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Blackstone Wastewater Treatment Works - Drainage Strategy

1. Introduction

WRc was commissioned by Southern Water to produce a drainage strategy for upgrades proposed to Blackstone Wastewater Treatment Works (WTW) in support of planning application WSCC/012/24. This technical note outlines the proposed surface water drainage strategy for the development.

To compile the report the following guidance was used. They are referenced in full at the end of this report.

- West Sussex LLFA Policy for the Management of Surface Water (West Sussex LLFA, 2018)
- The SuDS Manual (Woods Ballard, 2015)
- National Planning Policy Framework (Department for Levelling Up, Housing and Communities, 2023)
- Rainfall runoff management for developments (Kellagher, 2013)

- Flood risk assessments: climate change allowances (Environment Agency, 2016)

2. Drainage Overview

As described in the associated planning statement (February 2024), Southern Water is to upgrade the existing Blackstone WTW to comply with an enhanced discharge permit for phosphorus and iron. The proposed Motor Control Centre (MCC) kiosk would contain the necessary electronic control equipment which allow the operation of the new plant and equipment.

Figure 1 shows an overview of the proposed construction at the WTW (for details see 48560-ECE-XX-XX-DR-C-0012_P01). As per the supporting statement of the planning application, application WSCC/012/24 pertains only to the construction of the MCC Kiosk (7.75 m long x 3.3 m wide) and the associated hardstanding (approximately 35 m^2) (Southern Water, 2024). An assessment of the impermeable areas of the wider development, not covered by this planning application, is also provided to better establish the overall impact on surface water run-off. This includes an assessment of the on-site drainage to cope with run-off from the "dirty" and clean areas of the development.

The impermeable area, totalling an area of (425 m^2), of which 65 m^2 is relevant to this planning application, will be drained via two methods which are summarised here. An overview of the three impermeable areas is shown in Figure 1.

- **Impermeable Area A** – Comprises the MCC Kiosk (rounded to 30 m^2) and the associated hardstanding (35 m^2), and is the impermeable area covered by this planning application. The area is to be constructed on existing grassland and is to be drained to the surrounding grassland.

- **Impermeable Area B** – Comprises ferric dosing, chemical delivery, lamella and the associated hardstanding and has been estimated at 225 m^2. This area is classified as "dirty" and is required by Southern Water's standards to be fully treated. Therefore, this area will drain to the WTW and will not drain to surrounding permeable areas.

- **Impermeable Area C** – Comprises a new road (135 m^2). This is again to be drained to surrounding grassland and does not directly relate to this planning application but is included here for information.

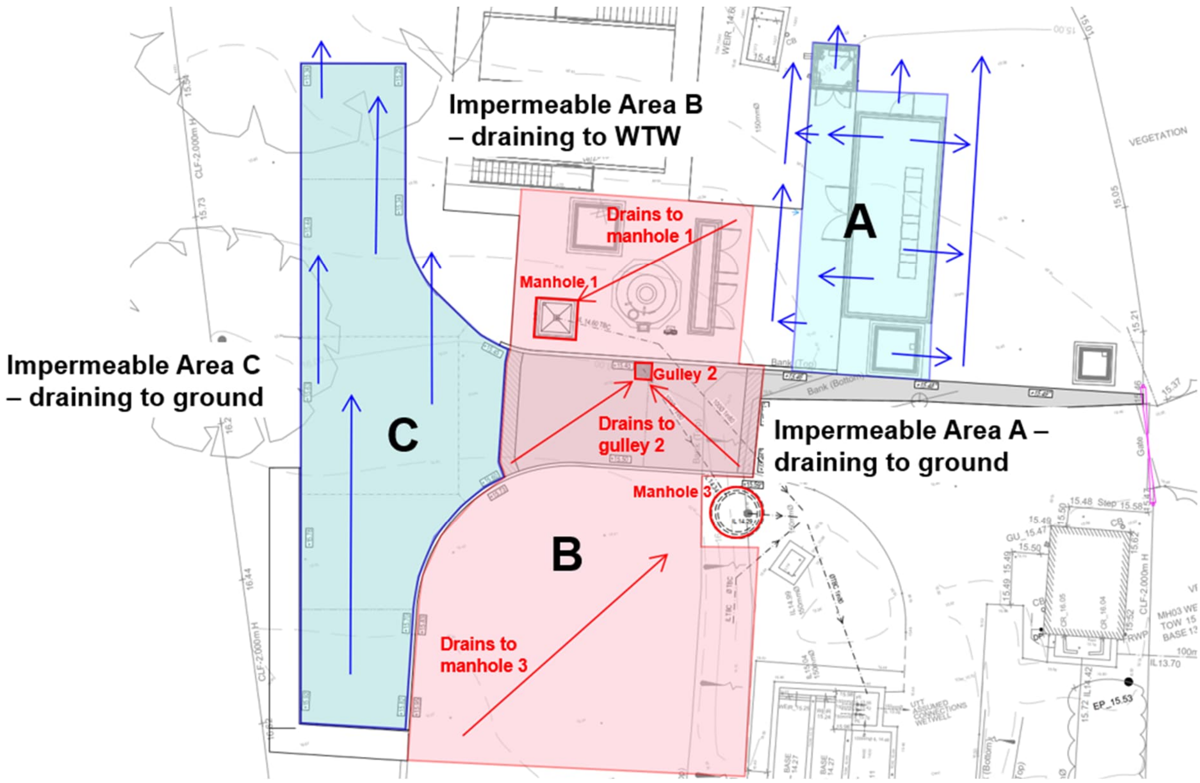

Figure 1 Overview of the proposed site, with the expected flow pathways.

Based on run-off assessment using existing LiDAR (Light Detection and Ranging), the impermeable area from the proposed development is to drain to an unnamed ditch running along the northern boundary of the site. North of this is the Chess Stream (also known in this area as Cutlers Brook) and this is the receiving watercourse to which the treated sewage effluent is discharged. This is shown in Figure 2. At its closest point, the development is situated 8.5 m south of the unnamed ditch, and around 36 m south of Cutlers Brook/Chess Stream. The level of the development site is 15 mAOD, the base of the unnamed ditch is 14.10 mAOD, and Chess Stream is situated at 13.3 mAOD (values from site topographical survey). It is not known how this ditch interacts with Chess Stream, though topographical surveys and LiDAR indicate that it drains to the west and subsequently north to rejoin Chess Stream around 850 m from the site.

An ordinary watercourse also runs along the eastern boundary of the site from south to north, receiving run-off from the adjacent Blackstone Lane. It is culverted for a short distance under the access track to Blackstone WTW. Based on site investigations, this ordinary watercourse is thought to enter Cutlers Brook/Chess Stream north-east of the site. It is unclear if the unnamed ditch to the north of the site interacts with this ordinary watercourse.

The development is to be constructed on existing grassland. Therefore, the increase in impermeable area draining to permeable is 200 m^2 (areas A and C) and the increase in area draining to the WTW is 225 m^2 (area B).

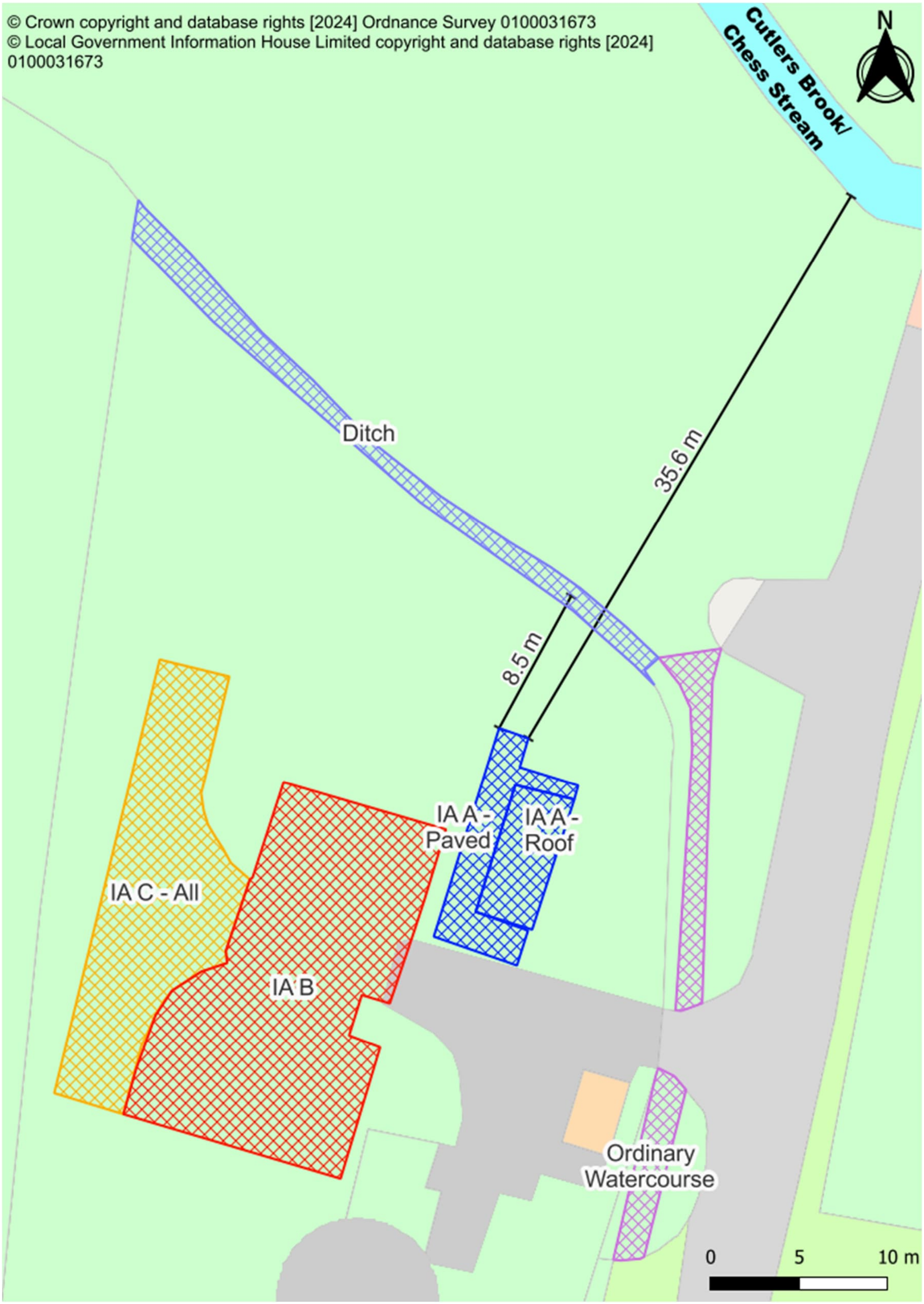

Figure 2 Overview of proposed development and Cutlers Brook/Chess Stream.

3. Greenfield Run-Off Rate

The total area to be developed is 425 m^2, or approximately 0.04 ha. Using the FEH Greenfield run-off rate method as set out in Rainfall run-off management for urban developments (Flood and Coastal Erosion Risk Management Research and Development Programme & Environment Agency, 2021) and FEH22 rainfall data (CEH, 2024) (Table 3.1), Greenfield run-off rates have been calculated and are shown in Table 3.2.

Table 3.1 Summary of FEH22 factors used

National Grid Reference	SAAR	BFIHOST19	PROPWET	Hydrological Region
TQ 24281 16599	829	0.31	0.34	7

Table 3.2 Summary of Greenfield Run-Off Rates

	QBAR (l/s)	100% AEP (1-yr return period – l/s)	3.3% AEP (30-yr return period – l/s)	1% AEP (100-yr return period – l/s)
Impermeable Area A (65 m^2)	0.051	0.044	0.118	0.124
Impermeable Area B (225 m^2)	0.178	0.151	0.409	0.430
Impermeable Area C (135 m^2)	0.107	0.091	0.245	0.258
Total (425 m^2)	**0.336**	**0.286**	**0.773**	**0.813**

Note: Due to rounding, total row may not equal totals of A-C

Furthermore, the 6-hour, 1% Annual Exceedance Probability (AEP) discharge volume ($Vol_{100yr6hr}$) for Impermeable Area A has been calculated to be 4.61 m^3, based on an expected 70.89 mm of rainfall and 65 m^2 area. The $Vol_{100yr6hr}$ for Impermeable Area C has been calculated to be 9.57 m^3, based on an expected 70.89 mm of rainfall and 135 m^2 area. Impermeable Area B (225 m^2) discharge volume is to undergo full treatment, but run-off would be expected to generate 15.95 m^3 run-off (or 23.1 m^3 with a 45% climate change uplift).

4. Run-off rates

Using FEH22 data (CEH, 2024), the following peak 5-minute run-off rates for the proposed development were obtained (Table 4.1). These exceed the Greenfield Run-Off Rate from the existing area. The climate change allowance of 45% is the upper end allowance for "Arun and Western Streams" (Environment Agency, 2016). A coefficient for volumetric run-off of 1.0 has been used.

Table 4.1 5-minute peak run-off rates for proposed impermeable areas in development

	50% AEP (2-yr return period)	1% AEP (100-yr return period)	1% AEP + 45% climate change
5-minute rainfall	4.60 mm	13.62 mm	19.75 mm
Impermeable Area A (draining to permeable area)	0.299 m^3	0.885 m^3	1.284 m^3
	0.997 l/s	2.951 l/s	4.279 l/s
Impermeable Area B (draining to inlet wet well)	1.035 m^3	3.065 m^3	4.444 m^3
	3.450 l/s	10.215 l/s	14.813 l/s
Impermeable Area C (draining to permeable area)	0.621 m^3	1.839 m^3	2.666 m^3
	2.070 l/s	6.129 l/s	8.888 l/s
Total	**6.517 l/s**	**19.295 l/s**	**27.979 l/s**

5. Capacity of Existing Drainage

Impermeable Area B is to be drained to the existing sewer system on site. This sewer system is shared with the humus tank desludge and eventually drains to the site inlet pumping station, which passes flow forward to treatment. Figure 3 shows the proposed drainage of this impermeable area. Values are derived from proposed design drawings and existing as-built drawings.

There are three sub-areas of impermeable area B which drain to separate manholes or gulleys. (Manhole 1, Gulley 2, Manhole 3). These then drain to a proposed manhole (Manhole 4), before draining to the humus desludge chamber. At the time of writing, the diameter of the pipe between Manhole 4 and the humus desludge chamber has not been confirmed, therefore a

150 mm pipe has been assumed, likewise the size of Manhole 1 has been inferred to be 1.5 m x 1.5 m. The humus desludge chamber then drains to the inlet pumping station.

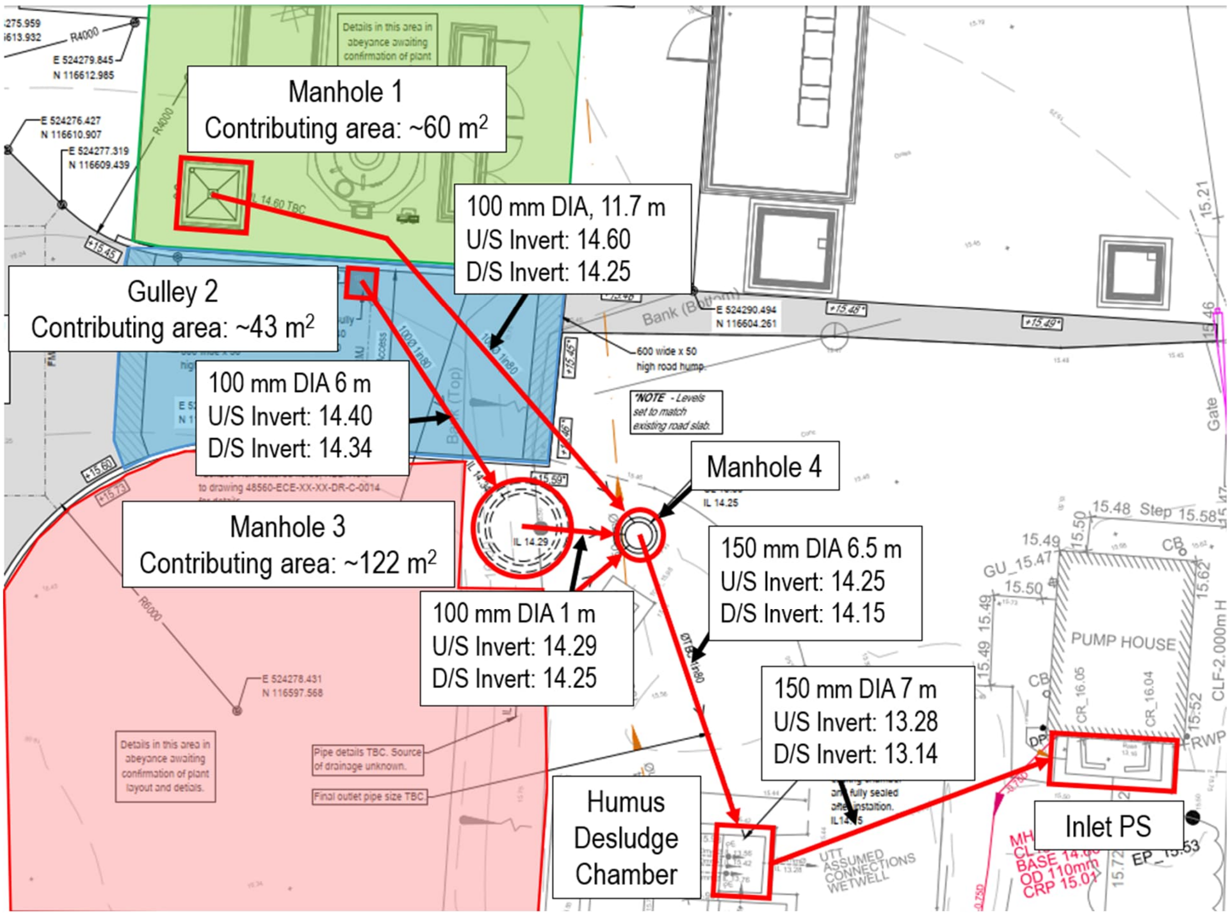

Figure 3 Overview of impermeable area B drainage.

As part of a previous scheme, a SAFF plant with an impermeable area of approximately 48 m^2 has also been installed, this drains to the humus desludge chamber and during a 1% AEP + 45% storm is expected to have a run-off of ~ 3.1 l/s.

The humus desludge and sludge holding tank supernatant return also return to the humus desludge chamber. However, these are manually operated and would not be operated during an extreme rainfall event and therefore no additional flows require consideration.

Table 5.1 shows estimated capacity of the pipes outlined in Figure 3 and the estimated flow from their upstream contributing areas. Pipe full capacity has been calculated by Infoworks ICM version 2021.9 based on the values presented in Figure 3, with run-off values shown in Table 4.1 in the previous section.

Table 5.1 Summary of drainage capacity

Upstream manhole	Downstream manhole	Sewer Diameter	Gradient	Capacity (l/s)	Upstream Incoming Flow	1% AEP Flow + 45%	Total Expected Flow
Manhole 1	Manhole 4	100	1 in 33	**9.1**	-	4.0	**4.0**
Gulley 2	Manhole 4	100	1 in 100	**5.2**	-	2.8	**2.8**
Manhole 3	Manhole 4	100	1 in 25	**10.5**	2.8	8.0	**10.8**
Manhole 4	Humus Desludge Chamber	150	1 in 73	**19.2**	14.4	-	**14.8**
Humus Desludge Chamber	Inlet Wet Well	150	1 in 50	**21.9**	14.4	3.2*	**18.0**
Inlet Wet Well	Treatment	Rising main	Rising main	**5.9 (pump)**	18.0**	-	**18.0**

*** from SAFF plant.**
**** does not include any foul flow from network.**

Table 5.1 shows that most site gravity drainage will be able to accommodate the estimated run-off from the proposed impermeable areas, even in the 1% AEP + 45% scenario. The exceptions are a slight incapacity in the gravity sewer between Manhole 3 and Manhole 4, which has a shortfall of 0.1 l/s, and the pump rate of the inlet pumping station, whose capacity falls significantly short of the incoming flow. This does not include any flow from the network.

Table 5.2 Storage required to mitigate shortfalls, and storage available prior to flooding occurring.

Shortfall Location	Capacity (l/s)	Expected Flow (l/s)	Shortfall (l/s)	Storage Required (m^3)	Storage Available (m^3)
Manhole 3	10.5	10.8	0.3	**0.09**	**2.0**
Inlet Wet Well	5.9	18.0	12.1	**3.63**	**19.7**

Table 5.2 shows that shortfalls in capacity during a peak 5-minute flow event are mitigated against by available storage in the wet well and the WTW's sewers, and that there should be no escape of flow during a 1% AEP + 45% climate change peak flow scenario. However, this excludes any contribution from the existing network upstream of Blackstone WTW. Based on the remaining available storage (16.1 m^3) and the duration and depth of rainfall of the storm (19.75 mm), the treatment works should have sufficient storage to accommodate a 5-minute average flow of around 50 l/s from the network without flooding. The sewer network upstream is small (with 150 mm diameter pipes) and is characterised as foul-only in Southern Water's

sewer records, therefore flows in excess of this are unlikely. Investigation beyond this is considered outside the scope of this work.

Though the WTW is expected, based on this assessment, to have the capacity to cope with the additional impermeable area, an overland run-off simulation was run to confirm run-off from any potential flooding does not impact on vulnerable or essential infrastructure (Annex 3 (Department for Levelling Up, Housing and Communities, 2023)). Figure 4 shows run-off from flooding during an extreme 1% AEP + 45% climate change event. Any run-off is predicted to pool around the humus tank, with no flood depths of greater than 5 cm around any of the existing or proposed buildings. Further run-off is predicted to collect in the ditch that surrounds the site. Figure 4 has been calculated based on 1 m DTM (digital terrain model) LiDAR data. A small amount of flooding is shown on the adjacent road, but this is thought to be a slight misalignment of LiDAR data to background mapping, and in actuality this flooding will be contained within the ditch. No flooding was predicted in the 3.33% AEP event.

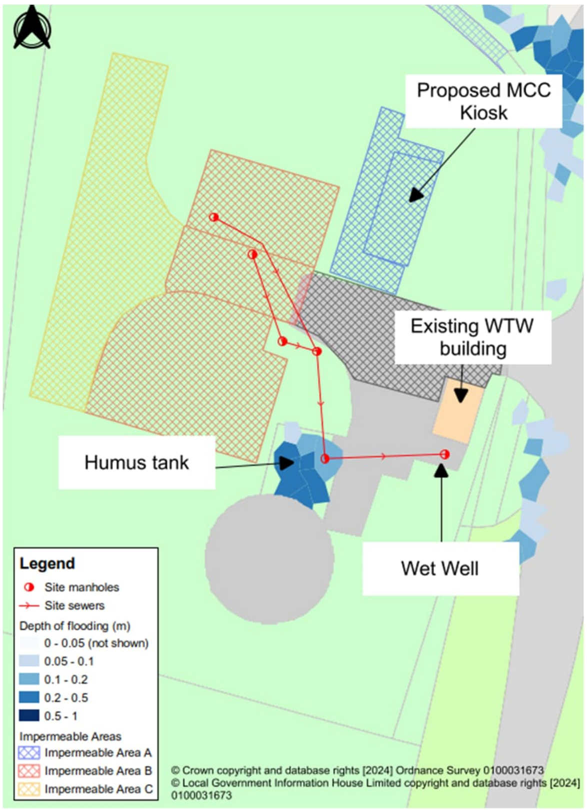

Figure 4 Likely areas of flooding, showing that flooding is not expected to impact on critical or vulnerable infrastructure within or outside of the WTW.

This section has shown that there is not expected to be any issues with site sewer capacity during a 3.33% AEP and below event, and the site should have the storage to accommodate the increase in impermeable area, and up to 50 l/s of flow from the network during a 5-minute 1% AEP + 45% climate change event. Should any flooding occur, run-off is mostly expected to be contained to the ordinary watercourse that runs to the east of the site, and to the ditch to the north of the site and is not expected to cause flooding to on-site vulnerable or critical infrastructure (such as buildings).

6. Ground Investigation

A February 2024 Ground Investigation report produced for Blackstone WTW by Structural Soils (sample WS01) indicated that the top 0.15 m of soil is comprised of silt with sand and gravel. Between 0.15 m and 1.5 m depth the soil is comprised of sandy, gravelly, clay in different ratios, with depths below this being Weald Clay.

7. Drainage to Permeable Area

Impermeable Areas A and C are proposed to drain to the permeable area surrounding the proposed development. Ground investigations undertaken for the overall site in 2024 indicate that below a depth of 15 cm gravelly silt, the site lies on clay alluvium down to at least 1.5 m. The permeability of clay makes the site unsuitable for infiltration. Therefore, any SuDS (Sustainable Drainage System) should be designed to provide attenuation and storage at- or near the surface.

The 1-year Greenfield run-off rate for the entire development site (425 m^2) has been calculated to be 0.336 l/s, with predicted run-off from Impermeable Areas A and C reaching 13.17 l/s in a 1% AEP + 45% climate change event (4.28 l/s and 8.89 l/s respectively). The neighbouring Cutlers Brook means that attenuating flows with a controlled discharge is geographically possible, but the very low Greenfield run-off rate required (<1 l/s) means it is not technically possible to achieve with a flow control device (HR Wallingford, 2024). Construction of attenuation storage with a suitable flow control device is therefore not feasible.

The development is likely to increase run-off to permeable areas by around 200 m^2 (compared to the existing drainage of the site). Based on design drawings and the site topography, run-off from Impermeable Areas A and C is likely to drain first into the 500 mm deep, ~ 37 m wide ditch to the north of the site. Assuming an approximate V-shape to the ditch, and it being ~ 1 m wide (as indicated by topographical survey), this provides 18.5 m^3 of attenuation storage. After this ditch is filled, overland flows would run-off to Cutlers Brook a further ~ 20-30 m north.

As per Section 3.3 of the SuDS Manual, (Woods Ballard, 2015), the 1% AEP 6-hour rainfall event can be used to derive the volume control for the site. This amounts to 14.18 m^3 for Impermeable Areas A and C. Therefore, it is thought that the existing ditch would allow flows to be attenuated sufficiently prior to any entry into Cutlers Brook.

The development is sited outside of flood risk zones 2 and 3 and therefore draining this small impermeable area directly to the surrounding permeable area should not increase flood risk, particularly as prior to discharge into Cutlers Brook, flows would be attenuated in the intervening ditch. The proposed development is sited at 15 mAOD, with Cutlers Brook situated at around 13.3 mAOD.

8. Water Quality Assessment

The hardstanding which is currently proposed to drain to permeable area (Impermeable Areas A and C) is unlikely to see more than 1-2 traffic movements per day and Impermeable Area A will likely only see occasional foot traffic when operational changes and checks are made in the MCC Kiosk.

Based on the CIRIA Simple Index Approach (Woods Ballard, 2015), it is therefore likely that the pollution hazard level of the impermeable areas is on the lower side of the "Low Traffic Roads" land use (as there will be very few traffic movements, and limited pedestrian movements), and is a "Low" pollution hazard level (Table 8.1). Based on this, a swale would be an appropriate SuDS feature, which is in essence the feature provided in the form of the ditch to the north of the site.

Table 8.1 Pollution Hazard Indices

Type of run-off	Pollution Hazard Level	TSS	Metals	Hydrocarbons
Individual property driveways, residential car parks, low traffic roads (e.g. cul de sacs, homezones and general access roads) and non-residential car parking with infrequent change e.g. schools, offices i.e. < 300 traffic movements/day	Low	0.5	0.4	0.4

Table 8.2 SuDS mitigation indices

Type of SuDS component	TSS	Metals	Hydrocarbons
Swale	0.5	0.6	0.6

Given the low discharge and volumes involved (14.18 m^3 in a 1% AEP, 6-hour storm), sufficient filtration of solids and sediment would likely be provided by both the ditch situated between Cutlers Brook and the WTW, and also by the intervening ~30 m of grassland.

9. Flood Risk

The proposed development lies outside of Flood Risk Zones 2 and 3 (Figure 5) and given the volumes of discharge and the attenuation provided is unlikely to significantly impact downstream of the point of discharge.

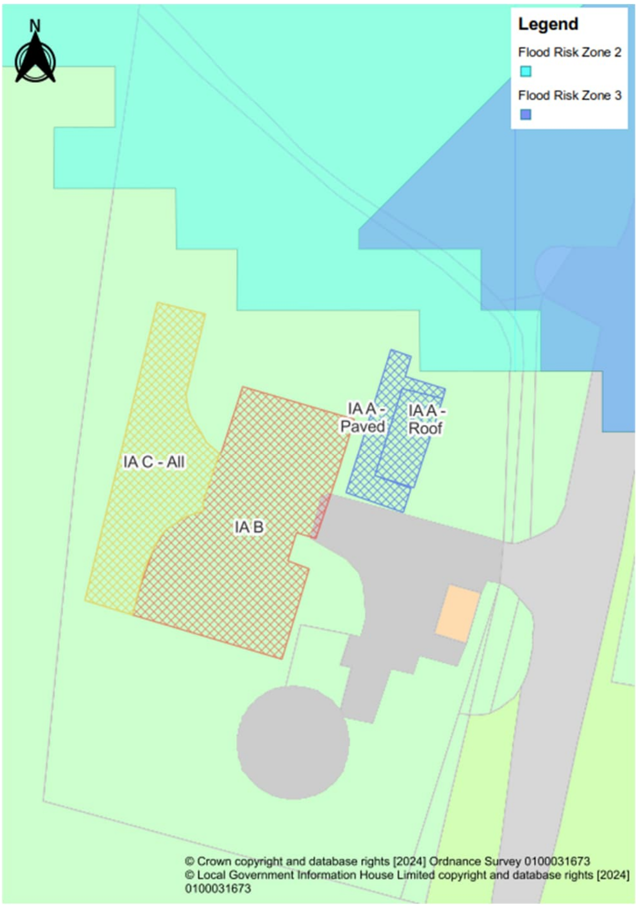

Figure 5 Flood Zones 2 and 3 in relation to proposed development

10. Conclusion

This technical note has been produced to outline the drainage strategy for the proposed installation of a new MCC kiosk at Blackstone WTW. Planning application WSCC/012/24 pertains only to the new MCC kiosk at the WTW (approximately 30 m^2), but the drainage strategy also includes the associated hardstanding, new Lamella and Ferric Dosing areas. These are not covered by this planning application, being allowed under Southern Water's permitted development rights.

The overarching surface water strategy is to drain approximately 225 m^2 of impermeable area into the existing treatment works (Impermeable Area B), and the remaining 200 m^2 would drain to permeable ground surrounding the development (Impermeable Area A and C).

Analysis of the existing and proposed drainage indicates that the existing drainage system has capacity to cope with surface run-off from impermeable area B up to a 3.33% AEP event without flooding. During a 5-minute 1% AEP + 45% climate change event, the site has the capacity to cope with up to 50 l/s arriving from the network and the run-off from Impermeable Area B without flooding. Given the small size of the network upstream and its characterisation as foul-only this is expected to be sufficient. Any potential flooding would be expected to drain to the ordinary watercourse to east of the site, and to a ditch to the north of the site and would not impact on any vulnerable or critical infrastructure on- or off-site.

The underlying geology of the site is Weald Clay, which means that using infiltration SuDS to mitigate the impact of the run-off from Impermeable Areas A and C is not practical. The volume generated by Impermeable Areas A and C during a 1% AEP 6-hour storm is expected to be 14.2 m^3. The ditch running to the north of the site is, based on topographical survey, estimated to provide 18.5 m^3 of storage. It is suggested that this attenuation storage, together with the ~ 30 m of intervening grassland between the development and the receiving watercourse (Cutlers Brook/Chess Stream) will provide sufficient attenuation of run-off from Impermeable Areas A and C and will not increase the risk of flooding to Cutlers Brook/Chess Stream.

Topographical survey and LiDAR indicate that the ditch drains to the west and subsequently north, eventually reaching Chess Stream ~ 850 m from the site. Based on site investigations, the ordinary watercourse to the east of site drains from south to north, discharging into Chess Stream north-east of the WTW and takes run-off from Blackstone Lane. Interaction between the ditch to the north and the ordinary watercourse to the east has not been confirmed.

The expected run-off from Impermeable Areas A and C (hardstanding and roof) has been deemed to have a “Low” pollution hazard index, based on the CIRIA Simple Index Approach (Woods Ballard, 2015). Therefore, it is thought that the settlement and filtration provided by the ditch (in essence a swale) and the intervening grassland provides sufficient qualitative treatment of run-off.

Based on the evidence provided in this report, it is suggested that no further mitigation of run-off is necessary from the proposed development.

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