



147 – 149 College Lane
Hurstpierpoint

Flood Risk Assessment & Drainage Strategy
For
Earlswood Homes

Document Control Sheet

147 – 149 College Lane

Hurstpierpoint

Earlswood Homes

This document has been issued and amended as follows:

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1.0 Introduction

- 1.1 This Flood Risk Assessment (FRA) and drainage strategy has been produced by Motion on behalf of their client, Earlswood Homes. It supports the proposed development of 7no. residential dwellings plus parking, access and landscaping on the land behind 147 – 149 College Lane, Hurstpierpoint. The layout of the proposed development can be seen in [Appendix A](#).
- 1.2 According to the Environment Agency's (EA's) Flood Map for Planning the site is located within Flood Zone 1. However, because the EA's Risk of Flooding from Surface Water (RoFSW) mapping shows a small area of 'Low' surface water flood risk on the site and an area of 'High' surface water flood risk in College Lane, an FRA has been produced that will review surface water flood risk, as well as flooding from other sources.
- 1.3 Although the development is minor in planning terms, it is a 'non-major' development in flood risk terms and, as such, a drainage strategy is also required to demonstrate how the development will manage and discharge surface water generated in all rainfall events up to and including the 1 in 100-year + 45% for climate change.
- 1.4 Therefore, this FRA and drainage strategy has been produced to discuss the flood risks to the proposed development, from all sources. This FRA and drainage strategy will also define how the development will manage its surface water and foul sewage so that the development does not increase flood risk in the area or to neighbouring properties/land.
- 1.5 This FRA and drainage strategy follows the guidance set out in:
 - » Mid-Sussex District Council's Flood Risk and Drainage Information Check List and Policy DP41 of Mid-Sussex District Council's Local Plan.
 - » West Sussex's Policy for the Management of Surface Water
 - » The National Planning Policy Framework (NPPF).
 - » The Planning Practice Guidance (PPG) to the National Planning Policy Framework.
 - » The CIRIA SuDS Manual 2015 (C753).
 - » The Environment Agency Rainfall Runoff Management for Developments.
 - » The Non-Statutory Technical Standards for SuDS (NSTSfs).
- 1.6 This FRA and drainage strategy report pertains only to the drainage strategy for the development. It does not provide details of how the site will be drained during the construction phase. This report is also not a drainage verification report, which can only be produced post-construction.
- 1.7 Similarly, this report does not provide information on how the drainage infrastructure will be protected during the construction phase of the project. The provision of this information is the responsibility of the appointed contractor.

2.0 Site Description

Table 2.1 – Site Summary

Site Name	Land behind 147 – 149 College Lane
Location	Hurstpierpoint, East Sussex, BN6 9AF
Grid Reference	TQ 29238 16940
Site Area	6,206m ² (0.621 ha)
Development Type	Residential development of 7no. dwellings plus access, parking and landscaping.
Flood Zone	1
Surface Water Flood Risk	Very Low to Low
Local Water Authority	Southern Water
Local Planning Authority	Mid-Sussex District Council (MSDC)
Lead Local Flood Authority	MSDC

Site Location and Description

- 2.1 The land behind 147 – 149 College Lane is an irregularly shaped plot of land located to the northeast of Hurstpierpoint in West Sussex. It is in a semi-rural area, with College Lane bounded by residential properties to the south of the site, but the area to the north comprising of open fields. The campus of Hurstpierpoint College lies 600m to the north of the site. A site location plan can be viewed in [Appendix B](#).
- 2.2 The site is currently greenfield and used intermittently as arable/grazing land for sheep. The land cover consists of low-level grass and a sparse covering of small trees. There are larger trees (oak, ash, fir, holly and willow) on the site's eastern boundary and some oak and birch on the site's south-eastern boundary.
- 2.3 The southern boundary of the site borders onto the back garden of number 143 College Lane. The northern and eastern boundaries of the site are bounded by open fields. The western boundary of the site leads onto College Lane or is bounded by the rear gardens of numbers 147 – 149 College Lane.
- 2.4 The access to the site will be from the south-western corner, which leads onto College Lane. The northern/north-western part of the site does not extend back to College Lane, although the landholding does, thus there is potential for services to cross this area of land if required.

Topography

- 2.5 A formal topographic survey of the site has not been carried to date. In lieu of this a review of Light Detection and Radar (LiDAR) data was carried out to understand the overall levels and gradients on the site.
- 2.6 The site slopes from the south-southwest to the north-northeast. The highest topographic levels are on the southern boundary, which are between 38 metres Above Ordnance Datum (mAOD) and 39 mAOD. The lowest site levels are in the northeast of the site and fall to 32.5 mAOD.

- 2.7 This elevation difference across the site, which is just over 100m from its southern boundary to the northern boundary, gives an average site gradient of 1 in 20.
- 2.8 The LiDAR elevation plan can be seen in [Appendix C](#).

Geology

- 2.9 The 1:50,000 British Geological Survey (BGS) online Geoindex Mapping identifies that the solid geology underlying the site is Weald Clay Formation (see [Appendix D](#)), which are characterised by thinly bedded mudstones (shales) and mudstones with subordinate siltstones. No superficial deposits are listed in the 1:50,000 BGS online Geoindex Mapping, but Defra's Magic Map online viewer¹ lists the superficial geology as predominantly being 'seasonally wet, slightly acid but base-rich loamy and clayey soils' with impeded drainage. This was experienced during the site visit with the surface soils being wet underfoot and seeming to resist infiltration of surface water.
- 2.10 There are no BGS boreholes on site, but boreholes at a similar location within the Weald Clay relative to the scarp-slope of the South Downs show that the Weald Clay extends to depth (over 50m).

Infiltration Testing

- 2.11 No infiltration testing has been carried out to date on the basis that the Weald Clay, which extends to depth below ground, would not be conducive to infiltration.
- 2.12 This follows the advice of literature values for infiltration in clay soils (BS 7533-13:2009), which suggests that they have infiltration rates typically of the order of 1×10^{-8} to 1×10^{-10} . Table 25.1 of CIRIA C753 supports this by suggesting infiltration rates of less than 3×10^{-8} are possible.
- 2.13 Infiltration rates lower than 1.0×10^{-6} m/s are not generally conducive to infiltration and usually only System B (partial infiltration) infiltration features are possible, if at all. If infiltration rates are indeed as low as 1×10^{-8} , then infiltration would not be possible on site.
- 2.14 In reflection of the above constraints, the drainage strategy will not pursue infiltration as a means of surface water discharge from the development.

Hydrogeology

- 2.15 Groundwater Source Protection Zones (SPZ's) are defined around groundwater abstraction sources such as wells, boreholes and springs that are used for public drinking water supply.
- 2.16 SPZ's show the risk of contamination to groundwater from any activities that might cause pollution in the area. The closer the activity to the source of abstraction, the greater the risk. The maps show three main zones; inner – Zone 1; outer – Zone 2 and; total catchment – Zone 3.
- 2.17 Certain geologies can contain fractures and pathways that allow groundwater to move more quickly than is defined above, so these should be used as a guideline and if it is suspected that there is potential for groundwater pollution even though a site is in a lower risk SPZ, professional advice should be sought from a geotechnical consultant.
- 2.18 Defra's Magic Map was reviewed to see where the site is in relation to the Groundwater SPZ's, and this shows that the site is not within a Groundwater SPZ.
- 2.19 The groundwater vulnerability map within Defra's Magic Map lists the underlying geology as unproductive.

¹ <https://magic.defra.gov.uk/magicmap.aspx>

Hydrology

- 2.20 There are no watercourses within or on the boundary of the site. The nearest known open water body is an ordinary watercourse that flows east-west approximately 280m north of the site. This watercourse lies at the base of a shallow valley feature on which the site sits on the southern side. This means that the site is within the catchment of this watercourse.

Existing Drainage Regime

- 2.21 The existing private sewerage was investigated during a site visit on 26th March 2024. The inspection chambers surrounding number 149 College Lane were lifted to review the below ground connections and the nature of the existing on-site drainage. The outputs of this review can be seen in [Appendix E](#).
- 2.22 The below-ground private drainage revealed that foul and surface water is combined within the property curtilages and flows to the public sewer in College Lane. With that in mind the public sewer records were obtained from Southern Water
- 2.23 Southern Water's Asset Location Plans can be seen in [Appendix F](#). These show that there is a single 300mm VC (Vitreous Clay) foul sewer running south-north along College Lane. A single drain cover of this run was lifted to review the sewer inputs and the manhole was found to be heavily silted up. Notwithstanding this, it was commuting both foul and surface water flows, as evidenced by the private on-site sewerage, which was reflected, by observation, by other properties on College Lane. Additionally, a manhole cover upstream of the site was a grated cover, designed to allow surface water to weir directly into the manhole and into the drainage system.
- 2.24 Therefore, although Southern Water's Asset Location Plans show a foul sewer in College Lane, this sewer is in fact performing as a combined sewer and its diameter of 300mm also suggests that it is designed to commute more than just foul flows.
- 2.25 Southern Water's Developer Services were contacted regarding the discrepancy between the mapped sewer type and the actual flows which are discharging to the sewer. They confirmed that although the asset plans designate a foul sewer, there may be permitted surface water connections to it which mean that it is taking combined flows. This is consistent with the situation evidenced through the site inspection. Southern Water also confirmed that whilst the asset location plans indicate a foul sewer, it does not mean that a surface water connection will not be accepted moving forward, particularly where there are existing combined flows (as is the case here) and it can be demonstrated that there are no other options available in the drainage hierarchy (Building Regs Part H).

Existing Runoff Rate

- 2.26 The site area is 6,206m² (0.621ha) and, as an undeveloped site, it is considered to be a greenfield site. The greenfield runoff rate for the whole site was calculated using HR Wallingford's UKSuDS greenfield calculator. This showed that the undeveloped site has the following runoff rates, as shown in Table 2.2, below and the full results in [Appendix G](#).

Table 2.2: Existing Greenfield Runoff Rates

Return Period	Greenfield Runoff Rate
QBAR	3.89 l/s
1 in 1	3.31 l/s
1 in 30	8.95 l/s
1 in 100	12.42 l/s

Arboriculture

- 2.27 It was discussed earlier in Section 2 that there are some mature trees on the site's eastern and south-eastern boundaries (oak, ash, fir, birch holly and willow are present). The site layout plan in [Appendix A](#) shows the canopy and RPZ extents of these trees. The RPZ constraints have been observed in the site layout and will also be avoided in the development of the drainage strategy.

3.0 Flood Risk

- 3.1 Because there is an isolated area of 'Low' surface water flood risk within the site and an area of 'High' surface water flood risk shown on College Lane, the LPA (MSDC) have requested that an FRA is carried out.
- 3.2 Flood Risks to the site are generally low, from all sources, thus this section of the drainage strategy will review flood risk from rivers, surface water, groundwater, reservoirs, canals and infrastructure.

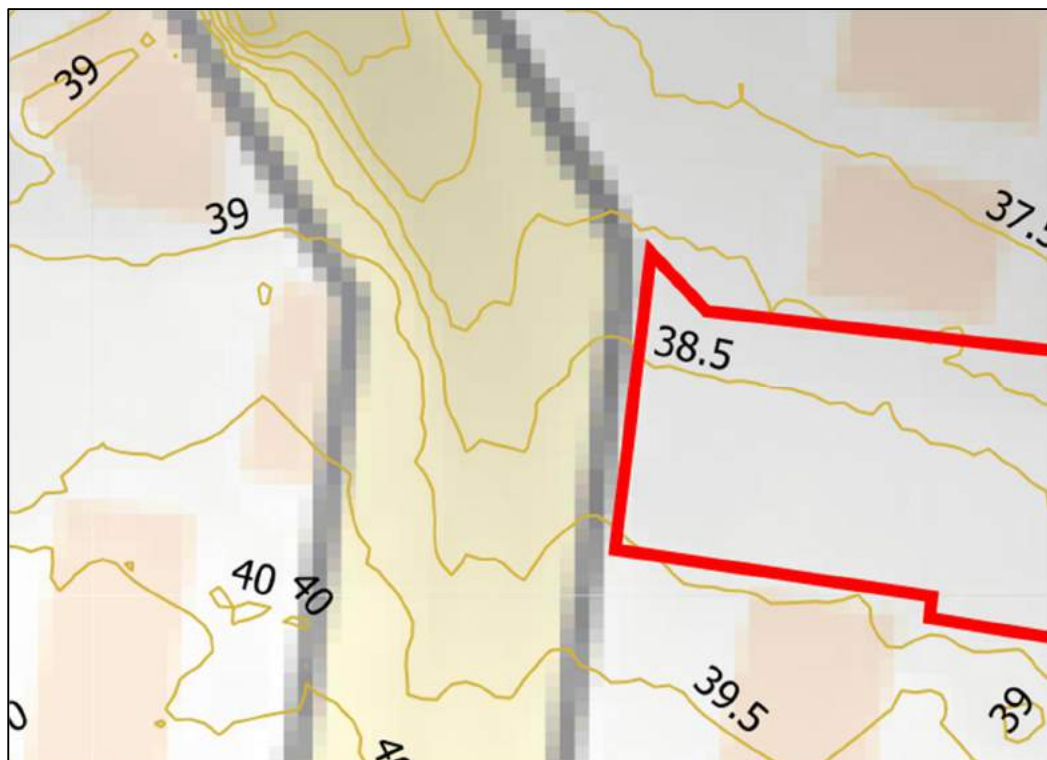
Fluvial Flood Risk

- 3.3 The Environment Agency's Flood Map for Planning ([Appendix H](#)) reveals that the site is within Flood Zone 1. Therefore, the site is at very low risk of flooding from rivers and sea.

Surface Water Flooding

- 3.4 The EA's RoFSW map ([Appendix I](#)) shows the site to largely be within an area of very low surface water flood risk. However, there is an isolated strip of 'Low' surface water flood risk (between 1 in 100-year and 1 in 1,000-year chance) within the site.
- 3.5 Because this isolated area of 'Low' surface water flood risk originates and terminates within the site, it means that it can be dealt with by the drainage strategy, which is discussed in the latter part of this report. Any surface water generated by the development will be conveyed and attenuated within surface water features on the site, which will remove this 'unattenuated' surface water flood risk.
- 3.6 The RoFSW map shows an area of 'High' surface water flood risk outside of the site boundary in College Lane. The LiDAR topographical information in [Appendix C](#) and excerpted, below, in Figure 2.1 show that the camber of the road is domed, and road levels fall from the centre of the carriageway to form a shallow drainage channel at the edges. The edge of the road on the eastern side are lower than the site entrance. This means that surface water in the roadway of College Lane would not be able to ingress to the site.

Figure 2.1: Topography showing the channels on the edge of College Lane



- 3.7 Consequently, although there is an area of 'High' surface water flood risk in College Lane, this is of no consequence to the site as this area of flood risk would not be able to ingress into the development.
- 3.8 The way the RoFSW mapping has been developed is worthy of discussion in the context of the 'High' surface water flood risk shown in the carriageway of College Lane and, indeed, many roads and highways within the mapped area.
- 3.9 The RoFSW maps are a national scale product and, consequently, use broad-scale modelling assumptions that effect the accuracy and reliability of the data at a local scale. The following discussion regarding the accuracy and reliability of the RoFSW mapping has been taken from the Environment Agency's document 'What is the Risk of Flooding from Surface Water map?' (Report version 2.0, April 2019).
- „ The RoFSW maps have digital terrain mapping (DTM) information at 2m resolution or finer for most urban areas in England and Wales. Rural areas (such as Hurstpierpoint) are often mapped with coarser resolution data (5m), which means that the RoFSW maps will be less accurate and still cannot represent every detail of the landscape and the local mechanisms of flooding.
 - „ The RoFSW also makes assumptions about the drainage rate in highways and how it 'removes' rainfall. This uses a nationalised average of 12mm/hour, and other average hydrological losses are applied in rural areas. Small changes to the drainage rate can have a big effect on the RoFSW and local drainage features and capacities are generally not taken into account.
 - „ During the production of the RoFSW mapping, road surfaces were lowered by 0.125m (the height of a British Standard HB2 kerb) to better delineate these potential flood pathways in the hydraulic model. The EA has stated that this approach overestimates the routing effect of roads in rural areas where there aren't raised kerbs. This is particularly relevant in College Lane, Hurstpierpoint, which does not have HB2 kerbs in the vicinity of the site.
- 3.10 This means that the surface water flood risk in College Lane is likely to be overestimated, but notwithstanding this, the development needs to ensure that it does not contribute to these flows and exacerbate local flood risk. This will be done in the drainage strategy.

Groundwater Flooding

- 3.11 Weald Clay geology exists to depth below site. As described in Defra's Magic Map application, the site is in an area where the geology is listed as unproductive. This is due to the impermeability of the clay soils.
- 3.12 With this in mind, the site cannot be considered at risk of groundwater flooding as there are no aquiferous geologies below the site and those that are present are hydraulically unproductive.

Reservoir Flood Risk

- 3.13 The Environment Agency (EA) provides a map showing the maximum potential flood extent should all reservoirs with a capacity of greater than 25,000 cubic metres fail and release the water they hold.
- 3.14 The map shows that College Lane would not experience flooding in this scenario as it is not within proximity of any reservoirs or watercourses that would commute reservoir flood flows.

Canals

- 3.15 There are no canals in the local area to create flood risk.

Flooding from Infrastructure Failure

- 3.16 Sewer flooding can occur when the capacity of the infrastructure is exceeded by excessive flows, or because of a reduction in capacity due to collapse, siltation, blockage, or if the downstream system becomes surcharged. This can lead to the sewers flooding onto the surrounding ground via manholes and gullies, which can generate overland flows.

- 3.17 Typically, sewer systems are constructed to accommodate rainstorms with a 30-year return period or less, depending on their age. Consequently, rainstorm events greater than 1 in 30-years would be expected to result in surcharging of some parts of the sewer system. In fact, due to most gullies being poorly maintained and often partially blocked with silt, leaves and other debris, their capacity is often estimated to be closer to the 1 in 10-year storm.
- 3.18 If the drainage in College Lane were to block it is expected that the development would not be at risk of flooding because of the shallow channels at the edges that are lower than the site entrance.
- 3.19 With regards to the proposed development's drainage system and risk of failure, the onsite drainage will be designed to capture and attenuate surface water from the 1 in 100-year + 45% rainfall event. A drainage management and maintenance plan will also be provided, which will prescribe how the onsite drainage infrastructure should be looked after so that it works at optimum capacity. This will ensure that residual flood risks to the site from its internal drainage systems will be minimised.

Summary

- 3.20 Overall, the flood risks to the site are very low and there is no flood risk that could preclude the development. There is a small, isolated area of surface water flood risk within the site boundary. Because this originates and terminates entirely within the site, it means it will be captured and attenuated by the development's drainage strategy.
- 3.21 The 'High' surface water flood risk in College Lane is predominantly as a result of how the RoFSW mapping is produced and the high-level assumptions that are made in the flood model's construction. Notwithstanding this, the levels and cambers on the carriageway of College Lane determine that any surface water flood risk within the road would not be able to ingress into the development site.
- 3.22 There are no other forms of flood risk that need consideration in this FRA.

4.0 Surface Water Drainage Strategy

Sustainable Drainage Overview

- 4.1 Current planning policy requires developments to employ SuDS (Sustainable Drainage Systems) techniques wherever feasible. Careful design of SuDS features can ensure that a development's surface water drainage closely reflects the natural hydrology of the pre-developed site.
- 4.2 SuDS will attenuate and treat surface water run-off quantities at the source (source control) in line with current guidance and best practice.
- 4.3 Source control systems treat surface water close to the point of origin, in features such as soakaways, permeable paving and swales, to name a few.
- 4.4 The key benefits of SuDS are as follows:
- „ Improving water quality over a conventional piped system by removing pollutants from diffuse pollutant sources (e.g., Lanes);
 - „ Improving amenity through the provision of open green space;
 - „ Improving biodiversity through increased areas for wildlife habitat; and
 - „ Enabling a natural drainage regime that recharges groundwater (where possible).
- 4.5 SuDS provide a flexible approach to drainage, with a wide range of components from soakaways to large-scale basins or ponds. The individual techniques should be used where possible in a management train that mimics the natural pre-developed pattern of drainage.

Site Areas

- 4.6 The site areas to undergo development are as follows in Table 3.1.

Table 3.1 – Site Areas

Breakdown of site areas	Proposed (ha)
Total developable area	0.621
Total impermeable areas in the development	0.211
Total permeable areas in the development	0.410

Greenfield Runoff Rate

- 4.7 The greenfield runoff rates for the developed area have been calculated using HR Wallingford's UKSuDS online greenfield calculator and are presented in Table 3.2, below. The greenfield runoff rates have been calculated from the proposed impermeable areas (0.211 ha) in accordance with the requirements of Paragraph 12 of The Environment Agency's Rainfall Runoff for Developments and ADC's SuDS guidance, which states that *"the calculations must be based on the positively drained area, rather than the entire greenfield site area, as these are the parts of the site that will positively drain into the on-site drainage infrastructure"*. For the purposes of the drainage design and hydraulic modelling, it has been assumed that the undeveloped parts of the site (gardens, verges, beds, etc.) will drain naturally by infiltration or that rainwater will be intercepted by vegetation.
- 4.8 The greenfield calculator has been run for the impermeable areas of 0.211 ha and the results are as follows in Table 3.2 and can be seen in full in [Appendix J](#).

Table 3.2 – Greenfield Runoff Rate/Volume – Total Impermeable Areas

Return Period	1 in 1	1 in 30	1 in 100	QBAR
Discharge Rate (l/s)	1.12	3.04	4.22	1.32

- 4.9 The calculated QBAR greenfield runoff rate of 1.32 l/s provides a runoff rate equal to 6.27 l/s/ha.
- 4.10 The above greenfield runoff rates will be used to guide the appropriate surface water discharge rates and volumes from the development.

The Drainage Hierarchy

- 4.11 The NPPF states that opportunities to reduce overall flood risk should be sought and achieved through sustainable development and careful drainage design. This can be achieved through the layout and form of development, including green infrastructure and the appropriate application of sustainable drainage systems (SuDS). SuDS are designed to control surface water runoff close to where it falls and mimic natural drainage as closely as possible. They provide opportunities to:
- „ Reduce the causes and impacts of flooding;
 - „ Remove pollutants from urban run-off at source;
 - „ Combine water management with green space with benefits for amenity, recreation and biodiversity.
- 4.12 To deliver SuDS benefits and ensure that a development reduces overall flood risk, there is an established hierarchy of surface water drainage methods that should be considered. The most preferable and sustainable are at the top and the least preferable and least sustainable at the bottom.
- 4.13 The drainage hierarchy is a sequential check that intends to ensure that all practical and reasonable measures are taken to manage surface water as high up the hierarchy (with '1' being the highest) as possible, and that the amount of surface water managed at the bottom of the hierarchy is minimised. The Planning Practice Guidance to the National Planning Policy Framework (NPPF) states that “*Generally, the aim should be to discharge surface run off as high up the following hierarchy of drainage options as reasonably practicable*”.
- 4.14 The principle of the drainage hierarchy comes from Building Regulations Part H, but has been adopted and expanded by others and now can be viewed as the following:
1. Store rainwater for later use
 2. Use infiltration techniques, such as porous surfaces in non-clay areas
 3. Attenuate rainwater in ponds or open water features for gradual release
 4. Attenuate rainwater by storing in tanks or sealed water features for gradual release
 5. Discharge rainwater direct to a watercourse
 6. Discharge rainwater to a surface water sewer/drain
 7. Discharge rainwater to the combined sewer
 8. Discharge rainwater to the foul sewer

- 4.15 Developers should not choose the method that is the most convenient or represents the lowest cost. LPA's, LLFA's and Water Authorities may enforce the surface water drainage hierarchy and demand that the highest practicable tier of the hierarchy is used.
- 4.16 The first two tiers of the drainage hierarchy ensure that surface water is retained within the site boundary and does not increase flood risk to others. This is always the most preferable method of surface water management.
- 4.17 The next six tiers of the hierarchy provide regional control, but with decreasing levels of pollution removal and reduced potential for amenity and habitat creation.
- 4.18 Within the lower six tiers of the drainage hierarchy, there must be some form of flow restriction, so that off-site surface water discharge is reduced, as much as is reasonably practicable. This requires on-site storage facilities, which may include ponds, swales, subsurface storage tanks and System C (non-infiltration) permeable pavements with flow control devices. Again, methods that provide the most potential for amenity and pollution removal should be favoured.

Drainage Strategy – Compliance with the Drainage Hierarchy

- 4.19 The drainage strategy for the development has been progressed in acknowledgement of the drainage hierarchy and the highest available tiers. In summary, the availability of the drainage hierarchy and the geo-environmental constraints that have informed the drainage strategy are as follows:

1. Store rainwater for later use

- 4.20 Water re-use systems can rarely manage 100% of the surface water discharged from a development. This requires the surface water yield from the building and hardstanding areas to balance perfectly with the demand from the proposed development; too much demand will result in lack of water supply; too little demand will cause the storage systems to become overwhelmed and could result in flooding when the next rainfall event happens. Consequently, even if there are opportunities and a need for rainwater recycling systems, further solutions for attenuating and discharging surface water will almost always be required.
- 4.21 There is likely to be a moderate rainwater yield from the roof areas of the development that could be used for domestic non-potable water uses. The proposed development includes gardens and landscaping that may benefit from having a supply of recycled rainwater for watering.
- 4.22 The opportunity for water re-use and recycling on site has been explored and the development will be specifying water butts for each property. The water butts will reduce the reliance on potable water supplies during activities such as gardening and car washing.
- 4.23 Water butts do also provide small amounts of surface water storage and can often assist in achieving zero discharge for rainfall depths up to 5mm, which covers 50% of annual rainfall events (according to the EA's Rainfall Runoff Management for Developments report – SC030219).
- 4.24 The drainage strategy will include at least 1no. 'Type 2' standard water butt on each property, which have a capacity of 210 litres.
- 4.25 The surface water storage available in the water butts has not been included in any hydraulic calculations as it can't be guaranteed that they will be empty at the start of a rainfall event.

2. Use infiltration techniques, such as porous surfaces in non-clay areas

- 4.26 Because the site is in an area of Weald Clay with hydraulically unproductive geology, infiltration as a form of surface water discharge is deemed unviable. As such, this tier of the drainage hierarchy is not available.

3. Attenuate rainwater in ponds or open water features for gradual release

- 4.27 Because of gradients across the site (approximately 1 in 20) and the narrow form of the site, there is limited space for open water features that can be used for surface water attenuation. As such, this tier of the drainage hierarchy is not available for the drainage strategy.

4. Attenuate rainwater by storing in tanks or sealed water features for gradual release

- 4.28 Attenuation of surface water using tanks and the use of permeable pavements is the only available tier of the drainage hierarchy for the storage of surface water on site. Thus, this tier of the drainage hierarchy has been used for the drainage strategy.

5. Discharge rainwater direct to a watercourse

- 4.29 There are no ponds, watercourses or ditches within the site or on the site boundaries (or, indeed, just beyond it) that can be used for the discharge of surface water. Consequently, this tier of the drainage hierarchy is not available.

6. Discharge rainwater to a surface water sewer/drain

- 4.30 As shown in the Southern Water asset location plans in [Appendix F](#), there are no public surface water sewers available in College Lane or in proximity to the site.

7. Discharge rainwater to the combined sewer

- 4.31 As discussed in Section 2, the public sewer in College Lane is currently performing as a combined sewer due to it commuting foul and surface water flows from the surrounding properties. The grated manhole cover upstream of the site would also allow surface water to weir directly into the manhole and into the drainage system, so it is likely that surface water from the highway is also making its way into the public sewer.

- 4.32 Although the public sewer is listed as a foul sewer, it is performing as a combined sewer and it is proposed to connect the development's surface water to this combined sewer.

8. Discharge rainwater to the foul sewer

- 4.33 Following the consideration and unavailability of the higher tiers of the drainage hierarchy, the only available outfall for surface water is to discharge to the foul sewer in College Lane, which is performing as a combined sewer.

- 4.34 As stated by Southern Water, connections to the foul sewer are permissible if no other options are available, which is the case for the proposed development, as has been shown through the drainage hierarchy. Permission will be sought to discharge the site's surface water to the public foul sewer in College Lane, which already commutes surface water from surrounding development on College Lane.

- 4.35 It has been demonstrated that the sequential approach to the drainage hierarchy has been fully tested and complied with. This has determined the available methods for attenuation and surface water discharge. How the available tiers of the drainage hierarchy have been used and the design details of the drainage strategy are discussed, below.

Drainage Strategy – Overview and Design Details

- 4.36 This overview of the drainage strategy should be reviewed in conjunction with the drainage strategy plan, which can be found in [Appendix K](#) of this report.

- 4.37 It is proposed to construct part of the access road from System C (tanked) permeable pavements with a 450mm-deep open-graded subbase. The permeable pavements will be constructed with baffles to ensure maximum attenuation is achieved on the sloping topography of the site.

- 4.38 The first part of the access road will drain into these permeable pavements. Rainfall falling directly onto the permeable pavements will drain to the subbase via the open joints in the pavements.
- 4.39 Rainfall falling on the roofs and driveways of the surrounding properties will also drain directly into the subbase of the permeable pavements using rainwater diffuser units.
- 4.40 To provide additional attenuation in the drainage system, a geocellular tank will be positioned below the turning head in the northeast of the site.
- 4.41 Because the topographically lowest point of the site is the furthest from the access, it is necessary to pump surface water from this part of the site back to the access. With this in mind, a packaged pump station will be used, which will pump surface water from the lowest part of the site to another attenuation tank near the access to College Lane.
- 4.42 The second attenuation tank allows a final gravity connection to the foul sewer in College Lane at a restricted rate. It is proposed that the final outfall for surface water will discharge at a maximum of 2 l/s, in all storms, which is the practicable minimum for an open-flowing system where other drainage elements and the open downstream connection can introduce debris either through flow inputs, or be introduced from the public sewer.
- 4.43 The attenuation volume provided by the 30% porosity subbase of the permeable pavements is 19.17m³. Because MicroDrainage cannot adequately model baffled pavement systems, the gradient of the permeable pavements has been set at 1 in 40 in the hydraulic model to represent the additional attenuation that would be offered by a 1 in 20-gradient permeable pavement with baffles.
- 4.44 The attenuation in the lower part of the site is increased by the geocellular tank that is positioned under the permeable pavements in the turning head. This geocellular tank has dimensions of 7m x 8m x 2m (L x W x D), which yields a volume of 106.4m³ at 95% porosity.
- 4.45 The pump that lifts surface water from the attenuation in the lower part of the site will do so at a rate of 5.0 l/s.
- 4.46 The geocellular attenuation tank at the top of the site, near the site entrance, will be 9m x 4m x 0.8m (L x W x D) and will have a volume of 27.36m³. This will receive surface water from the pumped rising main. Surface water will discharge from this attenuation tank by gravity via a hydrobrake set at 2 l/s, which will be the maximum surface water discharge in all storms.
- 4.47 Overall onsite attenuation totals 152.93m³.
- 4.48 The maximum surface water discharge rate of 2 l/s from the development's impermeable areas compares favourably to the developable area QBAR greenfield runoff rate of 1.32 l/s. It is also less than the 1 in 30-year and 1 in 100-year greenfield runoff rates of 3.04 l/s and 4.22l/s, respectively, which means that in the higher order rainfall events that the proposed drainage system will reduce overall surface water runoff and flood risk in the area.

Urban Creep

- 4.49 In accordance with West Sussex County Council's Policy for the Management of Surface Water, urban creep is recognised as playing a key role in future flood risk.
- 4.50 WSCC have produced their own guidance on the percentage of urban creep that should be applied. They state that the consideration of urban creep should be assessed on a site-by-site basis but is limited to residential development only. The allowances set out in Table 5.2 of WSCC LLFA Policy for the Management of Surface Water must be applied to the impermeable area within the property curtilage according to the proposed development density. Table 5.2 is shown below.

Table 5.2 of WSCC LLFA Policy for the Management of Surface Water

Residential development density (Dwellings per hectare)	Change allowance (% of impermeable area)
≤25	10
30	8
35	6
45	4
≥50	2
Flats & Apartments	0

- 4.51 A precautionary approach has been taken, which means that an urban creep increase of 10% has been applied to the impermeable areas within the residential curtilages of the proposed development (this does not include the roads or pathways, because these are not within the residential curtilages and are not subject to the process of urban creep).
- 4.52 Urban creep has been applied to the development's impermeable areas in accordance with the following parameters:
- „ The total developable area is 0.621 ha.
 - „ The total impermeable area is 0.211 ha.
 - „ The total privately-owned impermeable areas (dwelling roofs and driveways) are 0.093 ha.
 - „ 10% of the privately owned impermeable areas of 0.093 is 0.010 ha.
 - „ Therefore, the total impermeable areas with a 10% uplift due to urban creep is 0.221 ha (0.211 ha plus 0.010 ha).
- 4.53 The 10% increase in impermeable areas has been input to the hydraulic design of the drainage strategy and applied at the appropriate points in the drainage system.
- 4.54 The approach to the layout and design of the surface water drainage strategy for the development has been outlined and the drainage strategy layout has been presented in [Appendix K](#) of this report.

Climate Change Parameters

- 4.55 College Lane, Hurstpierpoint, is located within the Adur and Ouse Management Catchment. The 2070's epoch climate change increase parameters are 40% for the 1 in 30-year rainfall event and 45% for the 1 in 100-year rainfall event.
- 4.56 These climate change parameters have been applied to the MicroDrainage hydraulic model of the drainage strategy to make sure that future rainfall volumes and intensities have been incorporated into the conveyance and attenuation features of the drainage system.

MicroDrainage Hydraulic Modelling

- 4.57 The drainage strategy outlined above has been designed in MicroDrainage's Network hydraulic modelling module.
- 4.58 The results of the MicroDrainage hydraulic modelling for the proposed development can be seen in [Appendix L](#).
- 4.59 The results of the hydraulic modelling show that the drainage strategy discussed above can attenuate and discharge all surface water generated from the development's impermeable areas in the 1 in 100-year + 45% rainfall event without flooding.

-
- 4.60 The MicroDrainage Network model has a maximum half drain time of 1,120-minutes, which is less than the 1,440-minute requirement for this metric.
- 4.61 The attenuation structures maintain a minimum of 0.424m freeboard in the 1 in 100-year + 45% rainfall event. This means that the drainage system has attenuation available for exceedance events.
- 4.62 It is noted that WSCC as the LLFA require that a Cv value of '1' is used for hydraulic calculations. Clarity should be provided on how the Cv value is calculated to show that a value of 1 is inappropriate.
- 4.63 The value of C (the runoff coefficient) is derived in the Modified Rational Method (MRM) from a combination of two coefficients (Cv and Cr), which are multiplied together determine the appropriate runoff coefficient value (C).
- 4.64 Cr is a constant of 1.3, thus if a Cv value of 0.75 is specified, then this gives a resultant runoff coefficient (C) of 1.0, meaning 100% of rainfall inputs is included in the drainage calculations. This is the correct approach.
- 4.65 If Cv value of 1.0 is used, then when it is multiplied by the Cr of 1.3, it gives a C value that calculates that 130% of rainfall is included in the drainage calculations. This, therefore, results in an overestimation of 30% and is incorrect. However, the above is not often understood by risk management authorities and it is presumed that by specifying a Cv value 1.0 that it will capture 100% of rainfall inputs, when, in fact, a Cv value of 0.75 already achieves that.
- 4.66 The above information is evidenced by the information in [Appendix M](#), which is Volume 4 of The Wallingford Procedure, which discusses the Modified Rational Method. Section 4 (Page 5) clearly shows how the runoff coefficient of C should be calculated and that Cr is valued at 1.3.
- 4.67 Regarding the statement that the superseded SFA7 requires a Cv value of 1.0 is also not correct. The relevant sentence from SFA7 has been carried over to the extant DCG and it states that:
- "Surface water sewer systems should be designed to take the runoff from roofs, yards belonging to those buildings and, subject to the agreement of the sewerage company, the runoff from streets (see A8) including any flows through pervious pavements where they discharge into the system. For these areas, an impermeability of 100% should be assumed unless it can be demonstrated that the proposed management arrangements will limit the rate of runoff to a lower level."*
- 4.68 Therefore, SFA7 and the DCG doesn't say that a Cv of 1 should be used. What it does say is that areas that discharge to the drainage system should have their percentage impermeability (PIMP value) set to 100%.
- 4.69 In accordance with this, all impermeable areas in the MicroDrainage model have their PIMP value (percentage impermeable) set to 100%, so that all areas positively producing surface water are shown to have 100% of their surface water contributing to the drainage system.
- 4.70 And, as shown above, the correct calculation of Cv x Cr produces a 'total' runoff of 100% (and the winter coefficient of 0.84 actually overestimates runoff).
- 4.71 Consequently, the Cv value has been properly and appropriately determined. To use a Cv value of 1 would result in an additional 30% runoff, which is an unnecessary factor of safety when combined with that already used in the calculations (a FoS of 2 has been used, which is appropriate in this rural location) alongside the 45% increase for climate change.

5.0 Foul Water Drainage

- 5.1 Southern Water's Asset Location Plan in [Appendix F](#) show that there is a 300mm public foul sewer in College Lane. This represents a viable connection point for the development's foul waste.
- 5.2 The peak foul flow rate from the proposed development has been calculated based on Southern Water's foul sewerage modelling criteria, the details of which can be found in [Appendix N](#). In summary, the calculation is based on the foul flow element, plus an allowance for misconnected surface water.
- 5.3 Based on Southern Water's foul sewerage modelling criteria, the calculated design foul flow from the proposed residential development is 0.05 l/s. However, like the surface water system, the foul flows will need to be pumped from the lowest parts of the site up to the site entrance and a connection to the foul sewer in College Lane. Foul packaged pumping stations have a minimum pump rate of approximately 2.5 l/s, thus this will be the approximate flow rate from the foul system on site, although this flow rate will be intermittent (while the pump is active). Noting that the foul inputs from the development will be 0.05 l/s, this is 1/50th of the approximate minimum pump rate of 2.5 l/s, thus is indicative that the foul pump will only be 'on duty' 1/50th of the time.
- 5.4 It is anticipated that the sewer in College Lane, which has a diameter of 300mm, would have capacity for the minimal additional average foul sewage flows of 0.05 l/s from the development.
- 5.5 With regards to the foul discharge from the development, all Water and Sewerage Companies (WaSC's) have a legal obligation under Section 94 of the Water Industry Act 1991 (the Act) to provide developers with the right to connect to a public sewer regardless of capacity issues. This, in conjunction with Section 91(1) of the Act effectively means that Southern Water cannot object or for the LPA to refuse to grant planning permission on the grounds of insufficient capacity or that no improvement works are planned for an area. The case precedent for this is a Supreme Court decision in Barratt Homes vs Welsh Water, in which the court held that the developer has an absolute right to connect to the existing sewer, whether or not it overloads the system. It ruled that the specific wording of the legislation allows for this right to be exercised, at no cost to the developer, apart from the normal connection charges.
- 5.6 Where local sewerage infrastructure constraints are identified, network reinforcements are delivered by the WaSC through New Infrastructure Charges on developers. For non-strategic sites, the WaSC company have a maximum of 24 months to deliver sewerage improvements from the date of the outline or full planning consent.

6.0 Surface Water Runoff Quality

- 6.1 The NPPF states that development should not have a detrimental impact on the environment, including the water environment. The technical guidance to the NPPF provides further advice on the benefits of ensuring runoff quality is to an appropriate standard.
- 6.2 The CIRIA SuDS Manual provides guidance on the treatment of surface water runoff. With regards to the proposed development, Table 4.3 of the CIRIA SuDS Manual rates the pollution hazard from roof water runoff as 'very low'. The only requirement for roof water runoff is the removal of gross solids and sediments, which would be achieved using catchpits and silt traps upstream of the permeable paving and throughout the drainage network.
- 6.3 With regards to the access and driveways, Table 4.3 of the CIRIA SuDS Manual rates the pollution hazard from residential car parking and low traffic lanes as 'low'. To mitigate a 'low' pollution hazard, the CIRIA SuDS Manual recommends using a simple index approach in line with Section 26.7.1. This is discussed, below.
- 6.4 Table 26.2 of the CIRIA SuDS Manual provides pollution hazard indices for different land use classifications. The land use classification that requires consideration for low traffic lanes and parking areas is in Table 6.1 below.

Table 6.1: Excerpt from Table 26.2 of CIRIA SuDS Manual

Land Use	Pollution Hazard Level	Total Suspended Solids (TSS)	Metals	Hydro-Carbons
Individual property driveways, residential car parks, low traffic Lanes (e.g. cul-de-sacs, homezones and general access Lanes) with less than 300 traffic movements per day.	Low	0.5	0.4	0.4

- 6.5 To deliver adequate pollution treatment and mitigation, the CIRIA SuDS Manual recommends using a SuDS component that has a total pollution mitigation index (for each contaminant type) that equals or exceeds the pollution hazard index (for each contaminant type).
- 6.6 Table 26.3 of the CIRIA SuDS Manual provides indicative SuDS mitigation indices for each SuDS type when discharging to surface waters. Table 6.2, below, which is an excerpt from Table 26.3, shows the mitigation index for permeable pavements, which surface water from the whole site will pass through prior to discharging to the public sewer.

Table 6.2: Pollution Mitigation Indices for Permeable Pavements

Type of pollution removal component	Total Suspended Solids (TSS)	Metals	Hydro-Carbons
Permeable Pavements	0.7	0.6	0.7

- 6.7 The mitigation indices for permeable pavements exceed those of the highest pollution hazard index figures from Table 6.1. Therefore, the pollution hazards from the site will be fully mitigated and no other pollution prevention measures are required for the development.

7.0 Residual Risk

- 7.1 Whilst the drainage strategy for the development has been designed to attenuate surface water from the 1 in 100-year plus 45% rainfall event, there could be a small residual risk of flooding due to blockage or failure or poor performance of on-site infrastructure. Therefore, appropriate and regular maintenance of the drainage infrastructure should be undertaken by the site management company or their agents.
- 7.2 To assist with this process, a Drainage Management and Maintenance Plan has been prepared, which sets out the principles for the long-term management and maintenance of the proposed surface water drainage system on the development. The Drainage Management and Maintenance Plan can be seen in [Appendix O](#).
- 7.3 The purpose of this document is to ensure that those responsible for site maintenance have a robust inspection and maintenance plan going forwards. This will help ensure the optimum operation of the surface water drainage system and that it will be regularly maintained for the lifetime of the development. This will contribute to reducing the risk of surface water flooding both on- and off-site.

8.0 Exceedance Events

- 8.1 Exceedance events are those greater than the design rainfall event (i.e., greater than the 1 in 100-year rainfall event plus 45% for climate change).
- 8.2 Any rainfall events greater than the design rainfall event may cause flooding due to them 'exceeding' the capacity of the drainage system. In this situation it is imperative to check whether flooding would occur and, if so, whether it needs to be contained on site. Exceedance flows should not ingress into any properties on site and should not cause nuisance to any neighbouring sites or buildings.
- 8.3 As mentioned earlier in this report, freeboard exists within the on-site attenuation structures for exceedance events. This allows for the attenuation of surface water from storms beyond the 1 in 100-year + 45% event.
- 8.4 The site slopes from south to north and if exceedance flows were to occur, there would be some ponding of surface water against the HB2 (125mm face) kerbs that will line the northern side of the turning head. If surface water from exceedance flows is able to penetrate past these HB2 kerbs, down the northernmost property driveways, for example, then surface water would flow northwards to the back gardens and into the open fields beyond. This means that there is little to no impact of exceedance flows on- or off-site.
- 8.5 A high-level plan of exceedance flows has been produced to show the suspected direction that exceedance flows would take across the site. This can be seen in [Appendix P](#).

9.0 Summary and Conclusion

- 9.1 This FRA and drainage strategy has been produced by Motion on behalf of their client, Earlswood Homes. It supports the proposed development of 7no. residential dwellings plus parking, access and landscaping on the land behind 147 – 149 College Lane, Hurstpierpoint.
- 9.2 The EA's Flood Map for Planning shows that the site is within Flood Zone 1 – very low risk of flooding from rivers or the sea.
- 9.3 There is an isolated strip of 'Low' surface water flood risk within the site. Because this area of 'Low' surface water flood risk originates and terminates within the site, it will be dealt with by the drainage strategy for the site. Any surface water generated by the development will be conveyed and attenuated within surface water features on the site, which will remove this 'unattenuated' area of surface water flood risk.
- 9.4 The RoFSW map shows an area of 'High' surface water flood risk outside of the site boundary in College Lane. The road levels in College Lane fall from the centre of the carriageway and form a shallow drainage channel at the edges of the road that are lower than the site entrance. This means that surface water in College Lane, the risk of which is exaggerated due to the way the RoFSW mapping is modelled, would not ingress to the site. Therefore, the proposed development is at very low risk of surface water flooding.
- 9.5 The risk of flooding from groundwater, canals, reservoirs and infrastructure failure are also deemed to be low and climate change-related increases in flood risk are not considered to be an issue for the proposed development site.
- 9.6 The drainage strategy for the proposed development has been produced in line with the drainage hierarchy and Mid-Sussex District Council's Flood Risk and Drainage Information Check List and Policy DP41 of Mid-Sussex District Council's Local Plan.
- 9.7 The site's surface water system also uses geocellular attenuation tanks for additional attenuation and requires the use of a pump to lift surface water from the lowest parts of the site to the site entrance and the 300mm sewer in College Lane which is currently performing as a combined sewer and is evidenced as taking surface water from the surrounding properties and the highway. Therefore, while it is labelled as a foul sewer on asset plans, it is in fact a combined sewer and represents a viable connection point for surface water, which is the highest available tier of the drainage hierarchy.
- 9.8 Flow will be restricted to a maximum flow rate of 2 l/s using a hydrobrake. The discharge rate of 2 l/s compares favourably to the QBAR greenfield runoff rate of 1.32 l/s and is less than the 1 in 30 and 1 in 100-year rainfall runoff rates.
- 9.9 This drainage strategy has been hydraulically modelled in MicroDrainage's Network module, which has shown that the proposed drainage strategy provides the requisite attenuation for surface water generated by the impermeable areas of the development up to and including the 1 in 100-year + 45% rainfall event, without flooding.
- 9.10 The proposed drainage strategy can successfully mitigate the expected pollution hazards that will be generated on site. Pollution mitigation will be provided by the permeable pavements.
- 9.11 A drainage management and maintenance plan has been produced that shows how the proposed drainage system will be maintained in perpetuity.
- 9.12 Exceedance flows have been considered and an exceedance plan produced.
- 9.13 Foul waste from the development will also connect to the 300mm foul sewer in College Lane.
- 9.14 In conclusion, this drainage strategy has shown that the proposed development is at a very low residual risk of flooding from all sources. This means that the proposed seven dwelling 'non-major' development is appropriate in this location. Similarly, the drainage strategy has shown that the development can

manage its foul and surface water and is using the highest available and practicable tiers of the drainage hierarchy.

- 9.15 Mid-Sussex District Council's Flood Risk and Drainage Information Check List has been completed and is in [Appendix Q](#).

Appendix A

Proposed Development Layout

Do not scale from this drawing, except for planning purposes.
Any discrepancies are to be reported to DMS Architects.
Refer to Structural Engineers details for structural design
criteria.
This drawing remains the copyright of DMS Architects.



Rev	Date	Description



www.dmsarchitects.co.uk
Old Chambers 93-94 West Street,
Farnham, Surrey, GU9 7EB
Tel. 01252 246616 / 07812 504922

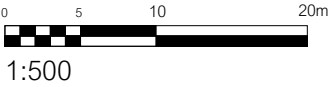
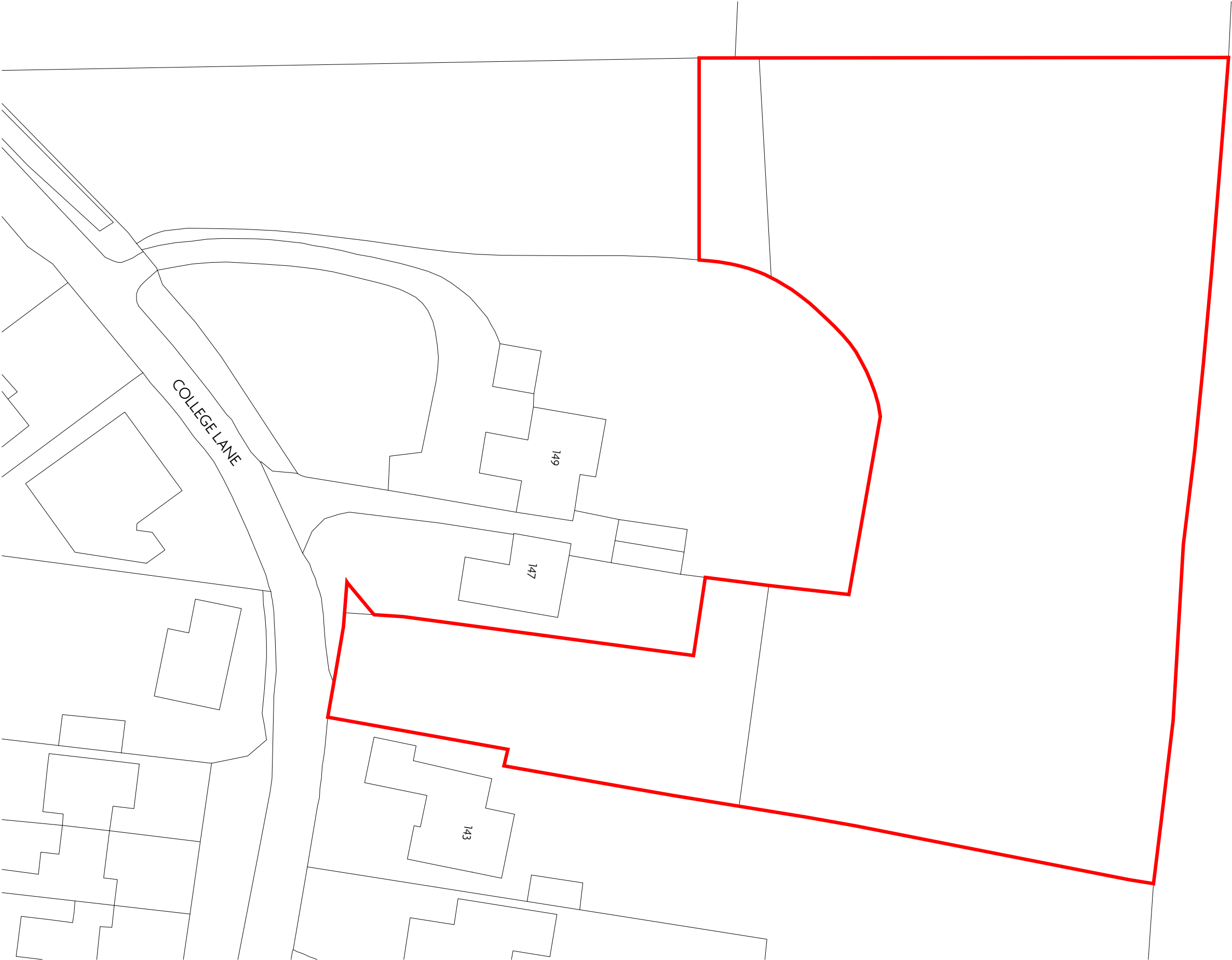
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Project: Land off College Land
Hurstpierpoint

Drawn by: DMS
Scale: 1:200 @ A1
Date: Mar 2024
Checked by:

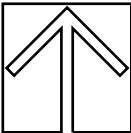
Drawing Number: 01497/PL01
Revision:

Appendix B

Site Location Plan



Any discrepancies to be reported to Earlswood Homes.



Rev	Date	Description



<u>Date:</u> Jan 2024	<u>Drawn by:</u> BCL
<u>Scale:</u> 1:500 @ A3	<u>Checked:</u> *

Client:
Earlswood Homes

Project:
Land at 147 to 149
College Lane
Hurstpierpoint

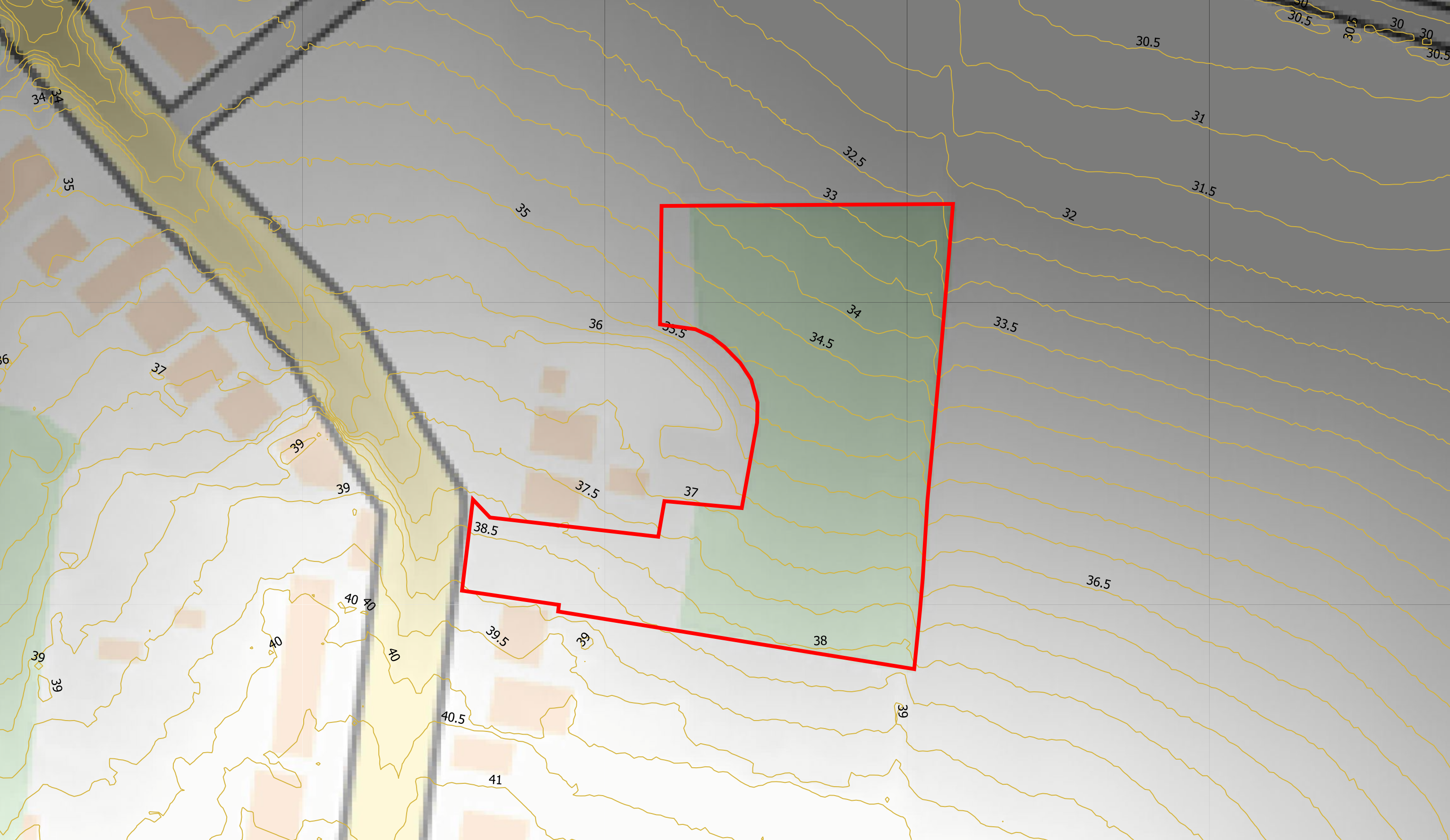
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Location

Drawing:

LOCATION PLAN (1:500)

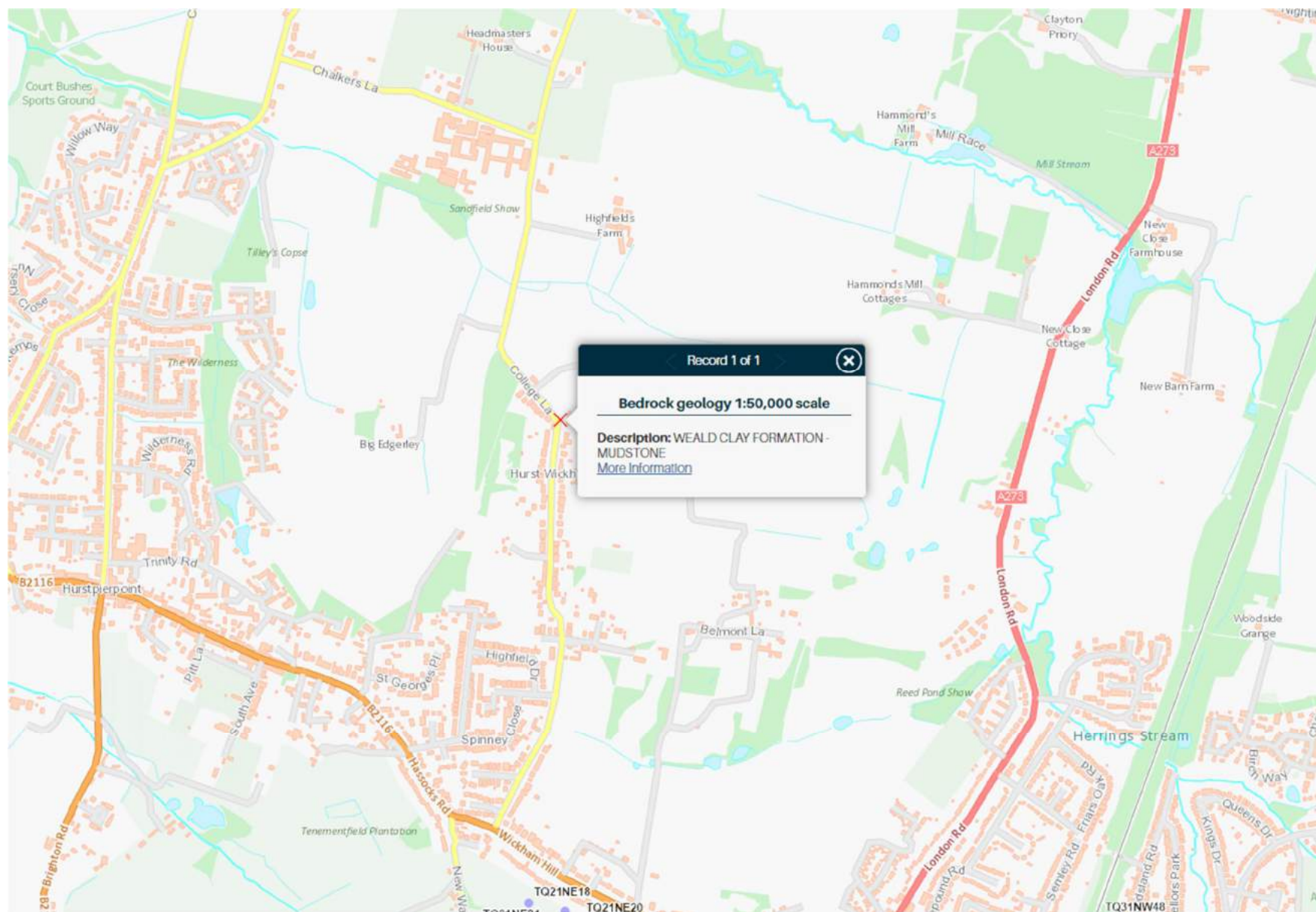
Appendix C

LiDAR Topographic Levels



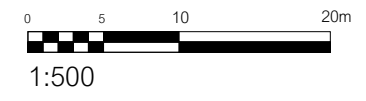
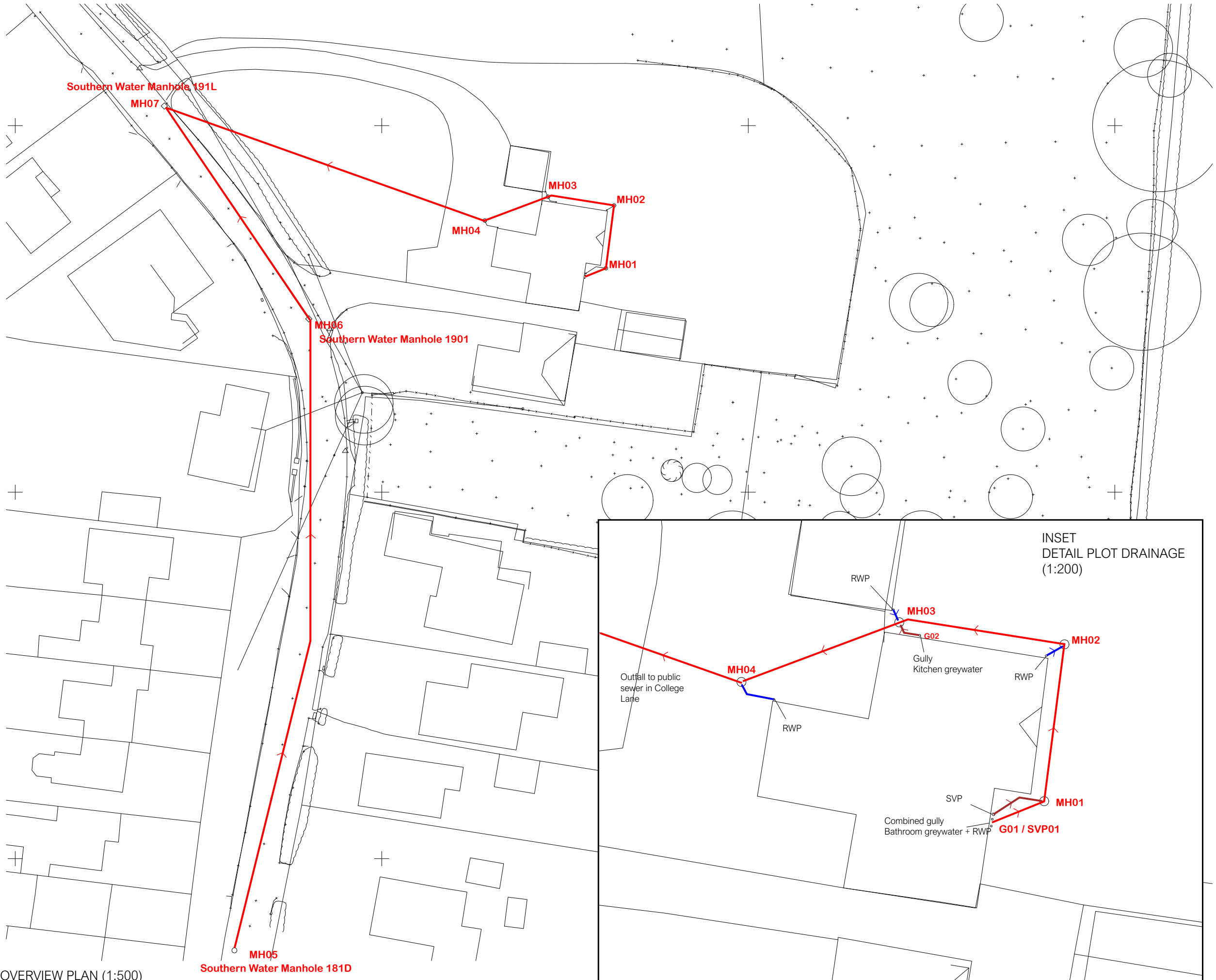
Appendix D

British Geological Survey 1:50,000 Geoindex Viewer

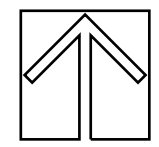


Appendix E

Existing Private Drainage Survey



Any discrepancies to be reported to Earlswood Homes.



- Surface Water
- Foul Water
- Combined

Arrows denote direction of flow

Rev	Date	Description



Date: April 2024	Drawn by: BCL
Scale: As specified @ A3	Checked:

Client:
Earlswood Homes

Project:
Land at 147 to 149
College Lane
Hurstpierpoint

Title:
Existing Drainage Survey Plan

Drawing:

OVERVIEW PLAN (1:500)

INSET
DETAIL PLOT DRAINAGE
(1:200)



G01 / SVP01
Rear elevation
SVP to bathroom
Combined gully pot taking bathroom greywater and rainwater downpipe from main roof of dwelling



MH01
Manhole chamber collecting G01 and SVP01



MH02
Inlet from MH01
Inlet from rainwater downpipe from main roof of dwelling



G02 / MH03
G02 in background - gully pot for kitchen greywater
MH03 in foreground



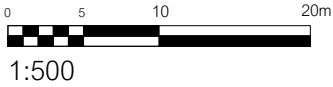
MH03
Inlet from MH02
Inlet from G02
Inlet from downpipe from garage



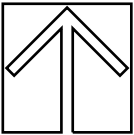
MH07
Public sewer manhole in College Lane



MH05
Grated public sewer manhole cover in College Lane



Any discrepancies to be reported to Earlswood Homes.



Rev	Date	Description



<u>Date:</u> April 2024	<u>Drawn by:</u> BCL
<u>Scale:</u> As specified @ A3	<u>Checked:</u> .

Client:
Earlswood Homes

Project:
Land at 147 to 149
College Lane
Hurstpierpoint

Title:
Existing Drainage Survey Photos

Drawing:

Appendix F

Southern Water Asset Location Plan



(c) Crown copyright and database rights 2024 Ordnance Survey 100031673 Date: 21/03/24 Scale: 1:1250 Map Centre: 529172,116918 Data updated: 12/01/24 Our Ref: 1427104 - 1 Wastewater Plan A3

The positions of pipes shown on this plan are believed to be correct, but Southern Water Services Ltd accept no responsibility in the event of inaccuracy. The actual positions should be determined on site. This plan is produced by Southern Water Services Ltd (c) Crown copyright and database rights 2024 Ordnance Survey 100031673. This map is to be used for the purposes of viewing the location of Southern Water plant only. Any other uses of the map data or further copies is not permitted.

WARNING: BAC pipes are constructed of Bonded Asbestos Cement.

WARNING: Unknown (UNK) materials may include Bonded Asbestos Cement.

pallen@motion.co.uk	
College Lane	
1eahur	



Appendix G

Whole Site Greenfield Runoff Rate

Calculated by:

Phil Allen

Site name:

College Lane

Site location:

Hurstpierpoint

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:

50.93733° N

Longitude:

0.16212° W

Reference:

876154165

Date:

Apr 08 2024 14:54

Runoff estimation approach

IH124

Site characteristics

Total site area (ha):

0.621

Methodology

Q_{BAR} estimation method:

Calculate from SPR and SAAR

SPR estimation method:

Calculate from SOIL type

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

Soil characteristics

	Default	Edited
SOIL type:	4	4
HOST class:	N/A	N/A
SPR/SPRHOST:	0.47	0.47

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

Hydrological characteristics

	Default	Edited
SAAR (mm):	875	875
Hydrological region:	7	7
Growth curve factor 1 year:	0.85	0.85
Growth curve factor 30 years:	2.3	2.3
Growth curve factor 100 years:	3.19	3.19
Growth curve factor 200 years:	3.74	3.74

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited
Q _{BAR} (l/s):	3.89	3.89
1 in 1 year (l/s):	3.31	3.31
1 in 30 years (l/s):	8.95	8.95
1 in 100 year (l/s):	12.42	12.42
1 in 200 years (l/s):	14.56	14.56

This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this tool are estimates of greenfield runoff rates. The use of these results is the responsibility of the users of this tool. No liability will be accepted by HR Wallingford, the Environment Agency, CEH, Hydrosolutions or any other organisation for the use of this data in the design or operational characteristics of any drainage scheme.

Appendix H

Environment Agency Flood Map for Planning

Flood map for planning

Your reference
<Unspecified>

Location (easting/northing)
529202/116932

Created
8 Apr 2024 10:13

Your selected location is in flood zone 1, an area with a low probability of flooding.

You will need to do a flood risk assessment if your site is **any of the following:**

- bigger than 1 hectare (ha)
- In an area with critical drainage problems as notified by the Environment Agency
- identified as being at increased flood risk in future by the local authority's strategic flood risk assessment
- at risk from other sources of flooding (such as surface water or reservoirs) and its development would increase the vulnerability of its use (such as constructing an office on an undeveloped site or converting a shop to a dwelling)

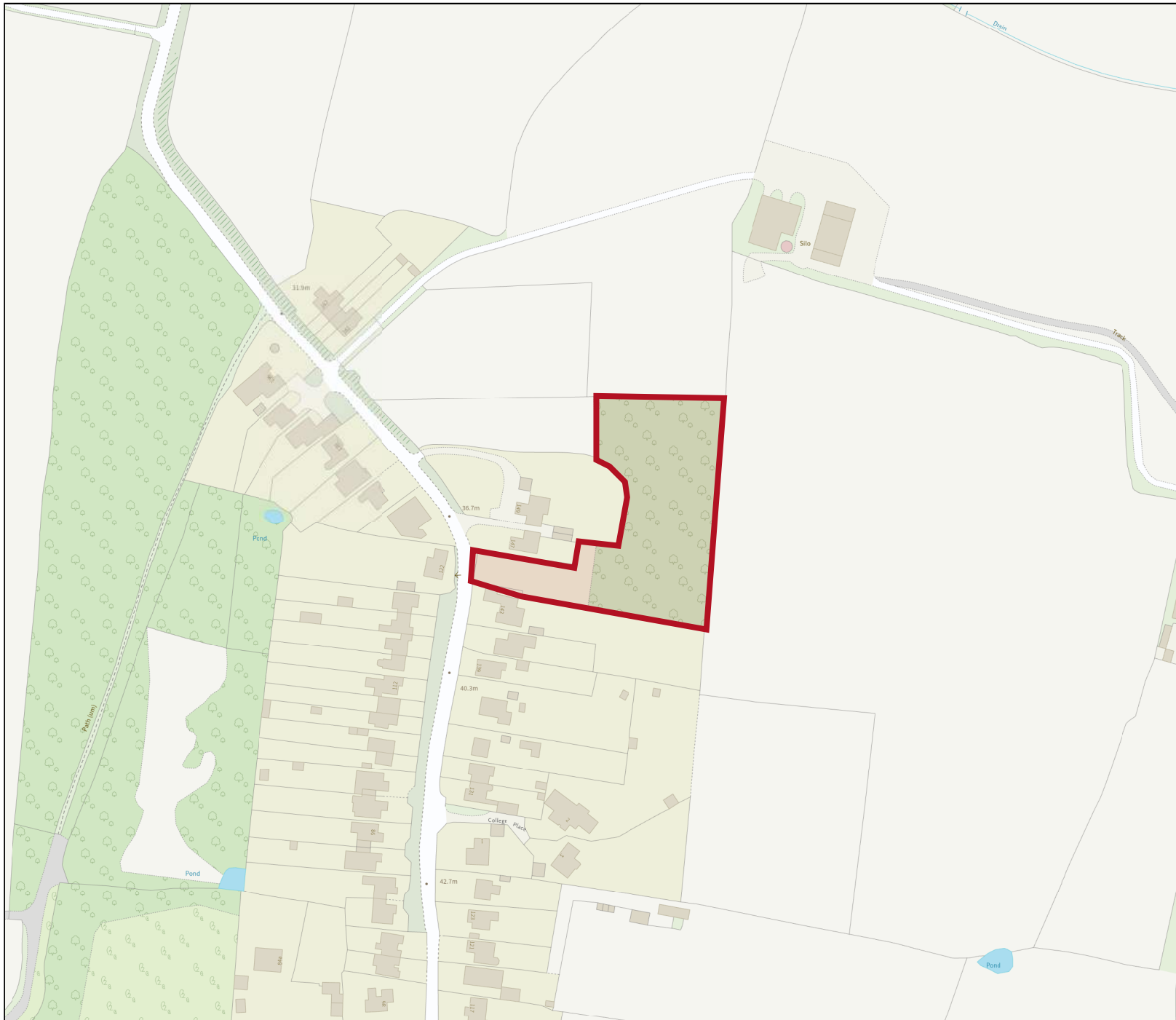
Notes

The flood map for planning shows river and sea flooding data only. It doesn't include other sources of flooding. It is for use in development planning and flood risk assessments.

This information relates to the selected location and is not specific to any property within it. The map is updated regularly and is correct at the time of printing.

Flood risk data is covered by the Open Government Licence **which** sets out the terms and conditions for using government data. <https://www.nationalarchives.gov.uk/doc/open-government-licence/version/3/>

Use of the address and mapping data is subject to Ordnance Survey public viewing terms under Crown copyright and database rights 2022 OS 100024198. <https://flood-map-for-planning.service.gov.uk/os-terms>



Flood map for planning

Your reference
<Unspecified>

Location (easting/northing)
529202/116932

Scale
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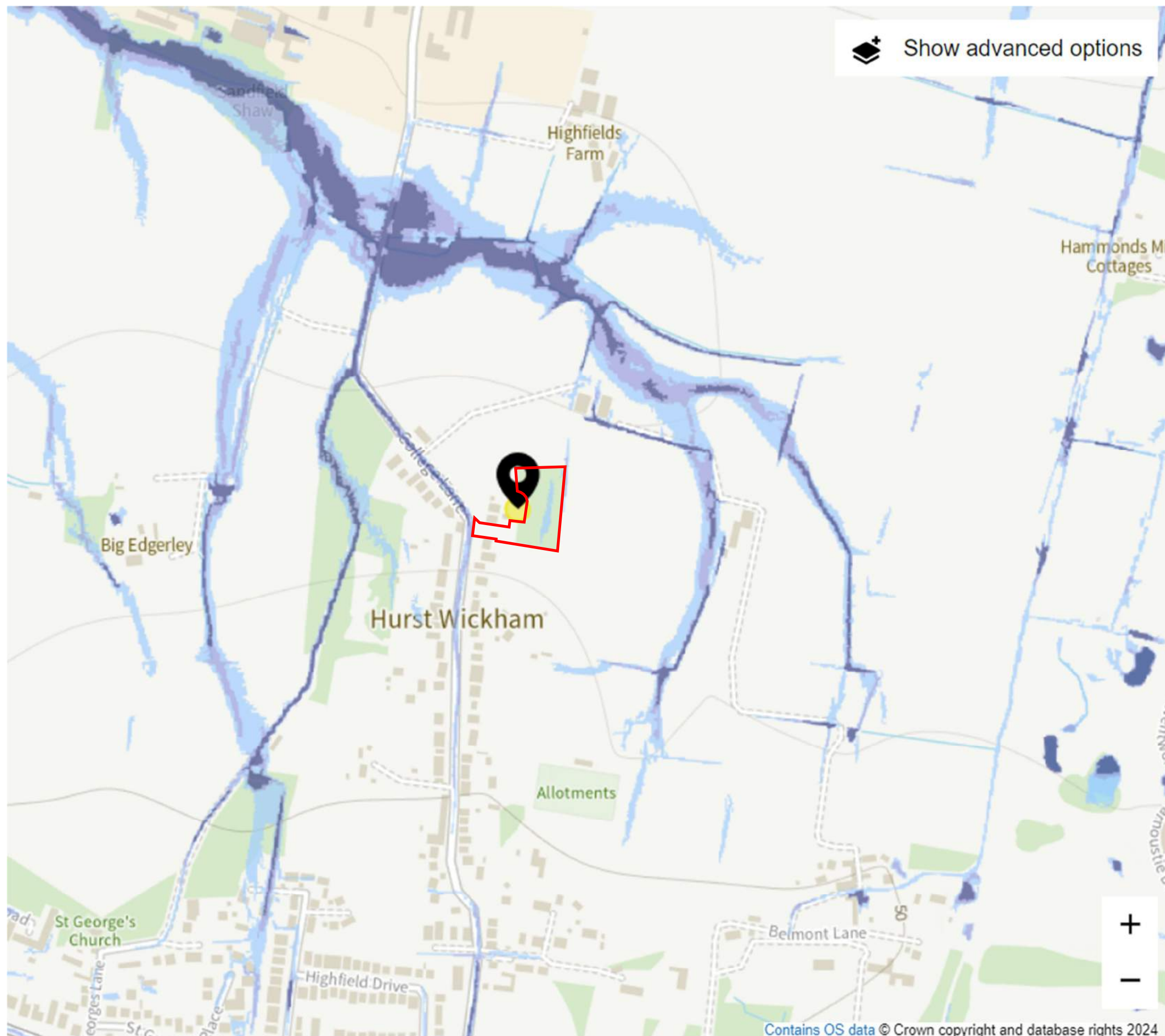
Created
8 Apr 2024 10:13

-  Selected area
-  Flood zone 3
-  Flood zone 2
-  Flood zone 1
-  Flood defence
-  Main river
-  Water storage area

0 20 40 60m

Appendix I

Environment Agency Risk of Flooding from Surface Water Map



Key

Surface water

-  Extent
-  High risk
More than 3.3% chance each year
-  Medium risk
Between 1% and 3.3% chance each year
-  Low risk
Between 0.1% and 1% chance each year

-  Depth

Map details

- ☒ Show flooding
- ☒  Selected address
-  15m boundary

Pause to updates of flood risk data

We have [paused updates to information about flood risk](#) from rivers and the sea and surface water while we get ready for new data.

Appendix J

Impermeable Areas Greenfield Runoff Rate

Calculated by:

Phil Allen

Site name:

College Lane

Site location:

Hurstpierpoint

Site Details

Latitude:

50.93705° N

Longitude:

0.16115° W

Reference:

151219168

Date:

Apr 09 2024 15:54

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Runoff estimation approach

IH124

Site characteristics

Total site area (ha):

0.211

Methodology

Q_{BAR} estimation method:

Calculate from SPR and SAAR

SPR estimation method:

Calculate from SOIL type

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

Soil characteristics

	Default	Edited
SOIL type:	4	4
HOST class:	N/A	N/A
SPR/SPRHOST:	0.47	0.47

Hydrological characteristics

	Default	Edited
SAAR (mm):	875	875
Hydrological region:	7	7
Growth curve factor 1 year:	0.85	0.85
Growth curve factor 30 years:	2.3	2.3
Growth curve factor 100 years:	3.19	3.19
Growth curve factor 200 years:	3.74	3.74

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited
Q _{BAR} (l/s):	1.32	1.32
1 in 1 year (l/s):	1.12	1.12
1 in 30 years (l/s):	3.04	3.04
1 in 100 year (l/s):	4.22	4.22
1 in 200 years (l/s):	4.95	4.95

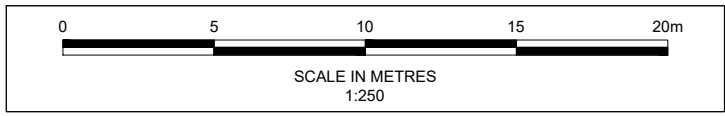
This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this tool are estimates of greenfield runoff rates. The use of these results is the responsibility of the users of this tool. No liability will be accepted by HR Wallingford, the Environment Agency, CEH, Hydrosolutions or any other organisation for the use of this data in the design or operational characteristics of any drainage scheme.

Appendix K

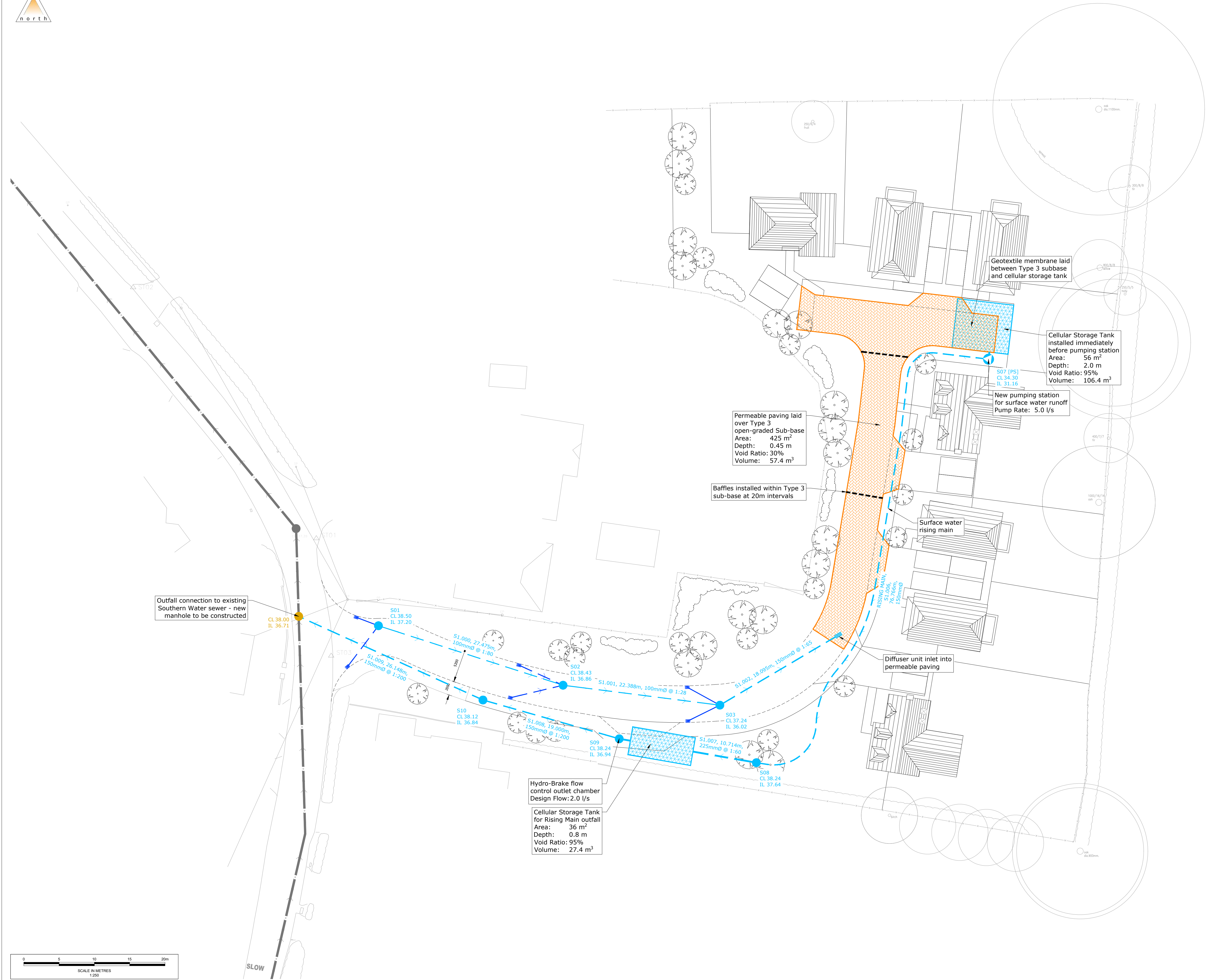
Drainage Strategy Plan



C:\Users\jhalley\Videos\SurfSite - Earlsur 2403069\Drawings\2403069-0501 [Drainage Strategy].dwg



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Notes

1. All levels and dimensions to be checked on site before any work commences. All dimensions in metres unless stated otherwise.
2. This drawing is based on survey information supplied by [add], and Motion cannot guarantee the accuracy of the data provided.
3. This drawing is based on OS mapping and Motion cannot guarantee the accuracy of the data.
4. Highway boundary information obtained from [add] and based on OS mapping, Motion cannot guarantee the accuracy of the data provided.
5. Motion accepts no liability for any vehicle specification errors or inaccuracies within the vehicle tracking software used / or it's vehicle libraries. The vehicles speeds used for the analysis are as follows: forward **mph / reversing **mph.

Legend

- Existing Southern Water Foul Sewer
- Proposed Surface Water Drain
- Proposed Surface Water Gully and Pipe Connection
- Proposed Surface Water Rising Main
- Proposed Surface Water Manhole
- Proposed Surface Water Pumping Station
- Proposed Diffuser Unit
- Proposed Permeable Paving
- Proposed Permeable Paving Baffle
- Proposed Cellular Storage Tank
- Proposed Foul Manhole - Outfall Connection to existing sewer network

P1	Revised Issue following client review	MH	MH	PA	19/04/2024
P0	First Issue	MH	MH	PA	12/04/2024
Rev.	Description	Drm	Chk	App	Date

Drawing Status:

FOR PLANNING
NOT FOR CONSTRUCTION

motion

Guildford - Reading - London
www.motion.co.uk

Client:
Earlswood Homes Ltd

Project:
Land off College Land
Hurstpierpoint

Title:
Proposed Drainage Strategy
Surface Water Drainage

Scale: 1:250 (@ A1)

Drawing:
2403069-0501

Revision:
P1

Appendix L

MicroDrainage Hydraulic Modelling Outputs

Motion		Page 1
84 North Street Guildford GU1 4AU		
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Innovyze	Network 2020.1.3	

STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Storm

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	100	PIMP (%)	100
M5-60 (mm)	20.000	Add Flow / Climate Change (%)	0
Ratio R	0.350	Minimum Backdrop Height (m)	0.200
Maximum Rainfall (mm/hr)	150	Maximum Backdrop Height (m)	1.500
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	0.400
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Soffits

Time Area Diagram for Storm

Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.000	4-8	0.000	8-12	0.000	12-16	0.153	16-20	0.067

Total Area Contributing (ha) = 0.220

Total Pipe Volume (m³) = 4.549

Network Design Table for Storm

- Indicates pipe length does not match coordinates
« - Indicates pipe capacity < flow

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S1.000	27.475	0.343	80.1	0.007	5.00	0.0	0.600	o	100	Pipe/Conduit	
S1.001	22.388	0.790	28.3	0.020	0.00	0.0	0.600	o	100	Pipe/Conduit	
S1.002	18.095	0.280	64.6	0.019	0.00	0.0	0.600	o	150	Pipe/Conduit	
S1.003	2.000#	1.840	1.1	0.033	0.00	0.0	0.600	o	100	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S1.000	146.63	5.53	37.200	0.007	0.0	0.0	0.0	0.86	6.8	2.6
S1.001	144.22	5.79	36.857	0.026	0.0	0.0	0.0	1.46	11.4	10.3
S1.002	142.05	6.03	36.017	0.045	0.0	0.0	0.0	1.25	22.1	17.3
S1.003	142.02	6.03	35.660	0.078	0.0	0.0	0.0	7.48	58.8	30.0

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Online Controls for Storm Hydro-Brake® Optimum Manhole: S9, DS/PN: S1.008, Volume (m³): 1.9 Unit Reference MD-SHE-0070-2000-0800-2000 Design Head (m)0.800 Design Flow (l/s)2.0 Flush-Flo™Calculated ObjectiveMinimise upstream storage ApplicationSurface Sump AvailableYes Diameter (mm)70 Invert Level (m)36.936 Minimum Outlet Pipe Diameter (mm)100 Suggested Manhole Diameter (mm)1200 Control Points Head (m) Flow (l/s) Design Point (Calculated)0.8002.0 Flush-Flo™0.2402.0 Kick-Flo®0.5041.6 Mean Flow over Head Range-1.7 The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated <table><tr><td>Depth (m)</td><td>Flow (l/s)</td><td>Depth (m)</td><td>Flow (l/s)</td><td>Depth (m)</td><td>Flow (l/s)</td><td>Depth (m)</td><td>Flow (l/s)</td></tr><tr><td>0.100</td><td>1.8</td><td>1.200</td><td>2.4</td><td>3.000</td><td>3.7</td><td>7.000</td><td>5.5</td></tr><tr><td>0.200</td><td>2.0</td><td>1.400</td><td>2.6</td><td>3.500</td><td>3.9</td><td>7.500</td><td>5.6</td></tr><tr><td>0.300</td><td>2.0</td><td>1.600</td><td>2.7</td><td>4.000</td><td>4.2</td><td>8.000</td><td>5.8</td></tr><tr><td>0.400</td><td>1.9</td><td>1.800</td><td>2.9</td><td>4.500</td><td>4.4</td><td>8.500</td><td>6.0</td></tr><tr><td>0.500</td><td>1.6</td><td>2.000</td><td>3.0</td><td>5.000</td><td>4.7</td><td>9.000</td><td>6.2</td></tr><tr><td>0.600</td><td>1.8</td><td>2.200</td><td>3.2</td><td>5.500</td><td>4.9</td><td>9.500</td><td>6.3</td></tr><tr><td>0.800</td><td>2.0</td><td>2.400</td><td>3.3</td><td>6.000</td><td>5.1</td><td></td><td></td></tr><tr><td>1.000</td><td>2.2</td><td>2.600</td><td>3.4</td><td>6.500</td><td>5.3</td><td></td><td></td></tr></table> <tr><td colspan="3">©1982-2020 Innovyze</td></tr>			Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	0.100	1.8	1.200	2.4	3.000	3.7	7.000	5.5	0.200	2.0	1.400	2.6	3.500	3.9	7.500	5.6	0.300	2.0	1.600	2.7	4.000	4.2	8.000	5.8	0.400	1.9	1.800	2.9	4.500	4.4	8.500	6.0	0.500	1.6	2.000	3.0	5.000	4.7	9.000	6.2	0.600	1.8	2.200	3.2	5.500	4.9	9.500	6.3	0.800	2.0	2.400	3.3	6.000	5.1			1.000	2.2	2.600	3.4	6.500	5.3			©1982-2020 Innovyze		
Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)																																																																						
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Offline Controls for Storm Pump Manhole: S7, DS/PN: S1.006, Loop to PN: S1.007 Invert Level (m) 31.157 <table><tr><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th></tr><tr><td>0.200</td><td>5.0000</td><td>1.800</td><td>5.0000</td><td>3.400</td><td>5.0000</td><td>5.000</td><td>5.0000</td></tr><tr><td>0.400</td><td>5.0000</td><td>2.000</td><td>5.0000</td><td>3.600</td><td>5.0000</td><td>5.200</td><td>5.0000</td></tr><tr><td>0.600</td><td>5.0000</td><td>2.200</td><td>5.0000</td><td>3.800</td><td>5.0000</td><td>5.400</td><td>5.0000</td></tr><tr><td>0.800</td><td>5.0000</td><td>2.400</td><td>5.0000</td><td>4.000</td><td>5.0000</td><td>5.600</td><td>5.0000</td></tr><tr><td>1.000</td><td>5.0000</td><td>2.600</td><td>5.0000</td><td>4.200</td><td>5.0000</td><td>5.800</td><td>5.0000</td></tr><tr><td>1.200</td><td>5.0000</td><td>2.800</td><td>5.0000</td><td>4.400</td><td>5.0000</td><td>6.000</td><td>5.0000</td></tr><tr><td>1.400</td><td>5.0000</td><td>3.000</td><td>5.0000</td><td>4.600</td><td>5.0000</td><td></td><td></td></tr><tr><td>1.600</td><td>5.0000</td><td>3.200</td><td>5.0000</td><td>4.800</td><td>5.0000</td><td></td><td></td></tr></table>			Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	0.200	5.0000	1.800	5.0000	3.400	5.0000	5.000	5.0000	0.400	5.0000	2.000	5.0000	3.600	5.0000	5.200	5.0000	0.600	5.0000	2.200	5.0000	3.800	5.0000	5.400	5.0000	0.800	5.0000	2.400	5.0000	4.000	5.0000	5.600	5.0000	1.000	5.0000	2.600	5.0000	4.200	5.0000	5.800	5.0000	1.200	5.0000	2.800	5.0000	4.400	5.0000	6.000	5.0000	1.400	5.0000	3.000	5.0000	4.600	5.0000			1.600	5.0000	3.200	5.0000	4.800	5.0000		
Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)																																																																			
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Innovyze			Network 2020.1.3		
<u>Storage Structures for Storm</u>					
<u>Porous Car Park Manhole: S5, DS/PN: S1.004</u>					
Infiltration Coefficient Base (m/hr)		0.00000	Width (m)		10.0
Membrane Percolation (mm/hr)		1000	Length (m)		41.0
Max Percolation (l/s)		113.9	Slope (1:X)		40.0
Safety Factor		2.0	Depression Storage (mm)		5
Porosity		0.30	Evaporation (mm/day)		3
Invert Level (m)		33.820	Cap Volume Depth (m)		0.450
<u>Cellular Storage Manhole: S7, DS/PN: S1.006</u>					
Invert Level (m)		31.157	Safety Factor		2.0
Infiltration Coefficient Base (m/hr)		0.00000	Porosity		0.95
Infiltration Coefficient Side (m/hr)		0.00000			
Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	56.0	56.0	2.001	0.0	116.0
2.000	56.0	116.0			
<u>Cellular Storage Manhole: S9, DS/PN: S1.008</u>					
Invert Level (m)		36.936	Safety Factor		2.0
Infiltration Coefficient Base (m/hr)		0.00000	Porosity		0.95
Infiltration Coefficient Side (m/hr)		0.00000			
Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	36.0	36.0	0.801	0.0	56.8
0.800	36.0	56.8			
<u>Volume Summary (Static)</u>					
Length Calculations based on Centre-Centre					
Pipe Number	USMH Name	Manhole Volume (m³)	Pipe Volume (m³)	Storage Structure Volume (m³)	Total Volume (m³)
S1.000	S1	1.470	0.216	0.000	1.686
S1.001	S2	1.779	0.176	0.000	1.955
S1.002	S3	1.383	0.320	0.000	1.703
S1.003	S4	1.470	0.016	0.000	1.486
S1.004	S5	0.656	1.150	19.170	20.976
S1.005	S6	1.463	0.282	0.000	1.745
S1.006	S7	3.555	1.166	106.418	111.138
S1.007	S8	0.679	0.426	0.000	1.105
S1.008	S9	1.475	0.336	27.371	29.182
S1.009	S10	1.447	0.462	0.000	1.909
Total		15.377	4.549	152.959	172.884
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1 year Return Period Summary of Critical Results by Maximum Level (Rank 1)
for Storm

Simulation Criteria

Areal Reduction Factor 1.000
Hot Start (mins) 0
Hot Start Level (mm) 0
Manhole Headloss Coeff (Global) 0.500
Foul Sewage per hectare (l/s) 0.000

Additional Flow - % of Total Flow 0.000
MADD Factor * 10m³/ha Storage 2.000
Inlet Coeffiecient 0.800
Flow per Person per Day (l/per/day) 0.000

Number of Input Hydrographs 0
Number of Online Controls 1
Number of Offline Controls 1

Number of Storage Structures 3
Number of Time/Area Diagrams 0
Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR
Region England and Wales Cv (Summer)
M5-60 (mm) 20.000 Cv (Winter)

Ratio R 0.350
0.750
0.840

Margin for Flood Risk Warning (mm) 300.0
Analysis Timestep
DTS Status

DVD Status ON
Fine Inertia Status ON
OFF

Profile(s)

Duration(s) (mins)
Return Period(s) (years)
Climate Change (%)

Summer and Winter
15, 30, 60, 120, 240, 360, 480, 960, 1440
1, 2, 30, 100
0, 0, 40, 45

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surchage	First (Y) Flood	First (Z) Overflow	Overflow Act.
S1.000	S1	15 Winter	1	+0%	100/15 Summer			
S1.001	S2	15 Winter	1	+0%	30/15 Summer			
S1.002	S3	15 Winter	1	+0%	100/15 Summer			
S1.003	S4	15 Winter	1	+0%	100/15 Summer			
S1.004	S5	15 Winter	1	+0%				
S1.005	S6	15 Winter	1	+0%	100/360 Winter			
S1.006	S7	120 Winter	1	+0%	1/15 Summer		1/15 Summer	72
S1.007	S8	30 Winter	1	+0%				
S1.008	S9	240 Winter	1	+0%	1/30 Winter			
S1.009	S10	60 Winter	1	+0%				

Water Surcharged Flooded					Half Drain		Pipe	
PN	US/MH Name	Level (m)	Depth (m)	Volume (m³)	Flow / Overflow Cap.	Time (mins)	Flow (l/s)	Status
S1.000	S1	37.224	-0.076	0.000	0.13		0.9	OK
S1.001	S2	36.893	-0.064	0.000	0.28		3.1	OK
S1.002	S3	36.068	-0.099	0.000	0.25		5.2	OK
S1.003	S4	35.693	-0.067	0.000	0.23		8.8	OK
S1.004	S5	33.883	-0.237	0.000	0.10	7	20.8	OK

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1 year Return Period Summary of Critical Results by Maximum Level (Rank 1)
for Storm

PN	US/MH Name	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status
S1.005	S6	33.094	-0.212	0.000	0.19			20.8	OK
S1.006	S7	31.420	0.113	0.000	0.00	5.0	56	0.0	SURCHARGED
S1.007	S8	37.680	-0.185	0.000	0.07			4.1	OK
S1.008	S9	37.347	0.261	0.000	0.17		108	2.0	SURCHARGED
S1.009	S10	36.882	-0.109	0.000	0.17			2.0	OK

US/MH Level
PN Name Exceeded

S1.000 S1
S1.001 S2
S1.002 S3
S1.003 S4
S1.004 S5
S1.005 S6
S1.006 S7
S1.007 S8
S1.008 S9
S1.009 S10

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[illegible]

[illegible]

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Appendix M

Volume 4 of the Modified Rational Method

**The Standing Technical Committee on Sewers and Water Mains
WORKING PARTY ON THE HYDRAULIC DESIGN OF STORM SEWERS**

Design and analysis of urban storm drainage

THE WALLINGFORD PROCEDURE

Volume 4

The Modified Rational Method

**Department of the Environment
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No 31**

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Design and analysis of urban storm drainage

THE WALLINGFORD PROCEDURE

Principles, methods and practice (Volume 1)

Describes the background and development of the Wallingford Procedure for the design and analysis of urban storm drainage for use in the UK. Four methods, incorporated within an economic framework are used to design sewer dimensions, depths and gradients and to simulate the behaviour of a system under surcharge and with surface flooding.

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Gives detailed instructions on the data to be collected for a sewer system and how to prepare the data for the computer programs implementing the procedure for the UK.

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Modified Rational Method (Volume 4)

Presents a hand calculation for a modified version of the Rational Method for use in the UK. This volume is suitable for those designing or analysing small sewer systems or for planning urban drainage schemes.

ISBN 0 946466 041.

Programmer's manual (Volume 5)

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MicroWASSP User's Guide (Volume 6)

Gives instructions on the use of the micro-computer version of the computer programs implementing the Procedure.

WASSPOS User's Guide (Volume 7)

The parallel volume to Volume 2 for use of the computer programs in applications outside the UK.

Course Notes

Contains extensive notes on lectures given in the course run by Hydraulics Research for users of the Procedure. The notes include details not described in Volume 1.

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The Modified Rational Method

1. Background

The Rational (or Lloyd-Davies) method is in widespread use and provides a convenient and easily understood tool for design. Studies^(1,2) have shown the variable accuracy of the method as normally applied in the UK, and recent research has suggested ways in which the method could be improved. A Modified Rational Method has therefore been included in the procedure for the design and analysis of storm drainage networks produced for the DoE/NWC Working Party on the Hydraulic Design of Storm Sewers.

The purpose of this brief volume is to explain the application by hand calculation of the Modified Rational Method. The modifications are concerned with the values used for the coefficient and the time of entry. A simple method for determining the appropriate rainfall intensities for any location in the UK is also recommended. Details of the other methods in the procedure, and of the relevant computer programs, are available in other volumes of this report.

2. The Rational Formula

The method gives the peak discharge from the equation:

$$Q_p = CiA \quad \dots \dots (1)$$

where Q_p is the peak discharge

C is a dimensionless coefficient

i is the average rainfall intensity during the time of concentration

and A is the contributing catchment area.

Additional factors may be necessary to allow for the dimensions used. If Q_p , i and A are expressed in l/s, mm/hr and ha respectively, equation 1 becomes

$$Q_p = \frac{CiA}{0.36} = 2.78 CiA \quad \dots \dots (2)$$

3. Scope of the method

The method may be used either to size diameters of pipes for a specified return period of flow in a storm sewer system of given layout and gradients, or to estimate peak discharges in an existing system for given rainfall conditions. It provides only a value of the peak runoff discharge; the hand calculation presented here cannot deal with sewer structures such as storm overflows. A computer version which can deal with storm overflows is available as part of the larger procedure. Networks incorporating more complex features such as storage tanks or pumping stations should be analysed using one of the hydrograph methods available.

Tests have shown³ that the Modified Rational Method is as accurate for the determination of peak runoff discharge as some more sophisticated urban runoff methods. These tests were limited to urban catchments up to 150ha in area with times of concentration up to about 30 minutes and outfall pipe diameters up to about one metre. The slope and distribution of impervious area in these catchments were reasonably uniform. The accuracy of the method when applied to larger or more irregular catchments is not known, and therefore the method cannot be positively recommended outside these limits.

4. Summary of data requirements

The data required for applying the method are:

- (a) Details of the proposed or existing catchment, including the impervious area contributing to each pipe length and the gradient of each pipe length.
- (b) A tabulation or graph of rainfall data relating average intensity to duration and the required return period(s). Such data for a specific location in the UK may be obtained from the Meteorological Office (Met 08, London Road, Bracknell) or may be derived by a simple hand calculation described in the accompanying appendix. The relevant return period is usually specified by the drainage authority.
- (c) Suitable values for the coefficient C and the time of concentration t_c , determination of which is described below:

Determination of C

The coefficient C may be regarded as a combination of two separate coefficients:

$$C = C_v C_R \quad \dots \dots (3)$$

where C_v is the volumetric runoff coefficient
and C_R is a dimensionless routing coefficient.

Value of C_v

The volumetric runoff coefficient C_v may be defined as the proportion of the rainfall on the catchment which appears as surface runoff in the storm drainage system.

The recommended value of C_v is affected by whether the whole catchment is being considered (impervious areas and pervious areas), or the impervious areas alone. For the purpose of this volume it is assumed that impervious areas (paved and roof) alone will be used; alternative approaches are described in volume 1.

An extensive study of runoff data from sewered urban catchments⁴ showed that the volume of runoff was related to the impervious area, the soil type and the catchment wetness. An approximate result may be obtained by assuming that the runoff derives from a proportion of the impervious area (paved and roof), the proportion varying according to soil type. On this basis the overall average value of C_v is about 0.75, ranging from about 0.6 on catchments with rapidly-draining soils to about 0.9 on catchments with heavy soils.

These values reflect the loss of some rainfall from impervious areas through cracks and into depressions and by drainage onto pervious (unpaved) areas. Similarly, any runoff from the pervious areas onto the impervious areas is also incorporated.

The above values of C_v should therefore be used in conjunction with the total impervious area (paved and roof) intended to drain to the storm sewer system.

Alternative methods of determining C_v which take account of specific soil characteristics and regional variations in catchment wetness are described in Volume 1 of this report and in reference 4.

Value of C_R

The routing coefficient C_R depends on the shape of the time-area diagram and on the variation of rainfall within the time of concentration. Examination of typical time-area diagrams, rainfall profiles and rainfall-runoff data led to the recommendation of a constant value for C_R of 1.30 for both design and simulation.

Equation (2) may therefore be rewritten

$$Q_p = 2.78 \times 1.30 C_v i A = 3.61 C_v i A \quad \dots\dots(4)$$

Determination of time of concentration

The time of concentration t_c is defined by:

$$t_c = t_e + t_r$$

where t_e is the time of entry

and t_r is the time of flow through the pipe system to the point under consideration.

The time of entry t_e may be regarded as representing the delay and attenuation of the flow over the ground surface. It has the effect of reducing the calculated discharge, since an increase in the time of entry and, consequently, in the time of concentration reduces the corresponding rainfall intensity. Recent research⁵ has shown that times of entry of 8 to 12 minutes are common amongst frequently occurring storms. However the data set used in this exercise was biased towards small rainfall events equivalent to a return period of a few weeks or months. For larger return periods, times of entry were developed which gave surface runoff attenuation similar to that given by the surface runoff part of the more sophisticated hydrograph methods described in Volume 1.

This analysis led to the following recommended values for the time of entry:

<i>Return period</i>	<i>Time of entry (minutes)</i>
5 years	3 – 6
2 years	4 – 7
1 year	4 – 8
1 month	5 – 10

For each return period the larger times of entry are applicable to large, flat subcatchments (area greater than 400m², slope less than 1 in 50) and the smaller values to small, steep subcatchments (area less than 200m², slope greater than 1 in 30). Note that these values of area and slope refer to the subcatchments contributing to each pipe length.

The time of flow t_r through the pipe system may be determined from the pipe full velocity given in design tables⁶. This velocity gives a good approximation to the actual water velocity at all depths likely to occur under design conditions.

5. Application of the method

Given the pipe network and rainfall intensity data and suitable values for C_v and t_e as described above, the Modified Rational Method is applied as described below. A blank copy of a suitable calculation sheet is included in this volume, and may be copied for use.

For ease of reference, the pipe network is given a reference system of branch numbers (b) and pipe numbers (p) such that each pipe has a unique reference b.p. Steps 1 to 8 are repeated for each pipe in the network.

1. If a new pipeline is being designed an initial diameter equal to the minimum permitted diameter is assumed. Further downstream an initial diameter equal to the diameter of the largest incoming pipe at the upstream manhole is assumed.
2. The pipe full velocity is obtained from design tables⁶ and the time of flow along the pipe is determined.

3. The time of concentration is taken as the cumulative time of flow plus a time of entry. The time of flow through long lengths of carrier pipe (i.e. pipes with no contributing sub-catchment) should not be included in the calculation of the time of concentration for that pipe. It should, however, be included in the calculations for any subsequent pipes downstream.
4. A rainfall intensity corresponding to the time of concentration and the required return period is read from the rainfall data.
5. The cumulative impervious area is determined. It is assumed that additional area is added at the upstream end of the pipe length to which it contributes.
6. The peak discharge is calculated from equation (4). Any dry weather flow is added to give the total discharge. Dry weather flow may be determined from local information or standard calculation methods.
7. If the pipe is being designed, the smallest available diameter which will convey Q_p , is determined from design tables⁶. If this diameter is equal to or less than the diameter originally assumed, the diameter given by the design tables is accepted (if permitted by local restrictions on minimum diameter and non-decreasing diameter). Otherwise, steps 1 to 7 are repeated using a larger assumed diameter.
8. Flow velocities in the pipeline should be checked from design tables⁶ and compared with minimum velocity requirements. If the pipeline is being designed, the diameter or gradient may be adjusted if necessary. Adjustment of the gradient may affect the gradients of adjacent pipes.
9. For the pipe downstream of a junction the cumulative time of flow should be determined along the branch with the longest time of concentration. In exceptional circumstances this may lead to a lower calculated discharge than that from a major branch entering the upstream junction. In these circumstances the design discharge should not be reduced below the largest value entering the junction. This check may have to be repeated for several subsequent pipe lengths downstream.

6. References

1. P. J. Colyer. Performance of storm drainage simulation models, Proc ICE, 63, Part 2, June 1977, pp 293-309.
2. C. H. R. Kidd and J. C. Packman. Selection of design storm and antecedent condition for urban drainage design, Institute of Hydrology, Report No 61, 1979.
3. R. W. Pethick. Comparative testing of urban drainage models, Hydraulics Research Station report, in preparation.
4. C. H. R. Kidd and M. J. Lowing. The Wallingford urban subcatchment model, Institute of Hydrology, Report No. 60, 1979.
5. C. H. R. Kidd (ed). Rainfall-runoff processes over urban surfaces, Proceedings of an International Workshop, Institute of Hydrology, Report No. 53, 1978.
6. Hydraulics Research Station. Tables for the hydraulic design of pipes, Third Edition, HMSO, 1977.

APPENDIX

Calculation of rainfall intensities for any location in the United Kingdom

Introduction

The following manual method for calculating rainfall intensities for urban drainage design is reproduced from volume 1 of this report. The method was developed by the Meteorological Office as a simplified version of a computerised method; output from the computerised version may be obtained for a specific location either from the Meteorological Office (Met 08, London Road, Bracknell) or by application of the computer programs for storm drainage design and analysis.

Summary of the method

The manual method permits the calculation of rainfall intensities for durations between 5 minutes and 48 hours, and return periods between one year and 100 years. A graph of intensity against duration may be produced from about ten values at each required return period. The steps involved are:

1. Read from maps the value of the five year – 60 minute rainfall depth (described by the notation M5-60min) and the ratio (r) of the five year – 60 min rainfall depth to the five year – two day rainfall depth.
2. Determine the rainfall depths of five year return period for all required durations (described as M5-D).
3. Convert the five year rainfall depths to rainfall depths of the alternative return period(s) required (MT-D).
4. Convert the rainfall depths into point intensities.
5. Apply an areal reduction factor if required.

Application of the method

1. Determination of M5-60 min and r

Values of M5-60min and r are obtained from Figures A.1 and A.2. M5-60min should be estimated to the nearest mm (or half mm in areas of weak gradient); r should be estimated to ± 0.01 .

2. Determination of M5-D

M5-D is obtained from the relationship:

$$M5-D = Z1(M5-60min)$$

The factor Z1 is read from Figures A.3a or A.3b for values of r between 0.12 and 0.45 and for durations between 5 minutes and 48 hours. Z1 should be read from the graphs to an accuracy of about 0.01.

3. Determination of MT-D

MT-D is obtained from the relationship:

$$MT-D = Z2(M5-D)$$

The factor Z2 is read from Table A1 for England and Wales and Table A2 for Scotland and Northern Ireland.

4. Determination of point rainfall intensities

The rainfall intensity i of return period T and duration D is obtained from:

$$i = \frac{MT-D}{D}$$

5. Application of areal reduction factor

The intensities determined in the previous step are multiplied by the appropriate areal reduction factor read from Figure A.4. In most urban drainage catchments the areal reduction factor will be greater than 0.9.

Example

The following example illustrates the use of the method to provide the two year 30 minute rainfall intensity for a location in Oxford (GR 45002100)

1. From Figure A.1 $M5-60\text{min} = 20 \text{ mm}$
from Figure A.2 $r = 0.42$
2. From Figure A.3b $Z1 = 0.79$
therefore $M5-30 \text{ min} = 0.79 \times 20 = 15.8 \text{ mm}$
3. From Table A1, for two year return period
 $Z2 = 0.80$
therefore $M2-30\text{min} = 0.80 \times 15.8 = 12.64 \text{ mm}$
4. Average point intensity $= \frac{12.64}{30/60} = 25.3 \text{ mm/hr}$
5. If the catchment with a 30 minute time of concentration has a total area of 2 km^2 ,
Figure A.4 gives an areal reduction factor of about 0.94.
therefore average areal intensity $= 0.94 \times 25.3 = 23.8 \text{ mm/hr}$.

Table A1: Relationship between rainfall of return period T(MT) and M5 – England and Wales (ratio Z2)

M5 Rainfall mm	M1	M2	M3	M4	M5	M10	M20	M50	M100
5	0.62	0.79	0.89	0.97	1.02	1.19	1.36	1.56	1.79
10	0.61	0.79	0.90	0.97	1.03	1.22	1.41	1.65	1.91
15	0.62	0.80	0.90	0.97	1.03	1.24	1.44	1.70	1.99
20	0.64	0.81	0.90	0.97	1.03	1.24	1.45	1.73	2.03
25	0.66	0.82	0.91	0.97	1.03	1.24	1.44	1.72	2.01
30	0.68	0.83	0.91	0.97	1.03	1.22	1.42	1.70	1.97
40	0.70	0.84	0.92	0.97	1.02	1.19	1.38	1.64	1.89
50	0.72	0.85	0.93	0.98	1.02	1.17	1.34	1.58	1.81
75	0.76	0.87	0.93	0.98	1.02	1.14	1.28	1.47	1.64
100	0.78	0.88	0.94	0.98	1.02	1.13	1.25	1.40	1.54
150	0.78	0.88	0.94	0.98	1.01	1.12	1.21	1.33	1.45
200	0.78	0.88	0.94	0.98	1.01	1.11	1.19	1.30	1.40

Table A2: Relationship between rainfall of return period T(MT) and M5 – Scotland and Northern Ireland (ratio Z2)

M5 Rainfall mm	M1	M2	M3	M4	M5	M10	M20	M50	M100
5	0.67	0.82	0.91	0.98	1.02	1.17	1.35	1.62	1.86
10	0.68	0.82	0.91	0.98	1.03	1.19	1.39	1.69	1.97
15	0.69	0.83	0.91	0.97	1.03	1.20	1.39	1.70	1.98
20	0.70	0.84	0.92	0.97	1.02	1.19	1.38	1.66	1.93
25	0.71	0.84	0.92	0.98	1.02	1.18	1.37	1.64	1.89
30	0.72	0.85	0.92	0.98	1.02	1.18	1.36	1.61	1.85
40	0.74	0.86	0.93	0.98	1.02	1.17	1.34	1.56	1.77
50	0.75	0.87	0.93	0.98	1.02	1.16	1.30	1.52	1.72
75	0.77	0.88	0.94	0.98	1.02	1.14	1.27	1.45	1.62
100	0.78	0.88	0.94	0.98	1.02	1.13	1.24	1.40	1.54
150	0.79	0.89	0.94	0.98	1.02	1.11	1.20	1.33	1.45
200	0.80	0.89	0.95	0.99	1.01	1.10	1.18	1.30	1.40

Modified Rational Method

Calculation sheet

SCHEME

1 Pipe number	2 Pipe length (m)	3 Pipe gradient	4 Assumed diameter (mm or m)	5 Pipefull velocity (m.s)	6 Time of flow (min) <u>(2)</u> 60 × (5)	7 Time of concentration (min)	8 Rainfall intensity (mm/hr)

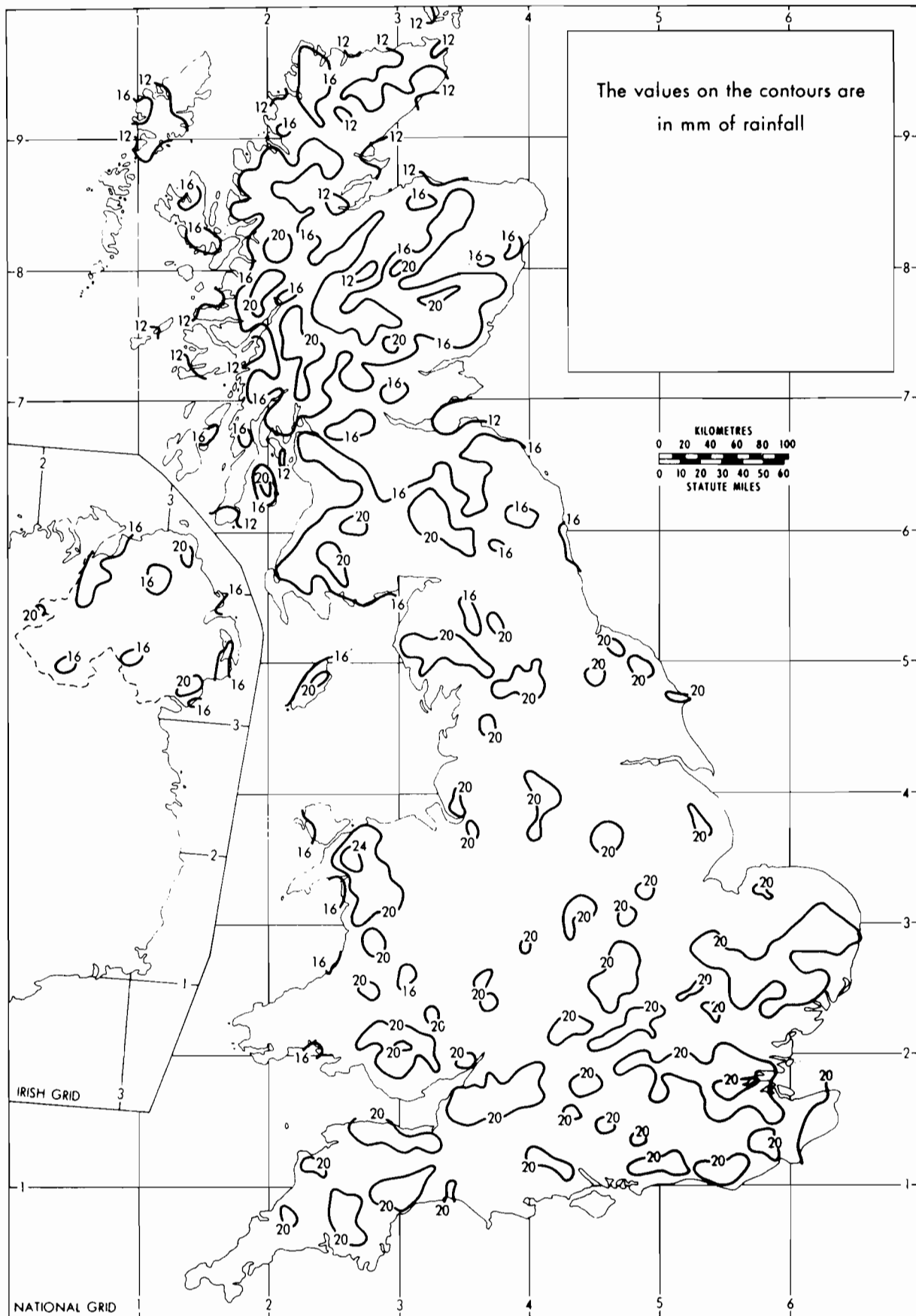
Rainfall return period..... years

Pipe roughness..... mm

Time of entry min

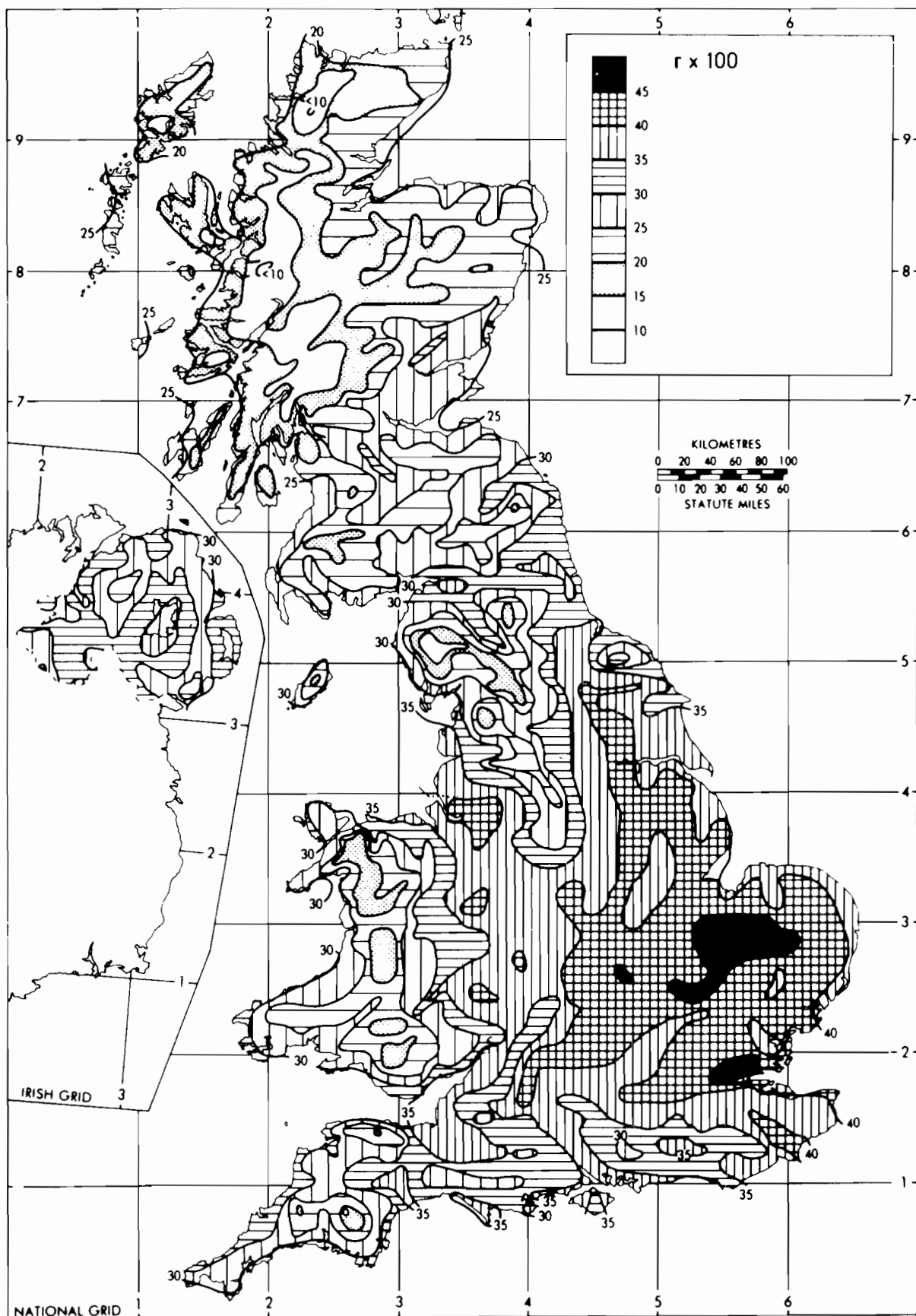
Coefficient C_v

9 Impervious area (ha)	10 Cumulative impervious area (ha)	11 Calculated discharge (l/s) $3.61C_v(8)(10)$	12 Dry-weather flow (l/s)	13 Total discharge (l/s) (11) + (12)	14 Required diameter (mm or m)	15 Comments



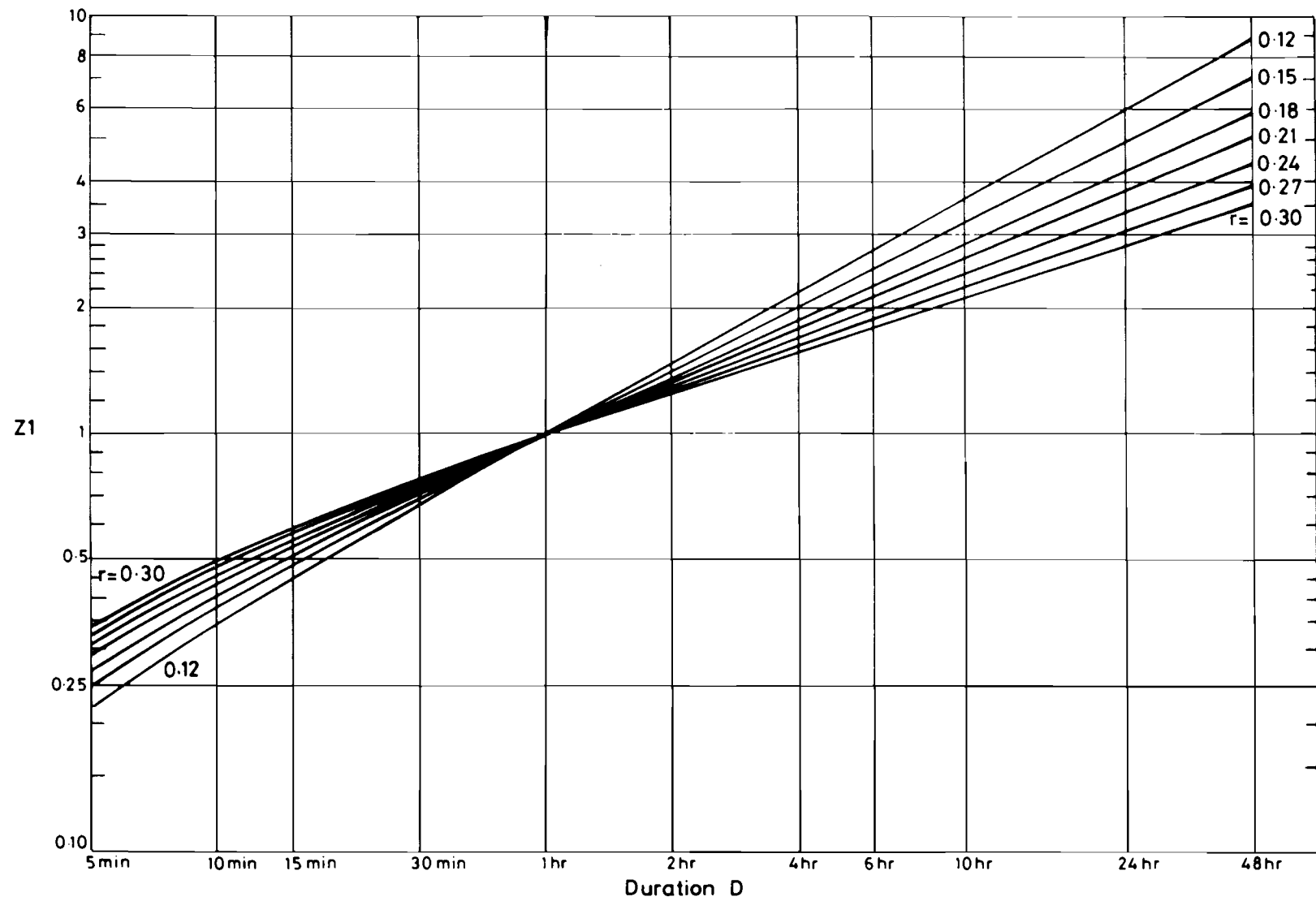
Rainfall depths of five year return period and 60 minutes duration (M5—60 min)

Fig.

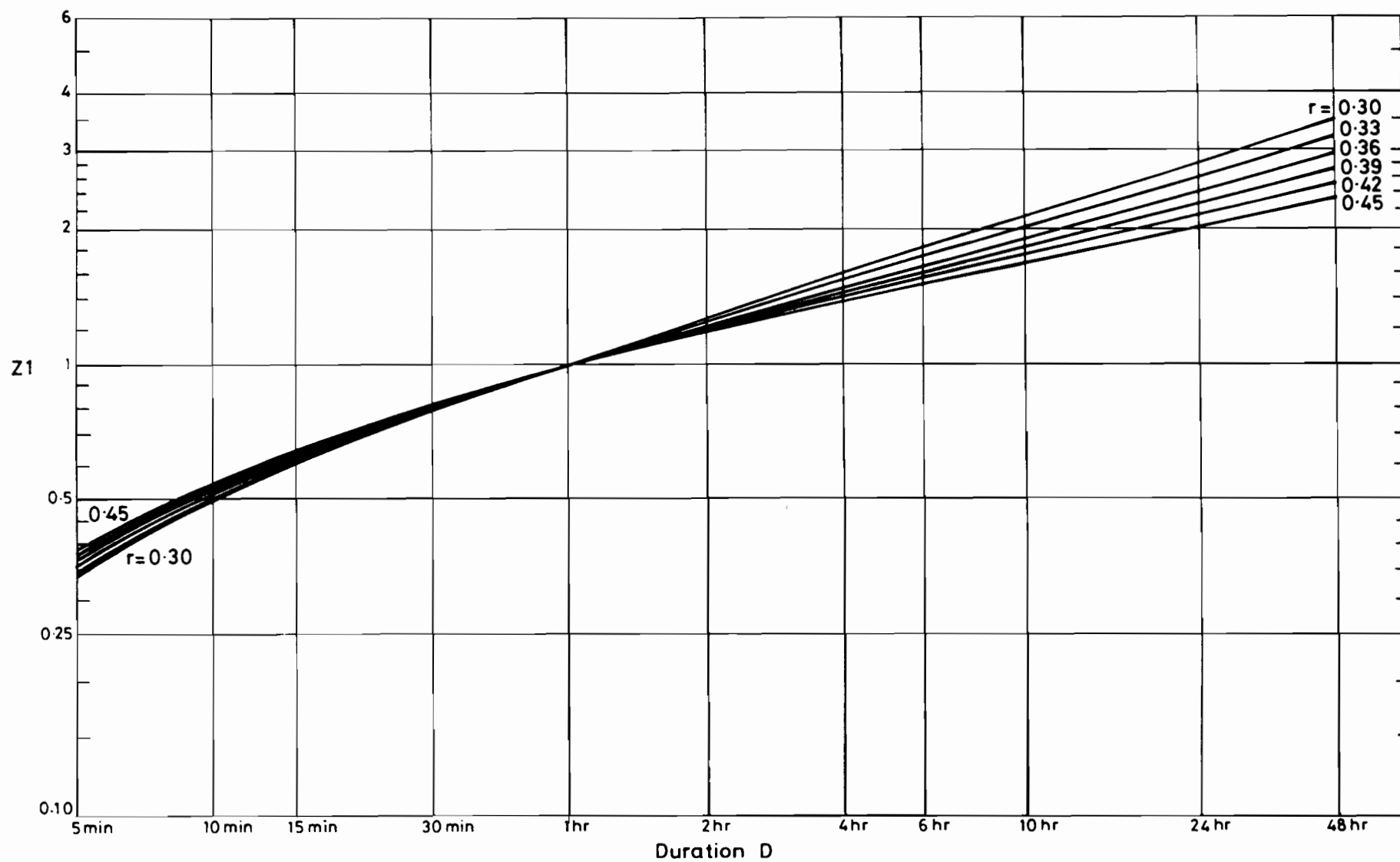


Ratio of sixty minute to two day rainfalls of five year return period (r)

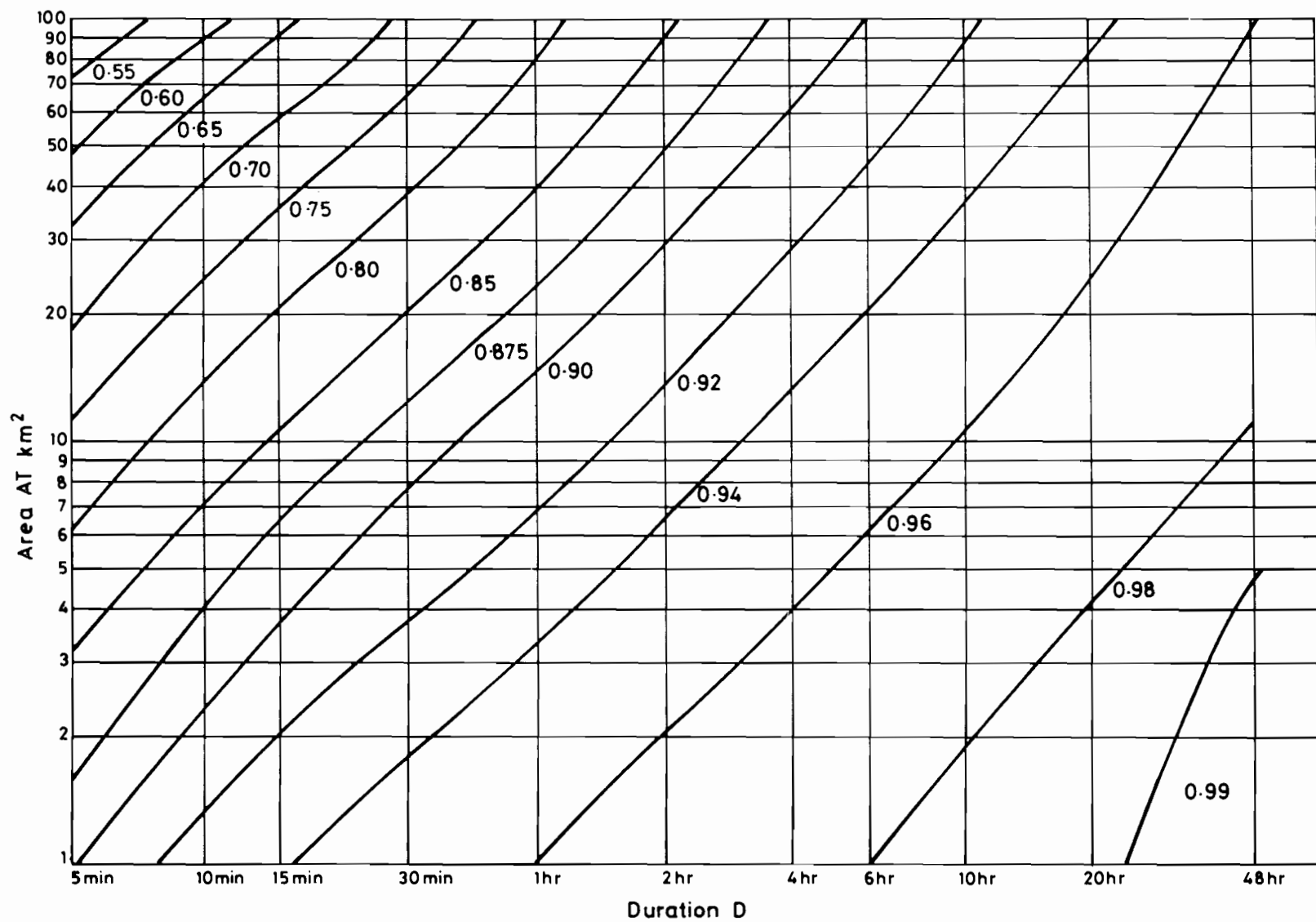
Fig. A.2



Relation between Z_1 and D for different values of r . ($0.12 \leq r \leq 0.30$)



Relation between Z_1 and D for different values of r . ($0.30 \leq r \leq 0.45$)



Areal reduction factor ARF related to Area A_T and Duration D

Appendix N

Southern Water Foul Modelling Criteria

Developer Services

Foul Sewerage Modelling Criteria:

Southern Water continues to review its modelling procedures and design standards. Our current update on the impact of a new development on the public sewer network is as below:

Item			
Development Size – Number of units	N		
Per Capita Flow -Litres/ head / day	G	125	(see note 4)
Infiltration – Percentage	I	10	(see note 5)
Occupancy – Persons/Dwelling	O	2.4	(see notes 6 & 7)
Dry Weather Flow multiplier (PF - Peaking Factor) (SD –Storm Duration– minutes)		SD	PF
		30 to 240	2.5
		240 to 480	2.0
		Above 480	1.4
Allowance for misconnected surface water		1.4 Square Metres per Dwelling (see note 8)	
Population – Number of people	P	N x O	

Hence: Design flow = (PF + 0.1) PG (foul flow element) plus the impact of 1.4 x N sq. m. (allowance for misconnected surface water)

Note that the above criteria applies subject to:

- 1) Only to the case of new domestic foul flow.
- 2) No proposed discharge of surface water into the foul sewer.
- 3) Southern Water supports the Hierarchy of H3 of Building Regulations with regards to the disposal of Surface Water.
- 4) Compliance with G2 of Building Regulations; that reasonable provision must be made by the installation of fittings and fixed appliances that use water efficiency for the prevention of undue consumption of water.
- 5) That upstream sewers are designed and constructed with materials and method fully compliant with Sewers for Adoption and Southern Water published addendum and corrigendum, in order to ensure that the infiltration of groundwater is minimised to the low rate of 10% of base flow.

- 6) That unless we are advised otherwise, we will assume the occupancy rate of 2.4 persons per property to be appropriate and in accordance with survey data that Southern Water has for development within its area.
- 7) Should the makeup of development be known and advised to Southern Water, with regards to the number of 1-bedroom, 2-bedroom units etc. then the modelling can be revised on the basis of:
- Number of bedrooms + 1, as the occupancy for each unit type.
 - This level of information may not be available for initial Capacity Check assessments and in this case the default figure of 2.4 persons/dwelling is considered appropriate.
- 8) Should the density of the development be known, (where development density is calculated based on the number of expected new addresses divided by the area of the proposed site development) and advised to Southern Water, then the allowance for misconnected surface water can be adjusted to the following:

Development Density (Properties / 4Ha)	Misconnected surface water allowance m ² /property
<=100	2.1
120	1.6
140	1.1
180	0.6
>=200	0.3

Flats, housing association, and commercial property addresses will have no misconnected surface water allowance applied.

- 9) With regards to the allowance for misconnected surface water, reference is made to published guidance or studies including LASOO, CIRIA, DEFRA, and UKWIR.
- 10) With regards to any proposed pumped flow rates.
- For the hydraulic design of pumping stations (and associated rising mains) we are guided by Sewers for Adoption and in the case of Edition 7, by clauses D4.6 and D5.3.1.
 - This has a range of velocity of discharge in the rising main between 0.75 to 1.8 m/s, when the pump is operating. Our preference would be for a higher velocity than the minimum.
 - If however the discharge is by gravity, then we would seek flows within the pipe capacity and self cleansing range.

Appendix O

Drainage Management and Maintenance Plan



Land Behind 147 – 149 College Lane,
Hurstpierpoint

Drainage Management & Maintenance Plan

For

Earlswood Homes

Document Control Sheet

Land Behind 147 – 149 College Lane, Hurstpierpoint
Earlswood Homes

This document has been issued and amended as follows:

Date	Issue	Prepared by	Approved by
10 th April 2024	FINAL	Phil Allen MCIWEM C.WEM	Neil Jaques

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1.0 Introduction

- 1.1 This document sets out the suggested principles for the long-term management and maintenance of the surface water drainage system on the proposed residential development on the land behind 147-149 College Lane, Hurstpierpoint, West Sussex.
- 1.2 The purpose of this document is to ensure that the adopting site management company has a robust inspection and maintenance plan in place for the lifetime of the development. This ensures the optimum operation of the surface water drainage system and that it will be maintained in perpetuity. This will contribute to reducing the risk of surface water flooding both on- and off-site.
- 1.3 All those responsible for maintenance should follow relevant health and safety legislation for all activities listed within this report (including lone working, if relevant). Method statements and risk assessments should always be undertaken and made available, if requested.
- 1.4 This document has been produced by Motion on behalf of their client, Earlswood Homes. This document describes the typical management and maintenance tasks that are known at the design stage (maintenance frequencies and typical tasks, for example). These have been drawn from industry guidance such as CIRIA C753 - The SuDS Manual – and manufacturer's own guidance.
- 1.5 Maintenance is considered as a construction activity under the CDM Regulations 2015. Under the CDM Regulations, it is a requirement that a competent person be appointed to carry out a required role. CDM defines a competent person as an individual with sufficient knowledge of the specific tasks to be undertaken, as well as sufficient experience and ability to carry out their duties in relation to the task in a way that secures health and safety on site.
- 1.6 In recognition of the requirements of the CDM Regulations 2015, this drainage management and maintenance plan expects that the maintenance work will be carried out by a competent person who must have prior knowledge of the drainage components and SuDS systems on site.
- 1.7 There are limitations on what this document can prescribe at the planning stage. At this stage this document cannot name the specific individuals who will carry out the maintenance and what equipment is to be used. Related to this, this document is unable to provide method statements for exactly how maintenance practices will be carried out. These can only be determined at the time of the maintenance being carried out and the exact maintenance need. Therefore, this is to be the responsibility of the adopting site management company and/or the individuals carrying out the work. We urge those who are carrying out the maintenance to record this information and make it available to the Local Planning Authority (LPA), if required to do so. This drainage management and maintenance plan needs to be a living document that is owned and maintained by the adopting site management company and should be adhered to for the lifetime of the development.

2.0 Maintenance Categories

2.1 There are three categories of maintenance activities referred to in this report. These are:

Inspection and Monitoring

- ▶ Inspection and monitoring tasks should be carried out frequently, nominally once a month, and should include a visual inspection of all components including all inlets and outlets.

Regular Maintenance (Monthly)

- ▶ Regular maintenance consists of basic tasks done on a frequent and predictable schedule, including vegetation management and litter removal.

Seasonal Maintenance (Quarterly)

- ▶ Seasonal maintenance comprises tasks that are likely to be required periodically, but on a much less frequent and predictable basis than the routine tasks (leaf litter and sediment removal is an example).

Remedial Maintenance

- ▶ Remedial maintenance comprises of intermittent tasks that may be required to rectify faults associated with the system that have been identified through visual inspections. The likelihood of faults can be minimised by correct installation, regular inspection and timely maintenance. Where remedial work is found to be necessary, it is likely to be due to site-specific characteristics or unforeseen events and, as such, timings are difficult to predict.

3.0 The Surface Water Drainage System

- 3.1 The proposed surface water drainage system is made up of a number of components/structures. These include:
- ▶ Permeable paving
 - ▶ Geocellular Attenuation Tanks
 - ▶ Catchpit manholes/silt traps
 - ▶ A Hydrobrake
 - ▶ Manholes
 - ▶ Pipes
 - ▶ Pumps
 - ▶ Water butts (although these will be in private ownership)
- 3.2 All components should be installed in accordance with the manufacturer's instructions and to the levels/arrangement as defined on the designer's drawings. Not doing so will invalidate any warranty provided by the manufacturer.
- 3.3 All maintenance and cleaning must be carried out in accordance with manufacturer's recommendations and by competent and suitably qualified staff, as defined in the CDM regulations 2015.
- 3.4 This document should be read in conjunction with the design drawings of the drainage system, so that the location and type of each feature can be recognised and understood.
- 3.5 Manufacturer's instructions are to be added to this document once specific products have been selected and installed as part of the detailed design. This document will subsequently form the basis for a drainage maintenance regime.

4.0 General Maintenance Principles

- 4.1 All surface water drainage systems, whether piped gravity systems, Sustainable Drainage Systems (SuDS), or flow control devices and pumps, require regular maintenance to keep them working at optimum efficiency and capacity. The maintenance of the surface water drainage system on the development on land behind 147 – 149 College Lane should be carried out alongside other regular maintenance tasks on site.
- 4.2 Timely and adequate maintenance will increase the lifespan of all the drainage components. Inadequate maintenance will do the reverse. Therefore, the projected lifespan and anticipated replacement date of each drainage component cannot be forecast at the time of this document being produced.
- 4.3 The site management company (or their agents) are responsible for the maintenance of the surface water drainage system for the lifetime of the development.
- 4.4 Construction activities can create and discharge significant quantities of sediment that will quickly clog the surface water drainage system. Therefore, construction-stage sediment removal is required immediately post-construction. The construction site manager should assess this and carry out cleaning as necessary.
- 4.5 Catchpit manholes/silt traps will be specified upstream of the permeable paved areas, as well as other locations on site. They will remove gross solids and the majority of silts. It is important that any debris build-up in the catchpit manholes/silt traps is removed at regular intervals. This will reduce the risk of the permeable paved areas becoming silted up. It will maintain the design capacity and function of this part of the drainage system.
- 4.6 Cleaning should also take place after large storms when there have been increased surface water flows and visible entrainment and deposition of debris.
- 4.7 An increased frequency of inspection and maintenance should be programmed into the autumn and winter months in acknowledgement that:
 - ▶ Leaf fall from deciduous trees in autumn will result in an increased amount of leaf litter and an elevated blockage risk of drainage infrastructure.
 - ▶ Increased rainfall during winter months will result in greater quantities of water moving through the drainage system and a greater input of silt and other debris.
- 4.8 Table 4.1, below, gives an overview of required maintenance tasks and the frequency with which they need to be undertaken. Section 5 – Inspection and Maintenance Frequency of Components – will assign typical maintenance frequencies and tasks to the specific components used within the surface water drainage system proposed for the development on the land behind 147 – 149 College Lane.

Table 4.1: Typical maintenance tasks and frequencies

Activity	Indicative Frequency	Typical Tasks
Inspection and Monitoring	Monthly	▶ Inspection of all inlets, outlets and control structures
Regular Maintenance	Monthly, for the lifetime of the development	▶ Litter picking and debris removal ▶ Weed removal and invasive plant control
Seasonal Maintenance	Quarterly, for the lifetime of the development	▶ Vegetation management around components ▶ Sweeping of pavement areas to remove surface silt ▶ Silt removal from system, including catchpits, cellular storage structures and control structures
Remedial maintenance	As required as a result of inspections, for the lifetime of the development.	▶ Inlet/outlet repairs ▶ Erosion repairs ▶ Reinstatement of edgings ▶ Reinstatement following pollution incidents ▶ Removal of silt build-up and leaf litter after storms ▶ Repair of vandalism ▶ Replacement of any blocked filter membranes/materials

5.0 Inspection and Maintenance Frequency of Components

- 5.1 Table 5.1 below lists each of the components used within the site's surface water drainage system. It suggests an indicative maintenance frequency for each component and ascribes typical maintenance tasks to them.
- 5.2 This list is not exhaustive, nor is it prescriptive. As mentioned in Section 3, additional, unscheduled maintenance may be required following adverse weather conditions or after autumn leaf falls. Additional maintenance tasks may be required to adequately clean and maintain individual components.
- 5.3 The list of components should be cross-referenced with the designer's drawings so that the location of each component can be identified.
- 5.4 It is the responsibility of the adopting site management company (or their agents) to ensure that all necessary maintenance activities are carried out in a timely manner and that the design performance of each drainage component is preserved.
- 5.5 If there is any uncertainty regarding the correct and safe methods of cleaning, or what equipment should be used, the manufacturer should be consulted.

Table 5.1: Maintenance Frequency and Task for Drainage Components

Activity	Indicative Frequency	Anticipated Tasks
Pipes	As required	▶ Identify any pipes that may not be operating properly and employ a competent, qualified contractor to inspect using CCTV. ▶ If the pipe is blocked with silt or debris, the pipe should be jetted clean from an upstream access point. All silt and debris should be captured and removed at a downstream access point. ▶ Inspect once clean. ▶ If any other defects are encountered (cracks, displaced joints, root ingress), appropriate solutions should be discussed with a competent and qualified contractor. These services are usually provided by the same companies that offer CCTV surveys and pipe jetting services.
Manholes	Annually and as required, for the lifetime of the development.	▶ Inspect/identify any damage or areas that are not operating correctly ▶ Remove silt, litter, leaves and other detritus. ▶ Inspect once clean.
Catchpit Manholes/Silt Traps	Annually and as required, for the lifetime of the development.	▶ Inspect/identify any damage or areas that are not operating correctly ▶ Remove silt, litter, leaves and other detritus. ▶ Inspect once clean.
Hydrobrake chambers	Every three months for the first year, then annually thereafter for the lifetime of the development.	▶ Contact manufacturer for instruction on approved and safe inspection and maintenance practices. ▶ Inspect Hydrobrake and check functionality. Remove any detritus as required. ▶ Inspect once clean.

Geocellular Attenuation Tanks	Annually	▶ Contact manufacturer for instruction on approved and safe inspection and maintenance practices. ▶ Inspect/identify any areas that are not operating correctly. ▶ Remove debris from catchment surface. ▶ Remove sediment from pre-treatment structures. ▶ Check for silt build-up and flush and remove as required (in accordance with manufacturer's instructions). ▶ Inspect once clean. ▶ See Table 21.3 of CIRIA C753 for more information. ▶ Most geocellular units have a 60-year creep limited life expectancy, so they should be planned for replacement by 2085 (approx.).
Water Butts (not the responsibility of the adopting site management agency, but individual homeowners)	Annually in Autumn to Winter	▶ Remove falling leaves and seeds from guttering or those that have found their way into the water butt. ▶ Water may stagnate slightly. If so, use a water butt cleaning disc into the tank. ▶ In autumn and winter, drain water off every 10 days (or less) to make sure that water butts don't overflow and that water is kept moving. This will stop larvae and flies from using the water butt. ▶ Use safe products such as vinegar to clean the outside of the tank and the inside of the lid and be careful not to contaminate water with chemicals. ▶ At least once a year, completely empty the water butt and scrub it out with warm soapy water and then rinse thoroughly. This is best done at a time when the water butt is already nearly empty (end of summer) or when it can readily refill (winter).
Pump	To be carried out by the manufacturer and their engineers. The site managers should enrol into a pump station maintenance contract (service agreement) with the pump manufacturer that provides 24/7 support.	▶ As per the standard service and maintenance schedule of the pump manufacturer (TBC). ▶ Timely and regular maintenance of the pump will increase the pump's lifespan and efficiency and will reduce the risk of failures and callouts.
Permeable paving	Once a year after autumn leaf fall, or reduced frequency as required, based on site-specific observations of clogging or manufacturer's recommendations.	▶ Agitate surface by means of mechanical sweeping or vacuuming to ensure no vegetation or moss is allowed to establish and grow in the joints. ▶ Mechanical sweeping of pavements and refilling of joints with the correct aggregate need only be carried out at intervals of 5 years or so

		▶ Remove weeds from the surface through the application of glyphosate-based weed killers ▶ Stabilise and mow contributing and adjacent areas. ▶ Inspect once clean. ▶ See Table 20.15 of CIRIA C753 for more information. ▶ Permeable paving has a nominal 25-year lifespan, if correctly and regularly maintained. ▶ When subjected to low level oil drips permeable pavements can continue to biodegrade hydrocarbons indefinitely. ▶ Major oil spills have the potential to contaminate the surface and the underlying crushed stone. In the event of a major oil spill, the area of block pavements and crushed stone that is affected should be removed, cleaned and reinstalled.
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5.6 Upon completion of maintenance activities, a record should be kept of the work carried out. This should be retained and an annual maintenance report should be compiled, which should include the following:

- ▶ Observations resulting from inspections
- ▶ Maintenance and operation activities undertaken during the year
- ▶ Recommendations for inspections and maintenance programmes for the following year

5.7 On the next page is a table with suggested information should be recorded and included with the maintenance plan. As mentioned in the introduction to this document, this should be a living document and regularly updated, as required and should be kept for the lifetime of the development.

5.8 The Local Planning Authority (Mid-Sussex District Council) may request to check and sign off any maintenance activities. Therefore, it is the recommendation that the LPA is contacted prior to any scheduled routine maintenance. The table mentioned above and on the next page, as well as the annual maintenance report, should be offered to the LPA for their records and approval.

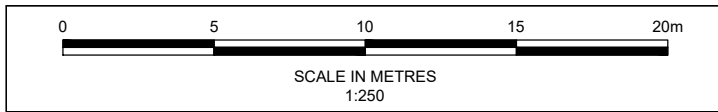
Date	Component requiring maintenance	Issues prompting maintenance	Scheduled maintenance (Y/N)	Maintenance carried out	Additional works required (Y/N). If yes, please detail	Next scheduled date of inspection and maintenance

Appendix P

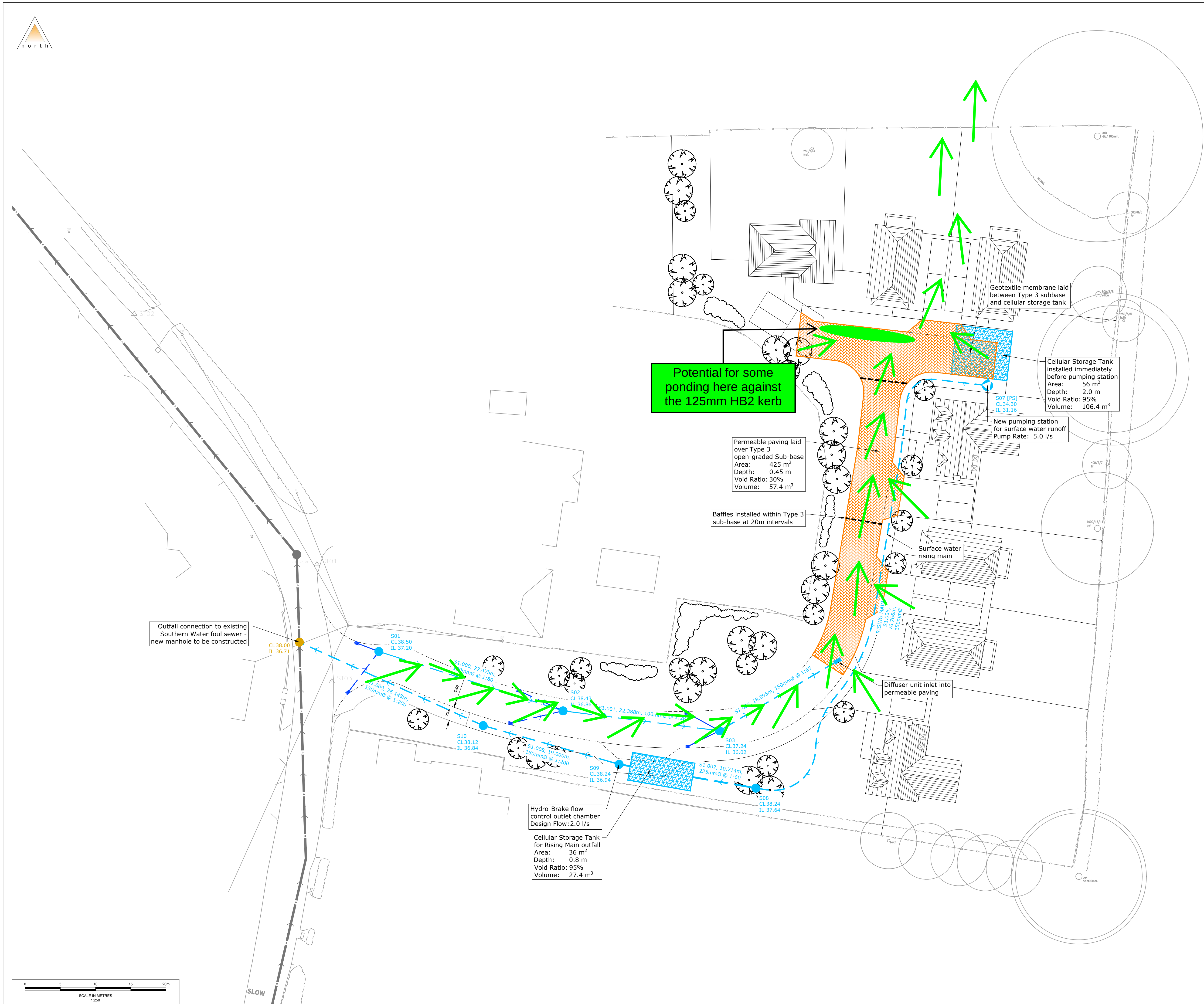
Exceedance Plan



C:\Users\matt\OneDrive\Documents\Drawings\2403069-0501 [Drainage Strategy].dwg



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Notes

1. All levels and dimensions to be checked on site before any work commences. All dimensions in metres unless stated otherwise.
2. This drawing is based on survey information supplied by [add], and Motion cannot guarantee the accuracy of the data provided.
3. This drawing is based on OS mapping and Motion cannot guarantee the accuracy of the data.
4. Highway boundary information obtained from [add] and based on OS mapping, Motion cannot guarantee the accuracy of the data provided.
5. Motion accepts no liability for any vehicle specification errors or inaccuracies within the vehicle tracking software used / or it's vehicle libraries. The vehicles speeds used for the analysis are as follows: forward **mph / reversing **mph.

Legend

- Existing Southern Water Foul Sewer
- Proposed Surface Water Drain
- Proposed Surface Water Gully and Pipe Connection
- Proposed Surface Water Rising Main
- Proposed Surface Water Manhole
- Proposed Surface Water Pumping Station
- Proposed Diffuser Unit
- Proposed Permeable Paving
- Proposed Permeable Paving Baffle
- Proposed Cellular Storage Tank
- Proposed Foul Manhole - Outfall Connection to existing sewer network

P0	First Issue	MH	MH	PA	12/04/2024
Rev.	Description	Drm	Chk	App	Date

Drawing Status:

FOR PLANNING
NOT FOR CONSTRUCTION

motion
Guldford - Reading - London
www.motion.co.uk

Client:
Earlswood Homes Ltd

Project:
Land off College Land
Hurstpierpoint

Title:
Proposed Drainage Strategy
Surface Water Drainage

Scale: 1:250 (@ A1)

Drawing:
2403069-0501

Revision:
P0

Appendix Q

Mid-Sussex Drainage Checklist

Flood Risk and Drainage Information Check List – Application Stage

This checklist is designed to support applicants within the planning process provide the information required by the flood risk and drainage team for them to provide a consultation response. This is not the complete planning validation list, however much of the information is also included with the validation list.

The level of detail provided to address each information point should reflect the scale and complexity of the development.

Receipt of this information does not guarantee the flood risk and drainage team will support an application or does it prevent a request for further information. It does, however, ensure the team has sufficient information to undertake a full review and provide detailed consultation response.

Table 1 - Application details

Application number	TBC
Site address	Land behind 147 – 149 College Lane, Hurstpierpoint, BN6 9AF
Site co-ordinates	TQ 29238 16940
Site area in hectares	0.621
Existing site usage	Greenfield – Paddocks and Grazing
Proposed development description	Seven residential units plus access and parking
Block plan	See Appendix A of FRA and Drainage Strategy
Existing site plan	See Appendix B (location plan) of FRA and Drainage Strategy
Proposed development plan	See Appendix A of FRA and Drainage Strategy

Table 2 - Flood Risk

Flood zone(s)	1
Surface water flood risk(s)	Low – Very Low
Flood risk assessment provided	Yes

The Environment Agency and the Department for Environment, Food and Rural Affairs (DEFRA) states that a Flood Risk Assessment is required for all development:

- within flood zones 2, 3 or 3b
- within flood zone 1 with a site area of 1 hectare or more
- within areas with critical drainage problems
- within flood zone 1 where your LPA's SFRA shows it will be at risk of flooding from rivers or the sea in future
- that increases the vulnerability classification and is in flood zone 1 where your LPA's SFRA shows it is at risk from other sources of flooding

Mid Sussex District Council's Strategic Flood Risk Assessment (SFRA) utilises the latest flood maps from the Environment Agency to determine flood risk.

Surface Water Drainage

At application stage the applicant must show that surface water drainage can, in principle be achieved on the site. The minimum information required is shown in Table 3.

If an applicant wishes to avoid a drainage condition, then full detailed drainage design is required at application stage. Details of the information needed for a detailed drainage design are set out in the Discharge of Conditions checklist.

Table 3 - Surface water drainage - minimum information

Proposed means of drainage	Discharge to foul sewer in College Lane
Infiltration potential on site	Unviable – Thick Weald Clay to depth
Drainage constraints plan (showing any areas drainage cannot be located such as root protection zones, main sewer buffers etc)	Drainage constraints discussed in Section 2 of FRA and Drainage Strategy. RPZ's shown on Proposed Site Layout in Appendix A.
Acknowledgement of design criteria (sized to cater for 1 in 100-year plus climate change storm, any off-site discharge restricted to the Greenfield QBar rate for all events up to and including the 1 in 100-year plus climate change storm)	Drainage tested through drainage hierarchy. Drainage modelled to 1 in 100-year + 45% in MicroDrainage Network Module. Greenfield QBAR runoff rate modelled (1.32 l/s) and site discharge rate of 2 l/s compare favourably. Please see FRA and drainage strategy for full discussion.
Water Authority's agreement in principle to connect surface water to public sewer. (if applicable)	Pre-planning enquiry to Southern Water submitted – response pending
Drainage strategy plan	Appendix K of FRA and Drainage Strategy

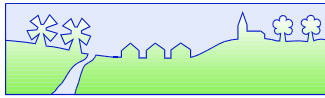
Foul Water Drainage

At application stage the applicant must show that foul water drainage can, in principle be achieved on the site.

If an applicant wishes to avoid a drainage condition, then full detailed drainage design is required at application stage. Details of the information needed for a detailed drainage design are set out in the Discharge of Conditions checklist.

Table 4 - Foul water drainage - minimum information

Proposed means of drainage	Pump from the lowest parts of the site to upper site and then gravity outfall to 300mm foul sewer in College Lane.
Public sewer records plan	Appendix F of the FRA and Drainage Strategy
Drainage constraints plan (showing any areas drainage cannot be located such as root protection zones, main sewer buffers etc)	As above



MID SUSSEX DISTRICT COUNCIL

Flood Risk and Drainage

Drainage strategy plan	As above
Outline on how non-mains drainage will meet Environment Agency's General Binding Rules for non-mains drainage. (if applicable)	N/A