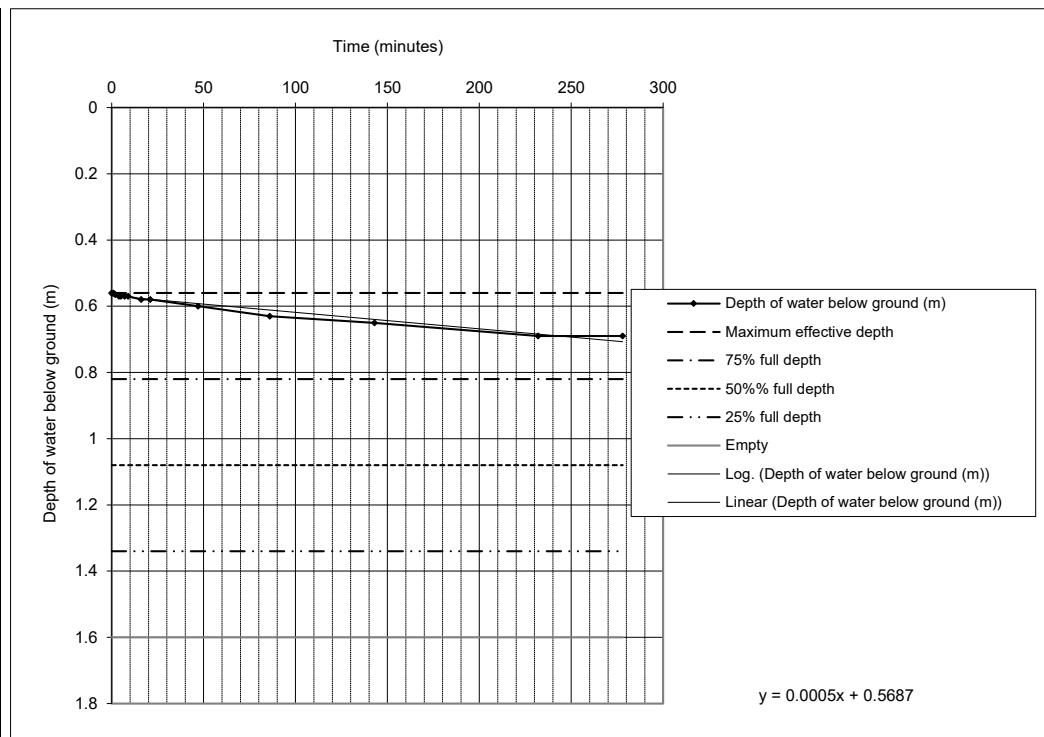
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.49
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.49
Internal surface area of pit up to 50% level, a _{p50} (m ²)	3.45
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.56
1	0.56
2	0.565
4	0.57
5	0.57
7	0.57
9	0.57
16	0.58
21	0.58
47	0.6
86	0.63
143	0.65
232	0.69
278	0.69

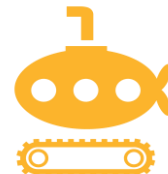


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Extrapolated Time for water to drop to 75% effective depth		Extrapolated using linear trendline equation
Extrapolated Time for water to drop to 25% effective depth		Extrapolated using linear trendline equation
Time taken for water to drop from 75% to 25% (mins)	0	
Time taken for water to drop from 75% to 25% (secs)	0	

$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.49}{3.45 \times 0}$$

Soil infiltration rate, f = Failed m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP03	Test number:	1	Date of test:	27/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.5
Effective depth of Pit, D ⁽¹⁾	(m)	1.61
Length of Pit, L	(m)	1.6
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

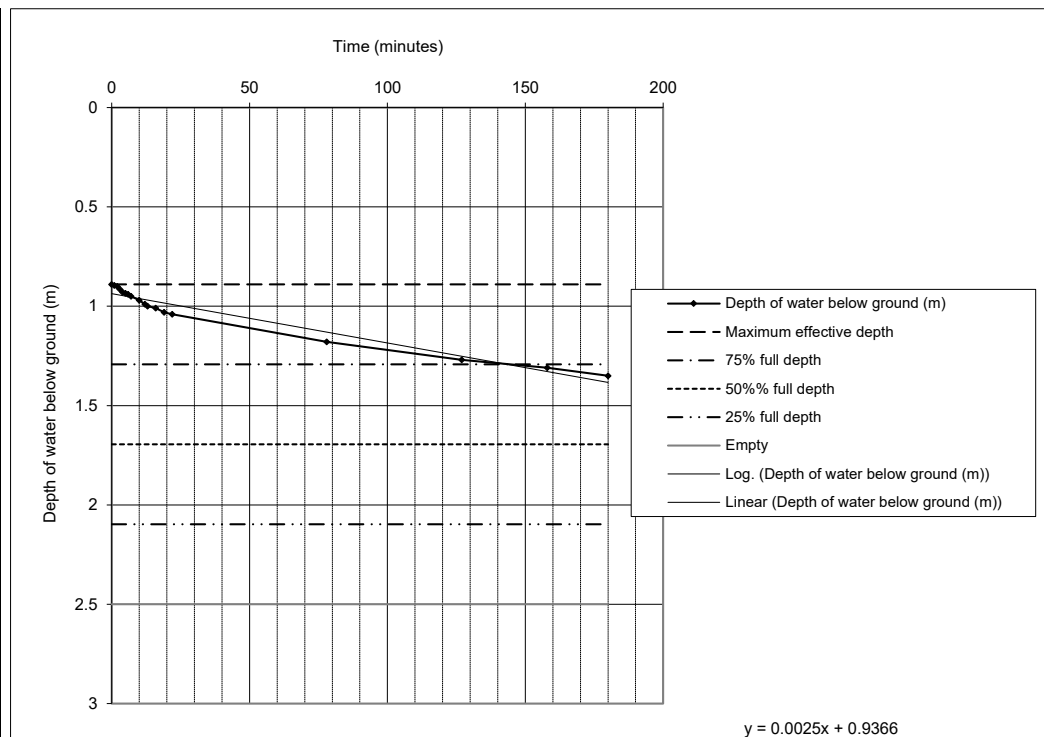
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.64
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.64
Internal surface area of pit up to 50% level, a _{p50} , (m ²)	4.18
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.89
1	0.895
2	0.9
3	0.915
4	0.93
5	0.935
6	0.94
7	0.95
10	0.97
12	0.99
13	1
16	1.01
19	1.03
22	1.04
78	1.18
127	1.27
158	1.31
180	1.35



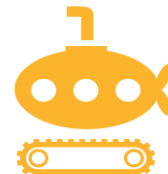
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Extrapolated Time for water to drop to 75% effective depth
 Extrapolated Time for water to drop to 25% effective depth
 Time taken for water to drop from 75% to 25% (mins)
 Time taken for water to drop from 75% to 25% (secs)

Extrapolated using linear trendline equation
 Extrapolated using linear trendline equation
 0
 0

$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.64}{4.18 \times 0}$$

Soil infiltration rate, f = Failed m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP02B	Test number:	1	Date of test:	27/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.9
Effective depth of Pit, D ⁽¹⁾	(m)	2.06
Length of Pit, L	(m)	1.8
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

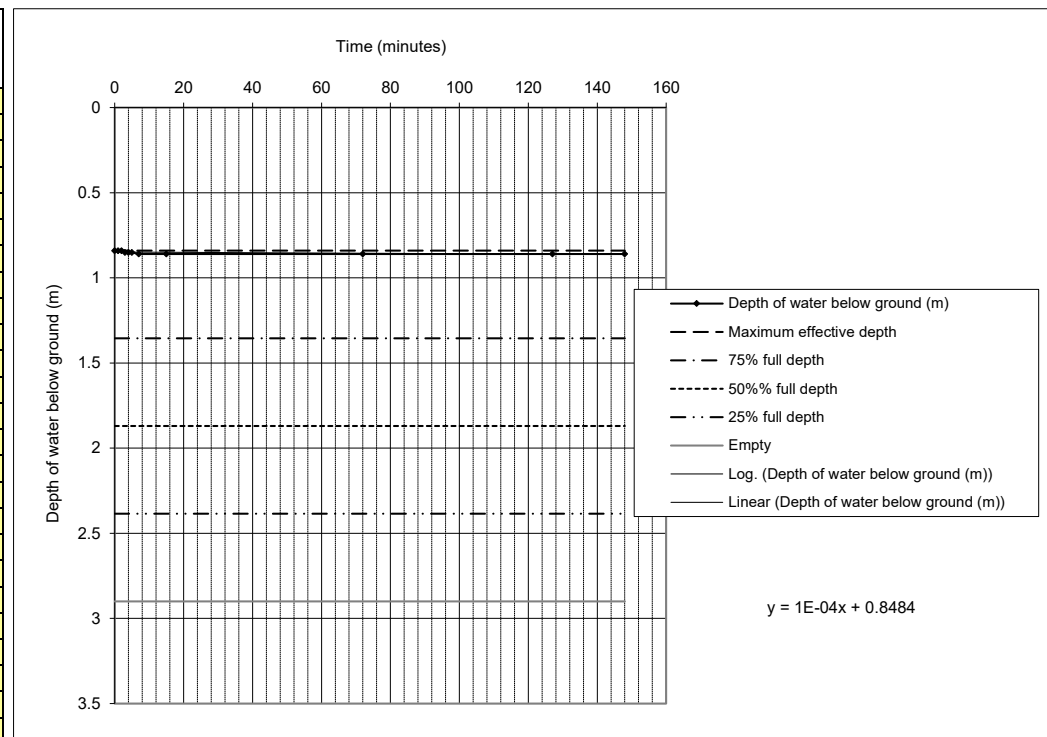
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.93
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.93
Internal surface area of pit up to 50% level, a _{p50} , (m ²) = area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	5.64

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	0.84
1	0.84
2	0.84
3	0.85
4	0.85
5	0.85
7	0.86
15	0.86
72	0.86
127	0.86
148	0.86

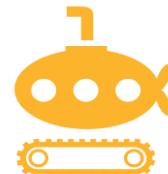


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Extrapolated Time for water to drop to 75% effective depth		Extrapolated using linear trendline equation
Extrapolated Time for water to drop to 25% effective depth		Extrapolated using linear trendline equation
Time taken for water to drop from 75% to 25% (mins)	0	
Time taken for water to drop from 75% to 25% (secs)	0	

$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.93}{5.64 \times 0}$$

Soil infiltration rate, f = Failed m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP02A	Test number:	1	Date of test:	27/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.5
Effective depth of Pit, D ⁽¹⁾	(m)	1.3
Length of Pit, L	(m)	2
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

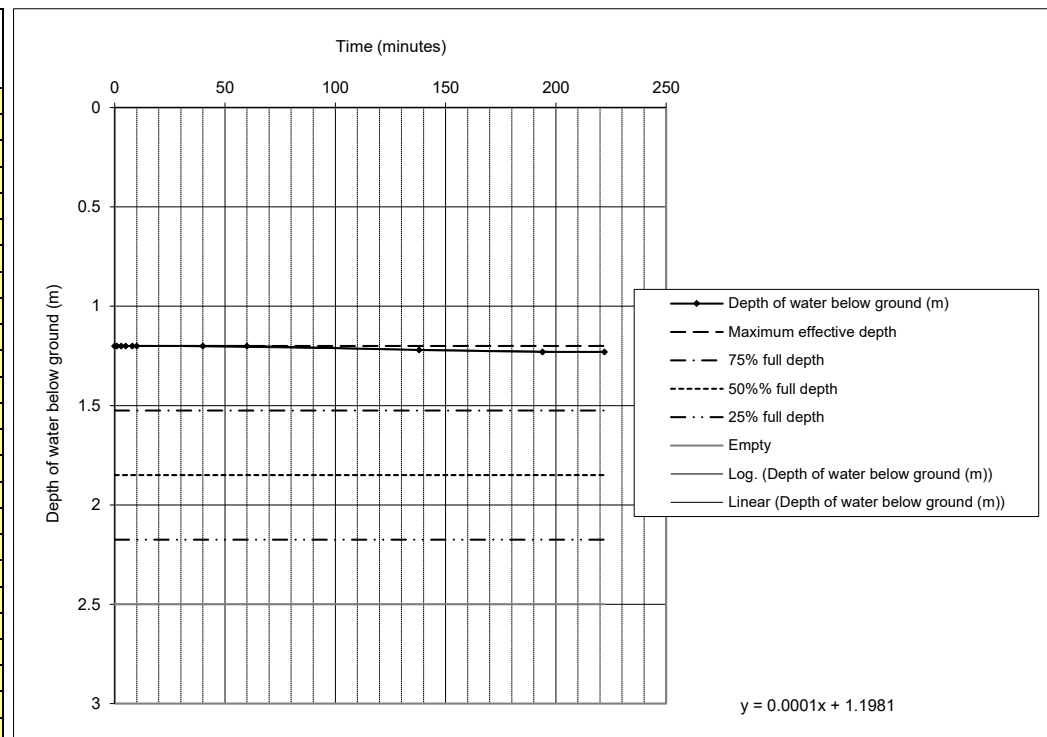
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.65
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.65
Internal surface area of pit up to 50% level, a _{p50} , (m ²)	4.25
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	1.2
1	1.2
3	1.2
5	1.2
8	1.2
10	1.2
40	1.2
60	1.2
138	1.22
194	1.23
222	1.23

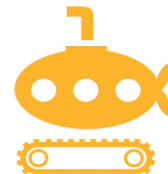


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Extrapolated Time for water to drop to 75% effective depth		Extrapolated using linear trendline equation
Extrapolated Time for water to drop to 25% effective depth		Extrapolated using linear trendline equation
Time taken for water to drop from 75% to 25% (mins)	0	
Time taken for water to drop from 75% to 25% (secs)	0	

$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.65}{4.25 \times 0}$$

Soil infiltration rate, f = Failed m/s



BRE SOAKAWAY INFILTRATION RATE CALCULATION

Sheet 1 of 1

The following spreadsheet calculations are based on the method given in BRE Digest 365

Note: enter values only in the yellow-highlighted cells - do not alter any other cell, even if it appears to be blank.

Project title:	Ansty Farm			Project number:	P21367
Trial pit number:	TP01	Test number:	1	Date of test:	27/06/2023
Test carried out by:	JRB	Calculations by:	JRB	Verified by:	JEM

Details of pit

Actual depth of pit, D'	(m)	2.8
Effective depth of Pit, D ⁽¹⁾	(m)	1.67
Length of Pit, L	(m)	1.6
Width of Pit, W	(m)	0.5
Proportion of pit volume occupied by gravel solids, P _g ⁽²⁾		0

Notes:

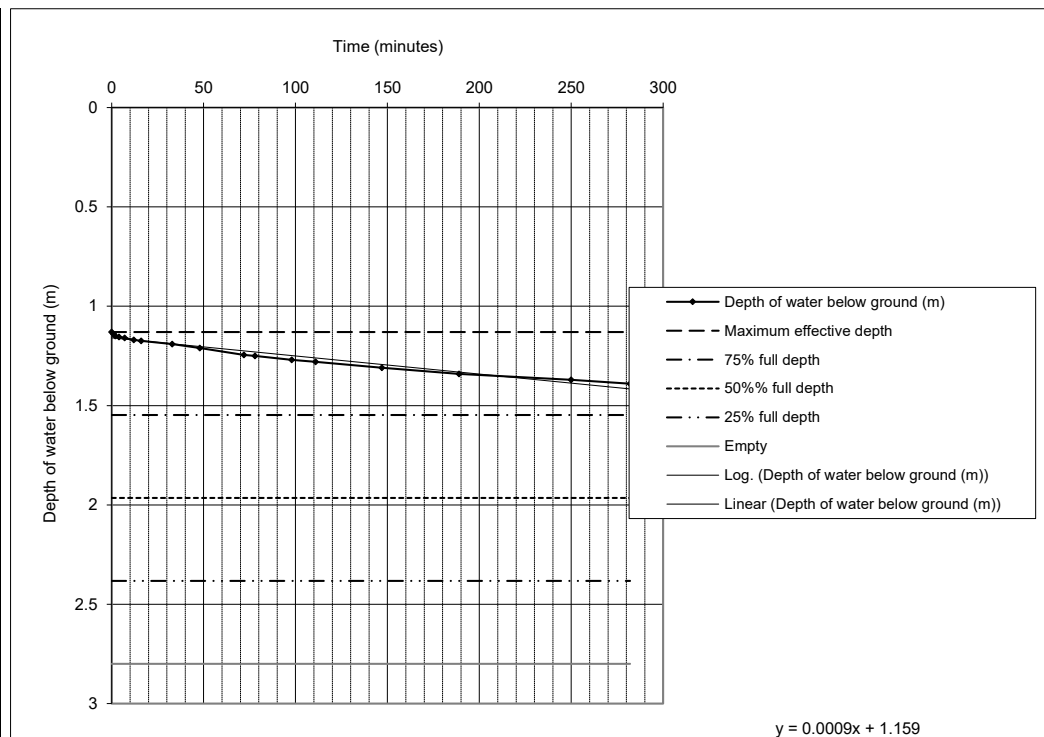
(1) For a standard test in which the pit is filled with water, the effective depth will be equal to the actual depth. If the pit is not completely filled, it will be equal to the depth at the start of each test.

Volume, V, of pit between 75% and 25% depths (m ³) = L x W x ½D =	0.67
Effective volume between 75% and 25% depths, V _{p75-25} = V x P _g =	0.67
Internal surface area of pit up to 50% level, a _{p50} (m ²)	4.31
= area of 2 sides + 2 ends + base = (2 x L x ½D) + (2 x W x ½D) + (L x W)	

(2) Typically, for a gravel of dry density 18kN/m³, specific gravity of rock solids 2.65, the proportion of volume occupied by rock is 0.69 (i.e. 69%). For dry density 16kN/m³, the rock volume is 0.61 (61%), and for dry density 20kN/m³, it is 0.77 (77%). If the pit is left open (gravel is not used) then put P_g = 0.

Water level readings

Time elapsed (min)	Depth of water below ground (m)
0	1.13
1	1.14
2	1.15
4	1.155
7	1.16
12	1.17
16	1.175
33	1.19
48	1.21
72	1.245
78	1.25
98	1.27
111	1.28
147	1.31
189	1.34
250	1.37
282	1.39



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Extrapolated Time for water to drop to 75% effective depth

Extrapolated Time for water to drop to 25% effective depth

Time taken for water to drop from 75% to 25% (mins)

Time taken for water to drop from 75% to 25% (secs)

Extrapolated using linear trendline equation

Extrapolated using linear trendline equation

0

0

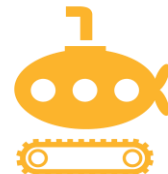
$$= \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}} = \frac{0.67}{4.31 \times 0}$$

Soil infiltration rate, f

=

Failed

m/s





Appendix E: Photos





TP2A Pit		TP2A excavated to 2.5m bgl	
Project	Ansty Farm	Date	26/06/23
Project No.	P21367	Engineer	JRB
Client	Fairfax Properties Ltd	Comments	



TP2B Pit		TP2B excavated to 2.9m bgl	
Project	Ansty Farm	Date	26/06/23
Project No.	P21367	Engineer	JRB
Client	Fairfax Properties Ltd	Comments	



TP3 Pit and Arisings		TP4E Pit and Arisings	
Project	Ansty Farm	Date	26/06/23
Project No.	P21367	Engineer	JRB
Client	Fairfax Properties Ltd	Comments	



TP4C Pit and Arisings		TP4D Pit and Arisings	
Project	Ansty Farm	Date	26/06/23
Project No.	P21367	Engineer	JRB
Client	Fairfax Properties Ltd	Comments	



TP10 Pit		TP8 Pit	
Project	Ansty Farm	Date	26/06/23
Project No.	P21367	Engineer	JRB
Client	Fairfax Properties Ltd	Comments	

Appendix E Sewer asset plans

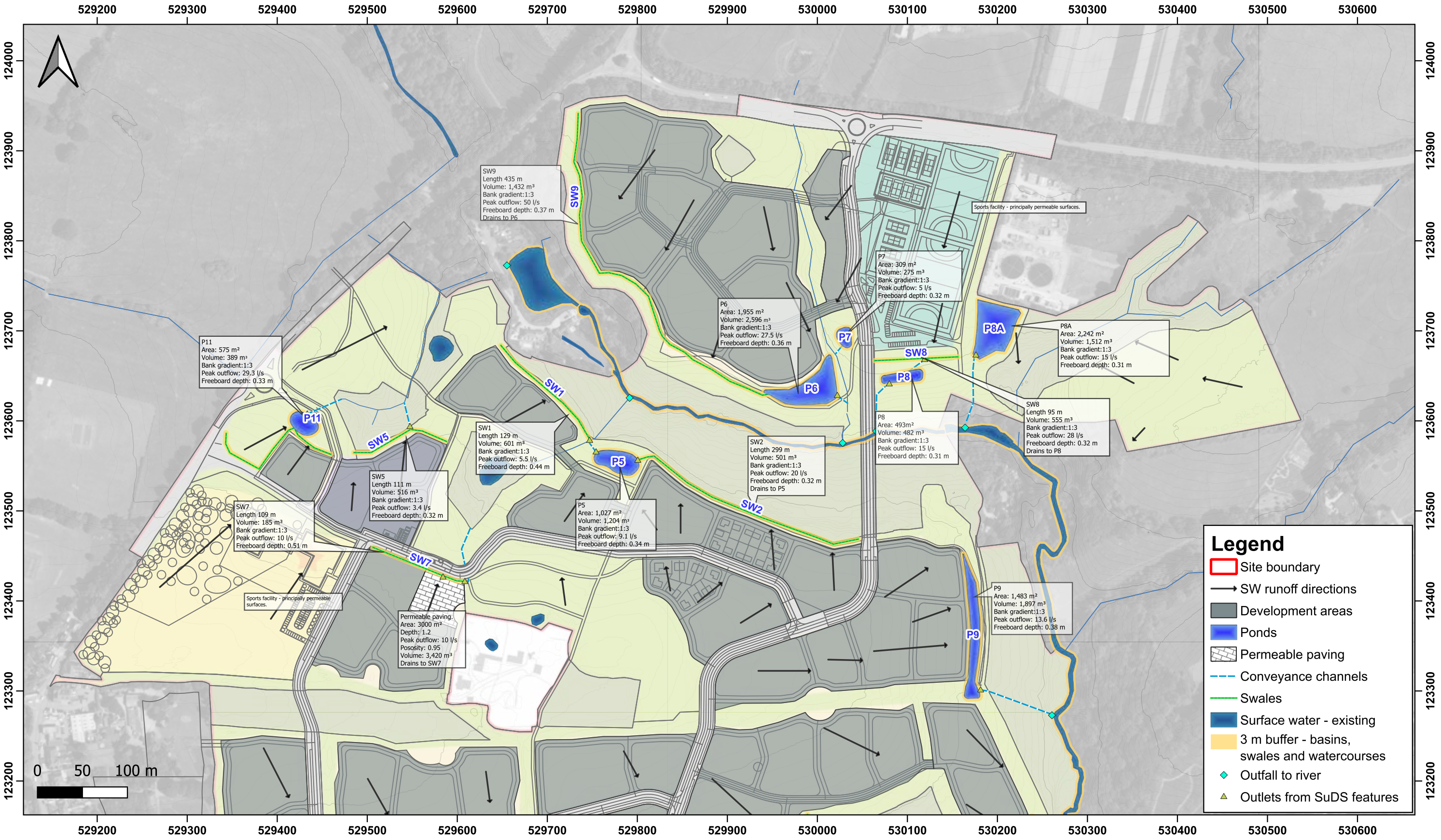
Appendix F Greenfield runoff calculations

Pre-development discharge

Site Makeup	Greenfield ▼	OK Cancel
Greenfield Method	IH124 ▼	
Positively Drained Area (ha)	101.740	Load
SAAR (mm)	813	
Soil Index	4 ▼	
SPR	0.47	
Region	1 ▼	
Betterment (%)	0	
	Calc	
QBar (l/s)	541.2	

Return Period (years)	Growth Factor	Q (l/s)
1	0.85	460.0
30	1.95	1055.3
100	2.48	1342.2

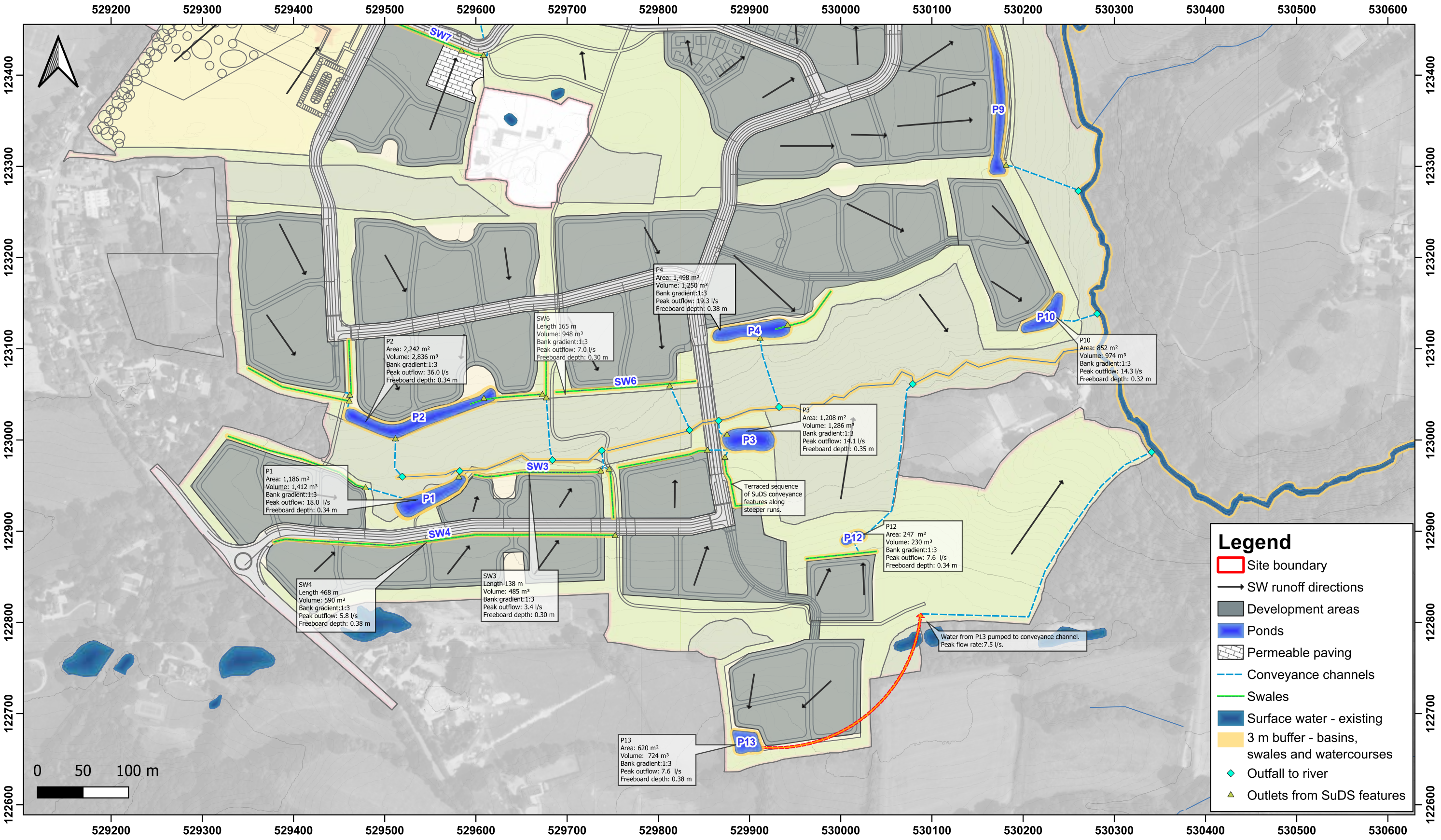
Appendix G Outline drainage strategy



Outline drainage layout and relevant design information.
Freeboard depth = max water level under T30+CCA event minus bank level

Date	2025	Drawn	HJK
Scale	1:4000	Checked	JEM
Original	A3	Revision	1
File reference		Projects\2266 - Ansty\GIS	





Outline drainage layout and relevant design information.
Freeboard depth = max water level under T30+CCA event minus bank level

Appendix H Drainage calculations

Design Settings

Rainfall Methodology	FEH-22	Minimum Velocity (m/s)	1.00
Return Period (years)	100	Connection Type	Level Soffits
Additional Flow (%)	0	Minimum Backdrop Height (m)	0.200
CV	1.000	Preferred Cover Depth (m)	1.200
Time of Entry (mins)	5.00	Include Intermediate Ground	✓
Maximum Time of Concentration (mins)	30.00	Enforce best practice design rules	✓
Maximum Rainfall (mm/hr)	50.0		

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Node Type	Diameter (mm)	Depth (m)
P10	1.221	15.00	42.000	Manhole	1500	1.500
P4	1.736	15.00	54.950	Manhole	1800	1.500
P3	1.432	15.00	46.770	Manhole	1500	1.300
P12	0.260	10.00	57.400	Manhole	1200	1.500
P13	0.846	15.00	56.930	Manhole	1200	1.650
P2	3.416	30.00	57.300	Junction		1.500
P1	1.689	15.00	52.500	Junction		1.500
P7	0.340	15.00	51.950	Manhole	1200	1.200
P6	1.500	5.00	49.150	Manhole	2100	1.650
P8A	1.000	20.00	49.750	Manhole	1500	0.750
P11	0.564	10.00	59.900	Manhole	1800	0.800
P5	0.884	15.00	52.150	Manhole	1500	1.500
P9	1.994	20.00	49.350	Manhole	1500	1.600
SW3	0.421	15.00	50.350	Junction		1.000
SW4	0.300	15.00	55.200	Junction		0.700
SW6	0.881	20.00	52.650	Junction		1.350
SW7	0.000	5.00	67.680	Junction		0.700
SW5	0.444	15.00	58.150	Junction		1.200
SW1	0.453	15.00	51.500	Junction		1.200
SW2	0.408	15.00	53.650	Junction		0.700
Perm1	2.442	30.00	69.150	Junction		1.200
P8	0.000	15.00	47.750	Junction		1.300
SW8	0.848	10.00	53.400	Junction		1.350
SW9	1.500	30.00	53.500	Junction		1.000

Simulation Settings

Rainfall Methodology	FEH-22	Drain Down Time (mins)	240	5 year (l/s)	682.2
Rainfall Events	Singular	Additional Storage (m³/ha)	0.0	30 year (l/s)	1279.1
Summer CV	1.000	Starting Level (m)		100 year (l/s)	1700.1
Winter CV	1.000	Check Discharge Rate(s)	✓	Check Discharge Volume	✓
Analysis Speed	Normal	1 year (l/s)	453.0	100 year 360 minute (m³)	35592
Skip Steady State	x	2 year (l/s)	469.0		

Storm Durations

15 | 30 | 60 | 120 | 180 | 240 | 360 | 480 | 600 | 720 | 960 | 1440

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
1	0	0	0
30	40	0	0
100	45	0	0

Pre-development Discharge Rate

Site Makeup	Greenfield	Growth Factor 30 year	2.40
Greenfield Method	IH124	Growth Factor 100 year	3.19
Positively Drained Area (ha)	101.740	Betterment (%)	0
SAAR (mm)	813	QBar	541.2
Soil Index	4	Q 1 year (l/s)	460.0
SPR	0.47	Q 2 year (l/s)	476.2
Region	7	Q 5 year (l/s)	692.7
Growth Factor 1 year	0.85	Q 30 year (l/s)	1298.9
Growth Factor 2 year	0.88	Q 100 year (l/s)	1726.4
Growth Factor 5 year	1.28		

Pre-development Discharge Volume

Site Makeup	Greenfield	Return Period (years)	100
Greenfield Method	FSR/FEH	Climate Change (%)	0
Positively Drained Area (ha)	101.740	Storm Duration (mins)	360
Soil Index	4	Betterment (%)	0
SPR	0.47	PR	0.509
CWI	121.926	Runoff Volume (m³)	35592

Node P2 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	55.800	Product Number	CTL-SHE-0250-3600-1500-3600
Design Depth (m)	1.500	Min Outlet Diameter (m)	0.300
Design Flow (l/s)	36.0	Min Node Diameter (mm)	2100

Node P1 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	51.000	Product Number	CTL-SHE-0182-1800-1500-1800
Design Depth (m)	1.500	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	18.0	Min Node Diameter (mm)	1800

Node P3 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	45.470	Product Number	CTL-SHE-0166-1410-1300-1410
Design Depth (m)	1.300	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	14.1	Min Node Diameter (mm)	1500

Node P4 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	53.450	Product Number	CTL-SHE-0188-1930-1500-1930
Design Depth (m)	1.500	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	19.3	Min Node Diameter (mm)	1800

Node P12 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	55.900	Product Number	CTL-SHE-0073-2800-1500-2800
Design Depth (m)	1.500	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	2.8	Min Node Diameter (mm)	1200

Node P13 Offline Pump Control

Flap Valve	x	Design Depth (m)	1.650	Switch off depth (m)	0.000
Loop to Node		Design Flow (l/s)	7.6		
Invert Level (m)	55.280	Switch on depth (m)	0.050		

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.050	7.600	1.650	7.600

Node P10 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	40.500	Product Number	CTL-SHE-0164-1430-1500-1430
Design Depth (m)	1.500	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	14.3	Min Node Diameter (mm)	1500

Node P9 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	47.750	Product Number	CTL-SHE-0159-1360-1600-1360
Design Depth (m)	1.600	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	13.6	Min Node Diameter (mm)	1500

Node P5 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	50.650	Product Number	CTL-SHE-0132-9100-1500-9100
Design Depth (m)	1.500	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	9.1	Min Node Diameter (mm)	1500

Node P11 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	59.100	Product Number	CTL-SHE-0236-2930-0800-2930
Design Depth (m)	0.800	Min Outlet Diameter (m)	0.300
Design Flow (l/s)	29.3	Min Node Diameter (mm)	1800

Node P6 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	47.500	Product Number	CTL-SHE-0219-2750-1650-2750
Design Depth (m)	1.650	Min Outlet Diameter (m)	0.300
Design Flow (l/s)	27.5	Min Node Diameter (mm)	2100

Node P8A Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	49.000	Product Number	CTL-SHE-0178-1500-0750-1500
Design Depth (m)	0.750	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	15.0	Min Node Diameter (mm)	1500

Node P7 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	50.750	Product Number	CTL-SHE-0103-5000-1200-5000
Design Depth (m)	1.200	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	5.0	Min Node Diameter (mm)	1200

Node SW1 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	50.300	Product Number	CTL-SHE-0107-5500-1200-5500
Design Depth (m)	1.200	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	5.5	Min Node Diameter (mm)	1200

Node SW2 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node	P5	Sump Available	✓
Invert Level (m)	52.850	Product Number	CTL-SHE-0202-2000-0700-2000
Design Depth (m)	0.700	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	20.0	Min Node Diameter (mm)	1500

Node SW3 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	49.350	Product Number	CTL-SHE-0088-3400-1000-3400
Design Depth (m)	1.000	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	3.4	Min Node Diameter (mm)	1200

Node SW4 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	54.500	Product Number	CTL-SHE-0118-5800-0700-5800
Design Depth (m)	0.700	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	5.8	Min Node Diameter (mm)	1200

Node SW5 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	56.950	Product Number	CTL-SHE-0085-3400-1200-3400
Design Depth (m)	1.200	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	3.4	Min Node Diameter (mm)	1200

Node SW6 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	51.300	Product Number	CTL-SHE-0119-7100-1350-7100
Design Depth (m)	1.350	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	7.1	Min Node Diameter (mm)	1200

Node SW7 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	66.980	Product Number	CTL-SHE-0147-9900-0900-9900
Design Depth (m)	0.900	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	9.9	Min Node Diameter (mm)	1200

Node Perm1 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node	SW7	Sump Available	✓
Invert Level (m)	67.950	Product Number	CTL-SHE-0143-1000-1200-1000
Design Depth (m)	1.200	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	10.0	Min Node Diameter (mm)	1500

Node P8 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node		Sump Available	✓
Invert Level (m)	46.450	Product Number	CTL-SHE-0171-1500-1300-1500
Design Depth (m)	1.300	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	15.0	Min Node Diameter (mm)	1500

Node SW8 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node	P8	Sump Available	✓
Invert Level (m)	52.050	Product Number	CTL-SHE-0225-2800-1350-2800
Design Depth (m)	1.350	Min Outlet Diameter (m)	0.300
Design Flow (l/s)	28.0	Min Node Diameter (mm)	2100

Node SW9 Offline Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Loop to Node	P6	Sump Available	✓
Invert Level (m)	52.500	Product Number	CTL-SHE-0294-5000-1000-5000
Design Depth (m)	1.000	Min Outlet Diameter (m)	0.375
Design Flow (l/s)	50.0	Min Node Diameter (mm)	2100

Node P7 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	50.750
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	150.0	0.0	1.200	308.5	0.0

Node P6 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	47.500
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	1200.0	0.0	1.650	1954.7	0.0

Node P8A Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	49.000
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	1800.0	0.0	0.750	2242.0	0.0

Node P11 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	59.100
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	130

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	400.0	0.0	0.800	575.4	0.0

Node P5 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	50.650
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	580.0	0.0	1.500	1027.8	0.0

Node P9 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	47.750
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	900.0	0.0	1.600	1482.8	0.0

Node P10 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	40.500
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	450.0	0.0	1.500	852.0	0.0

Node P4 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	53.450
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	750.0	0.0	1.500	1250.5	0.0

Node P3 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	45.470
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	775.0	0.0	1.300	1207.7	0.0

Node P12 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	55.900
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	60.0	0.0	1.500	247.2	0.0

Node P13 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	55.280
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	260.0	0.0	1.650	619.9	0.0

Node P2 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	55.800
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	1550.0	0.0	1.500	2241.7	0.0

Node P1 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	51.000
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	700.0	0.0	1.500	1185.7	0.0

Node SW3 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	49.350
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	100.0	0.0	1.000	875.0	0.0

Node SW4 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	54.500
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	140.0	0.0	0.700	1544.0	0.0

Node SW6 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	51.300
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	45.9	0.0	1.350	1360.8	0.0

Node SW7 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	66.980
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	0

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	40.0	0.0	0.700	488.8	0.0

Node SW5 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	56.950
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	32.6	0.0	1.200	831.0	0.0

Node SW1 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	50.300
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	38.6	0.0	1.200	965.9	0.0

Node SW2 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	52.950
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	99

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	79.8	0.0	0.700	1356.0	0.0

Node Perm1 Carpark Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Invert Level (m)	67.950	Slope (1:X)	1000.0
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)		Depth (m)	1.300
Safety Factor	2.0	Width (m)	30.000	Inf Depth (m)	
Porosity	0.95	Length (m)	100.000		

Node P8 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	46.450
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	250.0	0.0	1.300	492.5	0.0

Node SW8 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	52.050
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	206

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	27.9	0.0	1.350	798.0	0.0

Node SW9 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	52.500
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	182

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	130.5	0.0	1.000	2740.5	0.0

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
1 year 15 minute summer	70.680	20.000
1 year 15 minute winter	49.600	20.000
1 year 30 minute summer	45.942	13.000
1 year 30 minute winter	32.240	13.000
1 year 60 minute summer	31.407	8.300
1 year 60 minute winter	20.866	8.300
1 year 120 minute summer	24.028	6.350
1 year 120 minute winter	15.964	6.350
1 year 180 minute summer	20.233	5.207
1 year 180 minute winter	13.152	5.207
1 year 240 minute summer	16.856	4.455
1 year 240 minute winter	11.199	4.455
1 year 360 minute summer	13.613	3.503
1 year 360 minute winter	8.849	3.503
1 year 480 minute summer	11.040	2.918
1 year 480 minute winter	7.335	2.918
1 year 600 minute summer	9.209	2.519
1 year 600 minute winter	6.292	2.519
1 year 720 minute summer	8.313	2.228
1 year 720 minute winter	5.587	2.228
1 year 960 minute summer	6.944	1.829
1 year 960 minute winter	4.600	1.829
1 year 1440 minute summer	5.157	1.382
1 year 1440 minute winter	3.466	1.382
30 year +40% CC 15 minute summer	404.031	114.327
30 year +40% CC 15 minute winter	283.531	114.327
30 year +40% CC 30 minute summer	268.760	76.050
30 year +40% CC 30 minute winter	188.603	76.050
30 year +40% CC 60 minute summer	182.757	48.297
30 year +40% CC 60 minute winter	121.419	48.297
30 year +40% CC 120 minute summer	110.969	29.326
30 year +40% CC 120 minute winter	73.725	29.326
30 year +40% CC 180 minute summer	84.525	21.751
30 year +40% CC 180 minute winter	54.943	21.751
30 year +40% CC 240 minute summer	66.381	17.542
30 year +40% CC 240 minute winter	44.102	17.542
30 year +40% CC 360 minute summer	50.119	12.897
30 year +40% CC 360 minute winter	32.578	12.897
30 year +40% CC 480 minute summer	39.096	10.332
30 year +40% CC 480 minute winter	25.974	10.332
30 year +40% CC 600 minute summer	31.778	8.692
30 year +40% CC 600 minute winter	21.713	8.692
30 year +40% CC 720 minute summer	28.156	7.546
30 year +40% CC 720 minute winter	18.922	7.546
30 year +40% CC 960 minute summer	22.946	6.042
30 year +40% CC 960 minute winter	15.200	6.042
30 year +40% CC 1440 minute summer	16.564	4.439

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
30 year +40% CC 1440 minute winter	11.132	4.439
100 year +45% CC 15 minute summer	531.436	150.378
100 year +45% CC 15 minute winter	372.937	150.378
100 year +45% CC 30 minute summer	355.905	100.709
100 year +45% CC 30 minute winter	249.758	100.709
100 year +45% CC 60 minute summer	244.665	64.658
100 year +45% CC 60 minute winter	162.550	64.658
100 year +45% CC 120 minute summer	145.181	38.367
100 year +45% CC 120 minute winter	96.455	38.367
100 year +45% CC 180 minute summer	109.626	28.211
100 year +45% CC 180 minute winter	71.260	28.211
100 year +45% CC 240 minute summer	85.724	22.654
100 year +45% CC 240 minute winter	56.953	22.654
100 year +45% CC 360 minute summer	64.487	16.595
100 year +45% CC 360 minute winter	41.918	16.595
100 year +45% CC 480 minute summer	50.328	13.300
100 year +45% CC 480 minute winter	33.436	13.300
100 year +45% CC 600 minute summer	40.984	11.210
100 year +45% CC 600 minute winter	28.003	11.210
100 year +45% CC 720 minute summer	36.402	9.756
100 year +45% CC 720 minute winter	24.465	9.756
100 year +45% CC 960 minute summer	29.839	7.857
100 year +45% CC 960 minute winter	19.766	7.857
100 year +45% CC 1440 minute summer	21.743	5.827
100 year +45% CC 1440 minute winter	14.613	5.827

Results for 1 year Critical Storm Duration. Lowest mass balance: 100.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute summer	P10	232	40.777	0.277	43.0	135.3074	0.0000	OK
480 minute summer	P4	304	53.713	0.263	53.2	209.6465	0.0000	OK
480 minute summer	P3	312	45.699	0.229	43.9	186.8840	0.0000	OK
360 minute summer	P12	240	56.227	0.327	9.2	26.6828	0.0000	OK
240 minute summer	P13	172	55.556	0.276	37.0	80.3252	0.0000	OK
480 minute summer	P2	320	56.079	0.279	97.8	450.2993	0.0000	OK
480 minute summer	P1	304	51.274	0.274	51.8	204.0476	0.0000	OK
240 minute summer	P7	160	50.946	0.196	14.9	32.2145	0.0000	OK
600 minute summer	P6	405	47.789	0.289	64.5	367.1833	0.0000	OK
960 minute summer	P8A	615	49.099	0.099	19.3	180.8994	0.0000	OK
240 minute summer	P11	148	59.227	0.127	25.2	52.6785	0.0000	OK
480 minute summer	P5	328	50.939	0.289	37.6	180.8302	0.0000	OK
600 minute summer	P9	405	48.054	0.304	51.0	290.4890	0.0000	OK
480 minute summer	SW3	320	49.596	0.246	12.9	48.0885	0.0000	OK
240 minute summer	SW4	156	54.614	0.114	13.1	28.8877	0.0000	OK
360 minute summer	SW6	256	51.688	0.388	30.3	91.0434	0.0000	OK
960 minute summer	SW7	780	67.142	0.162	9.5	14.8467	0.0000	OK
360 minute summer	SW5	248	57.281	0.331	15.6	47.2825	0.0000	OK
240 minute summer	SW1	164	50.570	0.270	19.8	38.5050	0.0000	OK
120 minute summer	SW2	78	53.016	0.066	22.2	9.2312	0.0000	OK
960 minute summer	Perm1	660	68.153	0.203	47.1	434.6555	0.0000	OK
240 minute summer	P8	172	46.695	0.245	26.9	66.9025	0.0000	OK
120 minute summer	SW8	76	52.342	0.292	50.8	32.5262	0.0000	OK
240 minute summer	SW9	156	52.725	0.225	57.2	95.1768	0.0000	OK

US Node	DS Node	Outflow (l/s)	Discharge Vol (m³)
P10		13.8	237.2
P4		18.4	364.9
P3		13.5	288.7
P12		2.4	53.5
P13		7.6	150.8
P2		34.3	666.7
P1		17.3	357.2
P7		4.7	57.6
P6		26.2	655.7
P8A		6.2	196.0
P11		11.8	91.1
P5		8.8	250.8
P9		13.1	414.4
SW3		3.4	95.1
SW4		4.8	50.2
SW6		7.1	183.5
SW7		9.4	454.9
SW5		3.4	92.0
SW1		5.4	80.4
SW2	P5	15.5	51.9
Perm1	SW7	9.5	466.4
P8		14.5	145.2
SW8	P8	27.3	107.7
SW9	P6	36.1	265.0

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 99.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute winter	P10	352	41.677	1.177	109.0	717.0564	0.0000	OK
360 minute winter	P4	352	54.566	1.116	155.0	1047.0620	0.0000	OK
480 minute winter	P3	464	46.429	0.959	103.3	898.1678	0.0000	OK
360 minute winter	P12	352	57.066	1.166	23.2	156.2132	0.0000	OK
360 minute winter	P13	352	56.545	1.265	75.5	504.8858	0.0000	OK
480 minute winter	P2	472	56.960	1.160	243.1	2107.3750	0.0000	OK
360 minute winter	P1	352	52.156	1.156	150.8	1025.8840	0.0000	OK
240 minute winter	P7	240	51.634	0.884	41.1	185.1721	0.0000	OK
720 minute winter	P6	705	48.746	1.246	128.2	1854.5530	0.0000	OK
480 minute summer	P8A	376	49.311	0.311	103.8	589.5206	0.0000	OK
120 minute summer	P11	90	59.574	0.474	156.0	215.5431	0.0000	OK
600 minute winter	P5	585	51.812	1.162	72.9	877.8987	0.0000	OK
720 minute winter	P9	705	48.974	1.224	104.8	1376.6480	0.0000	OK
480 minute winter	SW3	472	50.065	0.715	30.4	269.8376	0.0000	OK
180 minute winter	SW4	180	54.822	0.322	44.2	149.2587	0.0000	OK
480 minute winter	SW6	472	52.323	1.023	63.0	556.6960	0.0000	OK
1440 minute winter	SW7	840	67.172	0.192	10.0	19.4409	0.0000	OK
480 minute winter	SW5	472	57.828	0.878	32.0	285.1809	0.0000	OK
360 minute winter	SW1	352	51.063	0.763	40.4	254.4168	0.0000	OK
60 minute winter	SW2	62	53.291	0.341	117.0	133.4213	0.0000	OK
1440 minute winter	Perm1	1410	68.685	0.735	75.5	1950.9280	0.0000	OK
600 minute winter	P8	585	47.437	0.987	28.0	337.5359	0.0000	OK
120 minute winter	SW8	118	53.086	1.035	169.5	334.7217	0.0000	OK
120 minute summer	SW9	120	53.133	0.633	297.3	605.6307	0.0000	OK

US Node	DS Node	Outflow (l/s)	Discharge Vol (m³)
P10		14.3	400.1
P4		19.3	527.1
P3		14.1	457.0
P12		2.5	79.3
P13		7.6	257.2
P2		36.0	1150.8
P1		18.0	496.9
P7		5.0	113.0
P6		27.5	1159.8
P8A		15.0	465.0
P11		29.3	314.0
P5		9.1	357.1
P9		13.6	593.8
SW3		3.4	118.9
SW4		5.8	127.3
SW6		7.1	256.9
SW7		9.7	767.7
SW5		3.4	122.9
SW1		5.5	162.4
SW2	P5	20.0	196.9
Perm1	SW7	10.0	781.3
P8		15.0	591.0
SW8	P8	28.0	495.9
SW9	P6	50.0	850.1

Results for 100 year +45% CC Critical Storm Duration. Lowest mass balance: 99.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
480 minute winter	P10	464	41.988	1.488	113.4	969.1225	0.0000	OK
480 minute winter	P4	472	54.879	1.429	161.2	1416.2210	0.0000	OK
600 minute winter	P3	585	46.706	1.236	111.4	1214.4040	0.0000	OK
480 minute winter	P12	472	57.318	1.418	24.1	212.2181	0.0000	OK
480 minute winter	P13	472	56.882	1.602	78.6	698.0696	0.0000	OK
480 minute winter	P2	472	57.300	1.500	313.0	2843.5550	0.0000	OK
480 minute winter	P1	472	52.478	1.478	156.9	1388.1100	0.0000	OK
360 minute winter	P7	352	51.870	1.120	39.0	252.1730	0.0000	OK
960 minute winter	P6	930	49.117	1.617	132.3	2544.6590	0.0000	OK
600 minute summer	P8A	495	49.414	0.414	113.7	795.8253	0.0000	OK
120 minute summer	P11	104	59.740	0.640	204.1	302.4618	0.0000	OK
960 minute winter	P5	930	52.142	1.492	68.0	1200.3560	0.0000	OK
960 minute winter	P9	930	49.332	1.582	109.5	1882.3670	0.0000	OK
720 minute winter	SW3	705	50.206	0.856	28.6	369.7329	0.0000	OK
240 minute winter	SW4	236	54.890	0.390	46.8	207.5174	0.0000	OK
720 minute winter	SW6	705	52.507	1.207	59.9	765.3816	0.0000	OK
1440 minute winter	SW7	750	67.169	0.189	10.0	18.9427	0.0000	OK
720 minute winter	SW5	705	57.989	1.039	30.2	393.0166	0.0000	OK
360 minute winter	SW1	360	51.205	0.905	52.0	351.4416	0.0000	OK
60 minute winter	SW2	65	53.374	0.424	156.6	197.4305	0.0000	OK
1440 minute winter	Perm1	1410	68.961	1.011	99.1	2739.6870	0.0000	OK
960 minute winter	P8	855	47.682	1.232	28.0	449.7381	0.0000	OK
120 minute winter	SW8	120	53.295	1.245	221.7	476.8415	0.0000	OK
120 minute winter	SW9	128	53.264	0.764	341.7	860.8084	0.0000	OK

US Node	DS Node	Outflow (l/s)	Discharge Vol (m³)
P10		14.3	527.3
P4		19.3	691.5
P3		14.1	583.1
P12		2.7	102.4
P13		7.6	311.9
P2		36.0	1269.9
P1		18.0	656.3
P7		5.0	151.9
P6		27.5	1584.5
P8A		15.0	543.9
P11		29.3	408.4
P5		9.1	556.7
P9		13.6	816.9
SW3		3.4	167.9
SW4		5.8	143.7
SW6		7.1	360.0
SW7		9.6	791.5
SW5		3.4	172.3
SW1		5.5	165.6
SW2	P5	20.0	263.8
Perm1	SW7	10.0	805.3
P8		15.0	846.1
SW8	P8	28.0	537.1
SW9	P6	50.0	940.3