**Project:** Flood Risk Assessment (FRA) and Surface Water Drainage Strategy (SWDS)

**Prepared for:** UK & European Property Developments Ltd

**Reference:** 3193 SWDS

**Date:** February 2019

**Version:** Final v2

# Document Issue Record

**Project:** Surface Water Drainage Strategy (SWDS)

**Prepared for:** UK & European Property Developments Ltd

**Reference:** 3193 SWDS

**Site Location:** Garages Adjacent to Nos. 72 - 75 Sontan Court, Churchview Road, Twickenham, Richmond, TW2 5BU

**Proposed Development:** It is understood that the development is for the demolition of an existing garage block and the erection of 2 mews properties.

| Consultant | | Date | Signature |
|---|---|---|---|
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| Amendments (3 dwellings -> 2 dwellings) | Bojidar Boiadjiev | 18/02/2019 | |

## Please Note:



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# Contents

# 1. Summary

| DEVELOPMENT DESCRIPTION | | |
|---|---|---|
| | EXISTING | PROPOSED |
| Development Type: | Developed | 2 mews properties |
| (Number of Bedrooms): | n/a | 2x2 |
| EA Vulnerability Classification: | Less Vulnerable | More Vulnerable |
| Ground Floor Level: | n/a | 11.39 or above |
| Level of Sleeping Accommodation: | n/a | 11.39 or above |
| RISK TO DEVELOPMENT | | |
| | SUMMARY | COMMENT |
| EA Flood Zone: | 1 | Small part of the proposed grass lawn lies in Flood Zone 3 |
| Flood Source: | Fluvial | River Crane |
| Flood level: | 11.09mAOD | Detailed modelling data provided by the EA. 70% climate change |
| Recorded Flood Events in Area: | NO | |
| Recorded Flood Events at Site: | NO | |
| SFRA Available: | Yes | The London Borough of Richmond upon Thames SFRA |
| MANAGEMENT MEASURES | | |
| | SUMMARY | COMMENT |
| Ground floor level above extreme flood levels: | YES | |
| Safe Access/Egress Route: | YES | Warning & Evacuation Plan |
| Flood Resilient Design: | YES | |
| Site Drainage Plan: | YES | |
| Flood Warning & Evacuation Plan: | YES | EA Flood Warning Service and EA Flood Alert |
| OFFSISTE IMPACTS | | |
| | SUMMARY | COMMENT |
| Displacement of floodwater: | NO | Buildings located outside extreme flood event |
| Increase in surface runoff generation: | YES | Addressed in drainage strategy |
| Impact on hydraulic performance of channels: | None | Does not affect channel |

| SITE DETAILS | | | |
|---|---|---|---|
| **Site Name** | Garages Adjacent to Nos. 72 - 75 Sontan Court, Churchview Road, Twickenham, Richmond, TW2 5BU | | |
| **Total Site Area (relevant for drainage)** | 0.0548 ha | | |
| **Site Area which is positively drained** | 0.0548 ha | | |
| **Significant Public Open Space** | 0.0000 ha | | |
| **Predevelopment Use** | Garages | | |
| Site Constraints | - Residential Site | | |
| | - Groundwater Source Protection Zone: | NO | |
| | - Groundwater Vulnerability Zone: | Major Aquifer High | |
| | - Poor Infiltration Soils | | |
| | - Unknown Groundwater Table | | |
| IMPERMEABLE/HARDSTANDING AREA | | | |
| | EXISTING | PROPOSED | DIFFERENCE (Proposed - Existing) |
| Impermeable Area (Ha) | 0.0123 ha | 0.0272 ha | 0.0149 ha |
| Drainage Method (Infiltration/Sewer/Watercourse) | | Sewer + Infiltration | N/A |
| PROPOSED TO DISCHARGE SURFACE WATER VIA | | | |
| | YES | NO | EVIDENCE |
| Infiltration | X | | |
| To Watercourse | | X | The land between the river and the site is not owned by the developer |
| To Surface water sewer | X | | |
| Combination of above | | X | |
| PEAK DISCHARGE RATES | | | |
| | Greenfield Rates (l/s) | Pre-development Rates (l/s) | Proposed Rates (l/s) |
| Greenfield QBAR | 0.1 l/s | N/A | N/A |
| 1 in 1 | 0.1 l/s | 1.7 l/s | 1.0 l/s |
| 1 in 30 | 0.2 l/s | 4.2 l/s | 1.0 l/s |
| 1 in 100 | 0.3 l/s | 5.4 l/s | 1.0 l/s |
| 1 in 100 plus climate change | N/A | N/A | 1.0 l/s |

| SITE STORAGE VOLUME | | | |
|---|---|---|---|
| Source Control Provided | Yes | | |
| Interception Volume (Capture and retention on site of the first 5 litres of the majority of all rainfall events) | 1.4 $m^3$ | | |
| Attenuation Volume (Storage - 1 in 100 year + CC) Volume to control discharge rate | 17.6 $m^3$ | | |
| Long Term Storage (1 in 100 years, 6 hours event) Difference in runoff volume between the development state and the equivalent greenfield (or predevelopment state) | Not taken into account. | | |
| Approach used for storage | Either:<br>• Approach A: Use Long Term Storage<br>• Approach B: All runoff above 1 in 1 year return period discharged to greenfield runoff rate | Approach B<br>Flow Control to 1.0 l/s | |
| Total site Storage Provided | **17.6 $m^3$** | | |

| INFILTRATION FEASIBILITY ANALYSIS | | |
|---|---|---|
| Site's Geology | Tapflow Gravel Formation | |
| Infiltration Rates | 0.36m/hr | This value was conservatively assumed for the existing soil. It should be confirmed through trial pit infiltration tests on site prior to the final detailed drainage design stage being carried out. |
| Infiltration Rates Suitability | **Suitable for nominal infiltration** | |
| Ground Water Level | Unknown | It is recommended that a groundwater level check be undertaken at the later detailed design stage in order to accurately identify the depth of the water table at the site. |
| Is the site within a known Source Protection Zones (SPZ)? Yes/No? | NO | |
| Is Infiltration feasible? | YES | |
| Site's Contamination | Unknown | |
| Storage Requirements Approach? | Simple Approach. Discharge Attenuation Volume at greenfield runoff rate. | |

# 2. Development Description and Site Area

2.1 This Flood Risk Assessment and Surface Water Drainage Strategy has been prepared by Ambiental Technical Solutions, in respect of a planning application for the development at Garages Adjacent to Nos. 72 - 75 Sontan Court, Churchview Road, Twickenham, Richmond, TW2 5BU, coordinates: X = 514612; Y = 173098.

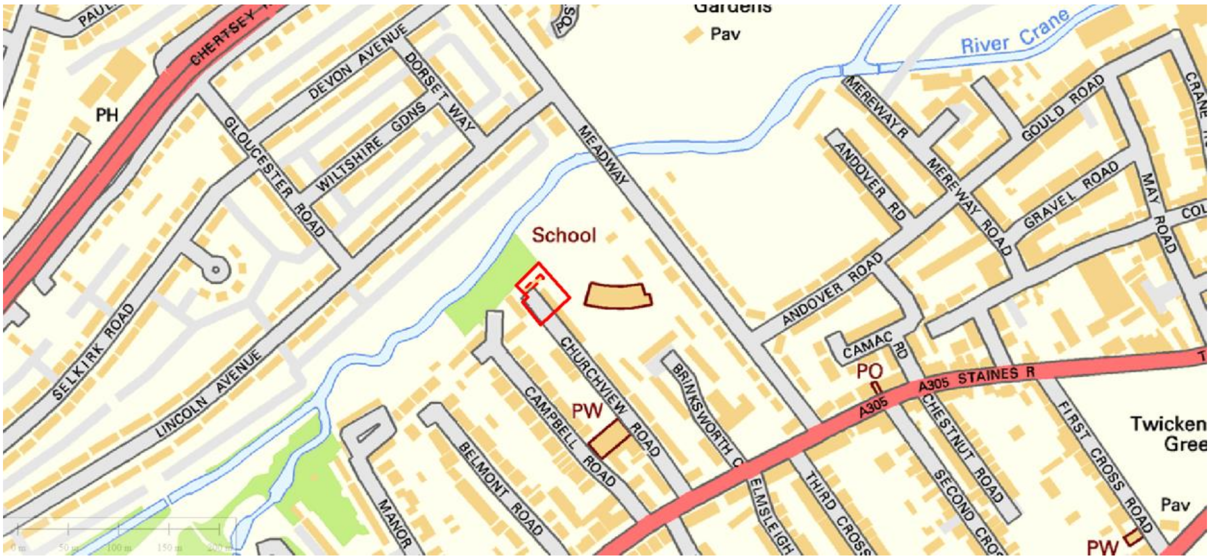


*Figure 1 - Site Location. Proposed development area outlined in red, red dash shows proposed building location.*

## Development Proposal

2.2 It is understood that the development is for the demolition of an existing garage block and the erection of 2 mews properties.

2.3 This study is based on plans included in Appendix A.

## Need for Study

2.4 The purpose of this assessment is to demonstrate that the development proposal outlined above can be satisfactorily accommodated without worsening flood risk for the area and without placing the development itself at risk of flooding, as per National guidance provided within the National Planning Policy Framework (NPPF).

## Site Area

2.5 The site is located at the northern end of Churchview Road. The River Crane is located to the north of the site. The distance from the proposed building to the river is approximately 40m.


*Figure 2 - Aerial View of Development Site. Proposed development area outlined in red. Shaded area indicates area relevant for drainage strategy.*

2.6 It is understood that the development is for the demolition of an existing garage block and the erection of 2 mews properties, within the footprint of the existing garages. The verge to the west of the site would be re-landscaped to provide x4 new carparking spaces. Two additional car spaces would be provided west of the proposed mews. The existing building and road will be retained. Reconfiguration of the existing on-site parking will be completed to provide thirteen spaces to the rear with three new parking spaces along the grass verge.

2.7 Since the existing dwelling building and access road will be retained, the drainage strategy will only address the proposed building with the garden and the proposed additional car park spaces.

2.8 The total area of the site that is relevant for the drainage strategy is approximately 548m$^2$ (0.0548 Ha), based on plans provided by the client. The majority of the site is considered pervious except for the existing garages. Following development, the impervious/hardstanding areas will be increased from 123m$^2$ to 272m$^2$ (approximately 0.0272 Ha).

2.9 The topography of this site ranges from approximately 11.89 to 13.82 mAOD[1] (Source: a topographic survey provided by the client and conducted by *MK Surveys*). The site is generally

[1] mAOD: Meters Above Ordnance Datum

sloping north to the River Crane. See Appendix A, Figure 2 – Existing Topographical Map 1of2 (Source: MK Surveys).

## Vulnerability classification

2.10 The proposed development is for the construction of dwelling houses and as such is classified as "More Vulnerable" under the NPPF.

2.11 The development site is mostly located within Flood Zone 1 as defined by the EA online Flood Map for Planning (see Figure 3). A small part of the site is located within Flood Zone 2 and Flood Zone 3. The extents of the proposed building location are within Flood Zone 1.

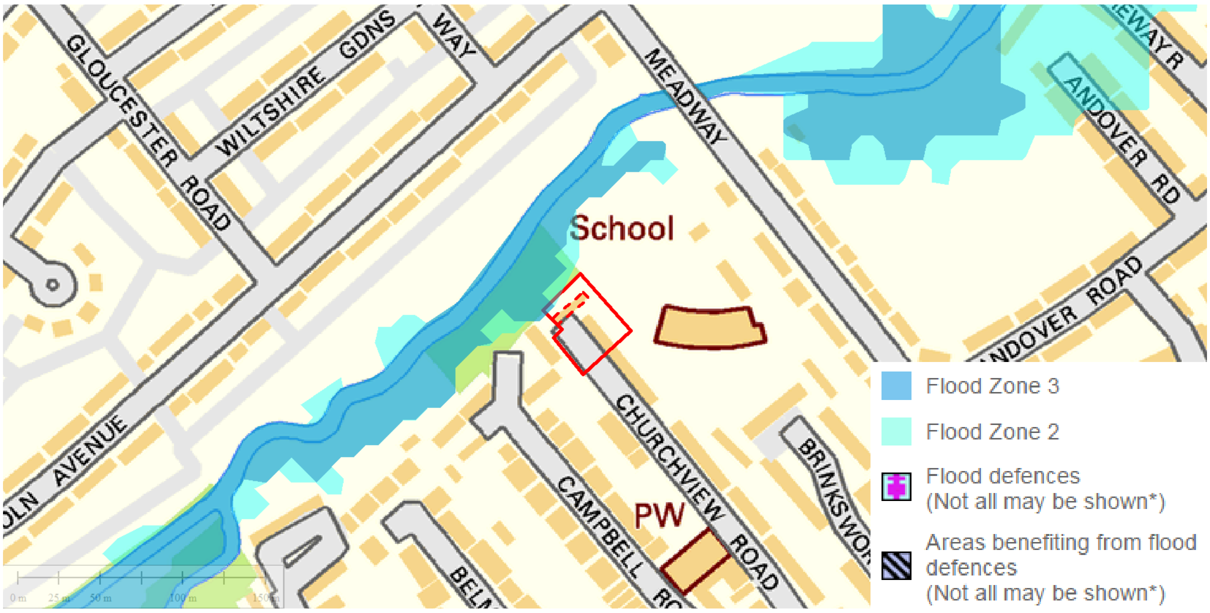


*Figure 3 – EA Online Flood Map. Proposed development area outlined in red, red dash shows proposed building location. (Source: EA)*

## Geology and Ground Conditions

2.12 The British Geological Survey (BGS) Map indicates that the bedrock underlying the site is the London Clay Formation – clay and silt. A sedimentary bedrock formed approximately 34 to 56 million years ago in the Palaeogene Period where the local environment was previously dominated by deep seas (see Appendix B, Figure 1 – Bedrock Geology).

2.13 The BGS Database indicates superficial deposits of Taplow Gravel Formation – sand and gravel (see Appendix B, Figure 2 - Superficial Deposits).

2.14 The Soil Parental Material in the most of the site was taken from the UK Soil Observatory (UKSO) website and it is classified as River Terrace sand/gravel, while the soil texture is sand to sandy loam (see Appendix B, Figure 3 - Soil Parental Material).

2.15 There are no boreholes in instant proximity to the site in the BGS database. The closest borehole is located some 400m north of the site. Refer to the Appendix B, Figure 5 - Boreholes Map.

2.16 The closest accessible borehole is TQ17SW68, which shows that the ground is composed of a wide range of soils varying from clay to flint gravel. The site is only 40m away from the river while

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this borehole is situated much further from the river and as such cannot be considered a reliable source of information.

2.17 Standard values from the specialized literature CIRIA 753 'The SUDS Manual' suggest the infiltration coefficient of sandy loam soils is ranged between 0.36 m/h ($1x10^{-7}$ m/s) and 36 m/h ($3x10^{-5}$ m/s), while it is more than 1080 m/h ($3x10^{-4}$ m/s) for gravel. Infiltration testing at the site has not been provided by the client, thus it is recommended that these values are checked through trial pit infiltration tests on site prior to the final detailed drainage design being carried out as well as a groundwater level check be undertaken in order to accurately identify the depth of the water table.

2.18 The site lies within a Major Aquifer High Groundwater Vulnerability Zone. The site does not lie within a Groundwater Source Protection Zone (see Appendix B, Figure 8 and Appendix B, Figure 9).

2.19 Given that the soil on site is presumably a "good infiltration media" (as defined by CIRIA 753 'The SUDS Manual') and that the site does not lie within a groundwater source protection zone, **nominal infiltration is deemed suitable. Since no infiltration testing has been provided, a very conservative infiltration coefficient of 0.36m/h has been assumed.**

## Nearby Watercourses and Drainage

2.20 The River Crane flows some 40m north of the site. It flows to the north-east and discharges in the Thames.

2.21 The land between the site and the River Crane is not owned by the developer. As such, it is not possible to discharge the surface water runoff directly into the river.

## Existing Drainage Infrastructure

2.22 The site is previously developed and as such there is assumed to be an existing drainage network.

# 3. Sequential Test/Exception Test

3.1 Under the NPPF, all new planning applications must undergo a *Sequential Test*. This test must be implemented by local planning authorities with a view to locating particularly vulnerable new developments (e.g. residential, hospitals, mobile homes etc.) outside of the floodplain.

3.2 The test refers to the EA Flood Zones described in Table 3. For reference, the NPPF *Sequential Test: Flood Risk Vulnerability and Flood Zone "Compatibility" Table* is reproduced below:

| Flood Risk Vulnerability Classification | | Essential Infrastructure | Water Compatible | Highly Vulnerable | More Vulnerable | Less Vulnerable |
|---|---|---|---|---|---|---|
| Flood Zone | Zone 1 | ✓ | ✓ | ✓ | ✓ | ✓ |
| | Zone 2 | ✓ | ✓ | Exception Test Required | ✓ | ✓ |
| | Zone 3a | Exception Test Required | ✓ | × | Exception Test Required | ✓ |
| | Zone 3b *Functional Floodplain* | Exception Test Required | ✓ | × | × | × |

*Table 1: The Sequential Test: Flood Risk Vulnerability and Flood Zone "Compatibility" Table as specified by NPPF. Shaded cells denote the proposed re-development. Please note: ✓ means development is appropriate; × means the development should not be permitted.*

3.3 Using the principles of the Sequential Test outlined above, the proposed development is "More Vulnerable". A very small part of the site is partly located within Flood Zone 3a (as defined by the EA online Flood Map for Planning). This small patch affects only the proposed grass lawn and is thus considered to pose low risk to the development. The dwelling development is fully within Flood Zone 1. See Figure 3.

3.4 As such the proposed development is deemed appropriate for this level of flood risk. Given the extents of the site lie partially within Flood Zone 3, the proximity of the site to the river, and the drainage challenges that might occur on site, the application submitted must be accompanied by an FRA which shows that the development can be achieved in a sustainable manner, with an overall reduction of flood risk to the site and surrounding area.

# 4. Site Flood Hazards

## Sources of Flooding

4.1 As outlined in Figure 3, the dwelling development lies within Flood Zone 1. While the overall site area contains cone elements within Flood Zone 3 (High Risk of flooding) the development area has been located in Flood Zone 1 (Low Risk of flooding). Residential development is considered to be "More Vulnerable" under the NPPF.

4.2 Communication with the EA has identified the following potential sources of flooding to the site:

| Source | Description |
|---|---|
| **Fluvial** | River Crane |
| **Surface Water** | On site |
| **Groundwater** | On site |
| **Sewer** | N/A |

*Table 2: Summary of flood sources.*

## Mechanisms of Flooding

4.3 The main mechanism of flooding on site is considered to be of fluvial nature.

## River (Fluvial)

4.4 According to the data provided by the EA, the probability of fluvial flooding across the development is less than 0.1% annually (less than 1:1000).

4.5 The nearest watercourse to the site is the River Crane, flowing north of the site.

4.6 Detailed modelling available for the site and provided by the EA demonstrates flood levels on site to be 11.02mAOD for the 1 in 100 year event +20% climate change. This value was taken from the closest upstream node to the site (thus adopting the most conservative approach).

4.7 Due to the more recent regulations concerning climate change, the flow in the river was linearly extrapolated to take account of 70% increase of rainfall as a result of climate change. The data from the EA was then used to obtain a relationship between the flow in the river and the flood level. A best-fit interpolation was then used to derive the flood level for the required flow. **This resulted in a flood level of 11.09mAOD.**

4.8 The lowest topographical point on site according to the survey provided by the client and conducted by MK Surveys is 11.89mAOD. The developer has agreed to locate the finished floor levels higher than 11.39mAOD (11.09m + 0.3m freeboard).

4.9 As such, the risk to the site from this source is deemed to be **relatively low**.

## Surface Water (Pluvial)

4.10 The EA online Risk of Flooding from Surface Water Map shows the majority of the site to be within a "Very Low" risk of flooding from surface water area, with a less than 0.1% chance of flooding from this source annually. Churchview Road lies partially within "Low" risk of flooding with an annual probability of flooding between 0.1% and 1% (Figure 4).

4.11 It has been mentioned that local residents report local pooling occurring at site. This anecdotal pooling should be mitigated against by providing a route for runoff in the event of an overflow.

4.12 As such, and given that drainage strategy will be addressed in more details later in this report, the risk to the site from pluvial flooding is considered **relatively low**.

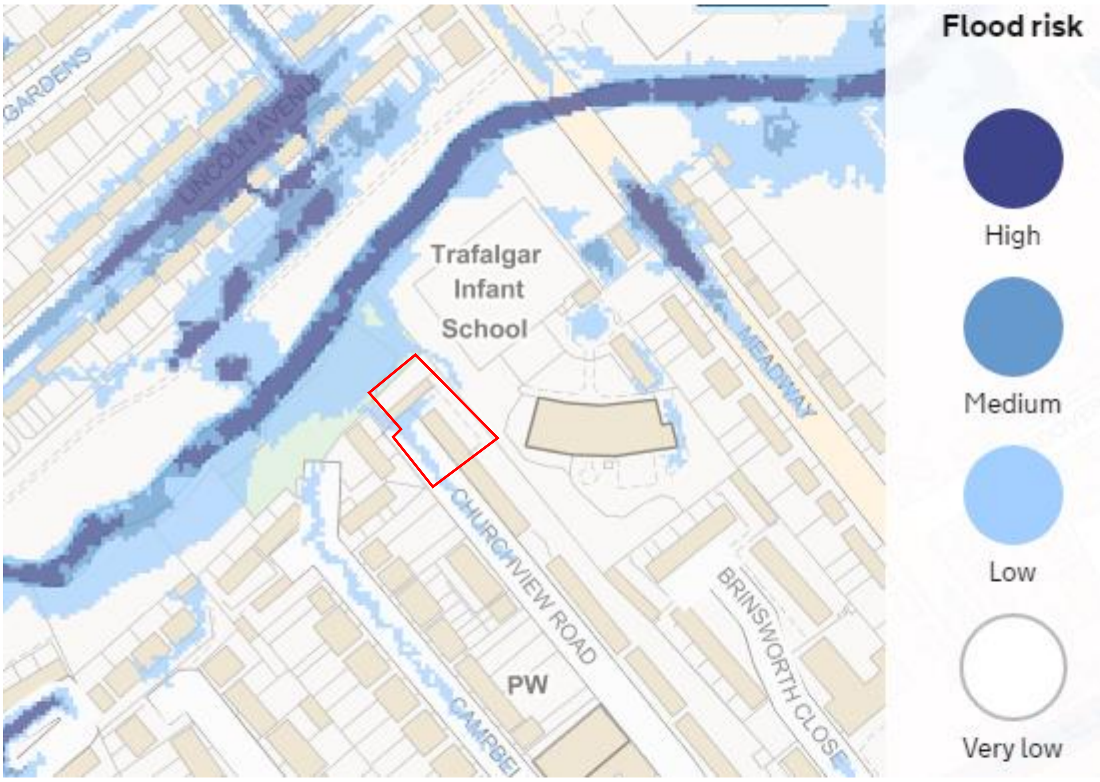



*Figure 4: EA Surface Water Flood Risk Map. Proposed development area outlined in red (Source: EA online)*

## Groundwater

4.13 The London Borough of Richmond upon Thames SFRA gives a map of regions susceptible to groundwater flooding. The site is located at the boundary between areas with "potential for groundwater flooding of property situated below ground level" and areas with "potential for groundwater flooding to occur at surface".

4.14 The SFRA also gives a map of incidents of groundwater flooding. None of the recorded incidents occurred in proximity to the site with the closest recorded some 1.8 km south east of the site.

4.15 The overall position of the SFRA is that "a *large proportion of the London Borough of Richmond upon Thames overlays London Clay and consequently the risk of groundwater flooding will typically be low*".

4.16 As such, and given that the proposed development does not include a basement, the risk of flooding from groundwater sources is expected to be **relatively low**.

## Sewer

4.17 The London Borough of Richmond upon Thames SFRA gives details of historic sewer flooding in the area. There is no specific data for the TW2 5 area. The closest area is TW2 6 in which the number of recorded incidents is between 1 and 5.

4.18 As such, given that communication will be sought with the local water companies regarding sewer outputs post development, the risk of sewer flooding to the site post development is deemed to be **relatively low**.

### Surface Water Drainage Strategy

4.19 Under the NPPF, following development, surface water runoff rates should be equivalent to (or below) the existing site run-off rate for all events up to the 1 in 100 year storm event, with an allowance for climate change.

4.20 A surface water drainage strategy is detailed later in this report.

### Records of Historical Flooding

4.21 The London Borough of Richmond upon Thames SFRA cites the EA Historic Flood Map, which indicates that the area around the site has not been affected by a historic event.

## 5. Probability of Flooding

5.1 According to the low detail, national-scale flood mapping created on behalf of the EA the probability of tidal flooding at the site is <0.1% (or less than 1 in 1000 year annual probability of fluvial flooding).

5.2 This information is supported by the EA Flood Map for Planning (Figure 3) which has been produced in part using JFLOW/HYDRO-F – a relatively coarse, national scale flood modelling strategy and in part through detailed modelling. It is important to note that only the *potential* floodplain is shown; ***the mitigating effects of any flood defences currently in place are not considered***. For reference, the definition of the NPPF flood risk zones is included below in Table 3.

| Zone | Description |
|---|---|
| 1 | **Low Probability**. This zone comprises land assessed as having a less than 1 in 1000 annual probability of river or sea flooding in any year (<0.1%). |
| 2 | **Medium Probability**. This zone comprises land assessed as having between a 1 in 100 and 1 in 1000 annual probability of river flooding (1% – 0.1%) or between a 1 in 200 and 1 in 1000 annual probability of sea flooding (0.5% – 0.1%) in any year. |
| 3a | **High Probability**. This zone comprises land assessed as having a 1 in 100 or greater annual probability of river flooding (>1%) or a 1 in 200 or greater annual probability of flooding from the sea (>0.5%) in any year. |
| 3b | **The Functional Floodplain**. This zone comprises land where water has to flow or be stored in times of flood. SFRA's should identify this Flood Zone (land which would flood with an annual probability of 1 in 20 (5%) or greater in any year or is designed to flood in an extreme (0.1%) flood, or at another probability to be agreed between the LPA and the EA, including water conveyance routes). |

*Table 3: Definition of the NPPF Flood Zones (Source: EA)*

## Climate Change on Site

5.3 Climate change is likely to increase the flow in rivers, raise sea levels and increase storm intensity. The range of allowances in Table 4 is based on percentiles. A percentile is a measure used in statistics to describe the proportion of possible scenarios that fall below an allowance level. The 50th percentile is the point at which half of the possible scenarios for peak flows fall below it and half fall above it.

5.4 The:

- central allowance is based on the 50th percentile
- higher central is based on the 70th percentile
- upper end is based on the 90th percentile

5.5 So, if the central allowance is 30%, scientific evidence suggests that it is just as likely that the increase in peak river flow will be more than 30% as less than 30%.

5.6 At the higher central allowance 70% of the possible scenarios fall below this value. So, if the higher allowance is 40%, then current scientific evidence suggests that there is a 70% chance that peak flows will increase by less than this value, but there remains a 30% chance that peak flows will increase by more (Source: EA).

5.7 The risk of flooding to the site would therefore be expected to increase following the effects of climate change. The likely increases in peak rainfall intensity would also lead to an increased risk of surface water flooding. The increase in river flows for the Thames Basin District have been provided below in Table 5.

| Flood Zone | Essential Infrastructure | Highly vulnerable | More Vulnerable | Less Vulnerable | Water Compatible |
|---|---|---|---|---|---|
| 2 | Higher Central and Upper End | Higher Central and Upper End | Central and Higher Central | Central | None of the allowances |
| 3a | Upper End | Development should not be permitted | Higher Central and Upper End | Central and Higher | Central |
| 3b | Upper End | Development should not be permitted | Development should not be permitted | Development should not be permitted | Central |

*Table 4: Allowance and Flood Zone Table (Source EA)*

## Table 1 peak river flow allowances by river basin district (use 1961 to 1990 baseline)

| River basin district | Allowance category | Total potential change anticipated for the '2020s' (2015 to 2039) | Total potential change anticipated for the '2050s' (2040 to 2069) | Total potential change anticipated for the '2080s' (2070 to 2115) |
|---|---|---|---|---|
| Thames | Upper end | 25% | 35% | 70% |
| | Higher central | 15% | 25% | 35% |
| | Central | 10% | 15% | 25% |

*Table 5: Peak river flow allowances by river basin district (Source EA)*

5.8 The worst case scenario was adopted and a value of 70% total potential change as a result of climate change was assumed.

5.9 The data provided by the EA included in-channel flows and corresponding flood levels. As such, obtaining a relationship between the flow in the river and the flood level, an extrapolation for the 1 in 100 year event + 70% CC was made.

5.10 The extent of the 1 in 100 year event + 35% CC is illustrated in Figure 3. The development is located outside this extent.

# 6. Flood Risk Management Measures

6.1 The following flood mitigation measures and recommendations are proposed:

- Air brick protection at ground floor level;
- Raise ground floors 300 mm above external ground levels where feasible;
- Non-return valves on sewers to prevent backflow;
- The route of all electrical services will run from ceilings down toward sockets at ground floor (where possible).

# 7. SUDS Assessment

7.1 In accordance with the SuDS management train approach, the use of various SuDS measures to reduce and control surface water flows have been considered in details for the development. Based on the hierarchy line provided by the London Borough of Richmond Upon Thames Development Management Plan surface water runoff should be addressed as follows:

| SuDS Drainage Hierarchy | | | |
|---|---|---|---|
| | | Suitability | Comment |
| 1. | store rainwater for later use | x | Not deemed feasible. |
| 2. | use infiltration techniques, such as porous surfaces in non-clay areas | ✓ | Possibly good infiltration rates (gravel) at shallow depths. To be confirmed. |
| 3. | attenuate rainwater in ponds or open water features for gradual release to a watercourse | x | Land between site and river not owned by developer |
| 4. | attenuate rainwater by storing in tanks or sealed water features for gradual release to a watercourse | x | Land between site and river not owned by developer |
| 5. | discharge rainwater direct to a watercourse | x | Land between site and river not owned by developer |
| 6. | discharge rainwater to a surface water drain | ✓ | Connection with existing infrastructure available |
| 7. | discharge rainwater to the combined sewer | - | |

*Table 6: SuDS Hierarchy*

7.2 There is scope for infiltration but this has not been tested by the Client to date. It was assumed that only nominal infiltration can be provided at this stage.

7.3 Thus, at this stage the practicality and viability of certain SuDS options are subject to confirmation of the on-site ground conditions and constraints presented by the site layout.

7.4 The suitability of various SuDS components has been assessed and the Table 7 - Suitability of SuDS components overleaf shows which are feasible on this site.

| Suitability of SuDS Components | | |
|---|---|---|
| SuDS Component | Description | Suitability |
| Infiltrating SuDS | Infiltration can contribute to reducing runoff rates and volumes while supporting baseflow and groundwater recharge processes. The suitability and infiltration rate depends on the permeability of the surrounding soils | ✓ To be confirmed |
| Permeable Pavement | Pervious surfaces can be used in combination with aggregate sub-base and/or geocellular/modular storage to attenuate and/or infiltrate runoff from surrounding surfaces and roofs. Liners can be used where ground conditions are not suitable for infiltration | ✓ |
| Green Roofs | Green Roofs provide areas of visual benefit, ecological value, enhanced building performance and the reduction of surface water runoff. They are generally more costly to install and maintain than conventional roofs but can provide many long-term benefits and reduce the on-site storage volumes | x |
| Rainwater Harvesting | Rainwater Harvesting is the collection of rainwater runoff for use. It can be collected form roofs or other impermeable area, stored, treated (where required) and then used as a supply of water for domestic, commercial and industrial properties | x |
| Swales | Swales are designed to convey, treat and attenuate surface water runoff and provide aesthetic and biodiversity benefits. They can replace conventional pipework as a means of conveying runoff, however space constraints of some sites can make it difficult incorporating them into the design | x |
| Rills and Channels | Rills and Channels keep runoff on the surface and convey runoff along the surface to downstream SuDS components. They can be incorporated into the design to provide a visually appealing method of conveyance, they also provide effectiveness in pre-treatment removal of silts | ✓ |
| Bioretention Systems | Bioretention systems can reduce runoff rates and volumes and treat pollution through the use of engineer soils and vegetation. They are particularly effective in delivering interception, but can also be an attractive landscape feature whilst providing habitat and biodiversity | ✓ |
| Retention Ponds and Wetlands | Ponds and Wetlands are features with a permanent pool of water that provide both attenuation and treatment of surface water runoff. They enhance treatment processes and have great amenity and biodiversity benefits. Often a flow control system at the outfall controls the rates of discharge for a range of water levels during storm events | x |
| Detention Basins | Detention Basins are landscaped depressions that are usually dry except during and immediately following storm events, and can be used as a recreational or other amenity facility. They generally appropriate to manage high volumes of surface water from larger sites such as a neighbourhoods | x |
| Geocellular Systems | Attenuation storage tanks are used to create a below-ground void space for the temporary storage of surface water before infiltration, controlled release or use. The inherent flexibility in size and shape means they can be tailored to suit the specific characteristics and requirements of any site | ✓ |
| Proprietary Treatment Systems | Proprietary treatment systems are manufactured products that remove specific pollutants from surface water runoff. They are especially useful where site constraints preclude the use of other methods and can be useful in reducing the maintenance requirements of downstream SuDS | ✓ |
| Filter Drains and Filter Strips | Filter drains are shallow trenches filled with stone, gravel that cerate temporary subsurface storage for the attenuation, conveyance and filtration of surface water runoff. Filter strips are uniformly graded and gently sloping strips of grass or dense vegetation, designed to treat runoff from adjacent impermeable areas by promoting sedimentation, filtration and infiltration | ✓ |

*Table 7 - Suitability of SuDS components*

7.5 Consequently, several SuDS components are deemed appropriate. It is suggested to use a SuDS train formed by ***Permeable Pavements*** with full infiltration (Type A) and ***Cellular Storage*** prior to outfall to the existing surface water sewer network, see Appendix D.

## Water Quality

7.6 Adequate treatment must be delivered to the water runoff to remove pollutants through SuDS devices which are able to provide pollution mitigation. Pollution Hazards and the SuDS Mitigation have been indexed in the **Ciria SuDS Manual.**

7.7 The pollution indices for the runoff from the proposed car park consisting of 4 spaces present within this development are mitigated by the treatment offered by the permeable pavement.

7.8 The runoff from the roof of the proposed building within this development are considered to pose very low pollution hazard.

## Adoption and Maintenance

7.9 All onsite SuDS and drainage systems will be privately maintained. A long term maintenance regime should be arranged by the site owners with a managing agent for all common areas before implementation.

7.10 In addition to a long term maintenance regime it is recommended that all drainage elements implemented on site should be inspected following the first rainfall event post construction and monthly for the first quarter following construction. Table 8 - Schedule of maintenance for drainage outlines the maintenance regime for below ground drainage on site.

| Item | Visual Inspection | Cleanse / De-sludge | CCTV Survey | Comments |
|---|---|---|---|---|
| Foul Drainage System (pipework, chambers etc.) | 5 years | 10 years | 10 years | Cleansing to be carried as necessary |
| Surface Water Drainage System (pipework, chambers etc.) | 5 years | 10 years | 10 years | Cleansing to be carried as necessary |
| Gullies/Channels | 1 year | 1 year | N/A | Cleansing to be carried as necessary |
| Petrol interceptor: Chamber & Alarm | 1 year | 1 year | N/A | Maintenance in accordance with manufacturer's instructions |
| Soakaways and catchpits | 1 year | | N/A | Cleansing to be carried as necessary |
| Permeable Tarmac Paving | 1 year | 'Swept' clean of debris every 2 years. | N/A | Jetwash or suction roadsweep permeable tarmac as performance levels reduce. |
| Permeable Block Paving | 1 year | 'Swept' clean of debris every 2 years. | N/A | Lift blocks and remove sand bedding and replace and re-bed paving – refer to individual manufacturers recommendations. |
| Cellular storage | 1 year | | 5 years | Cleansing to be carried as necessary |

*Table 8 - Schedule of maintenance for drainage*

# 8. Surface Water Drainage

## Drainage Strategy

8.1 Appendix D, Figure 1 illustrates the preliminary drainage strategy of the site.

8.2 The drainage strategy of the proposed development will comprise two systems:

- A geocellular tank will attenuate the surface water runoff from the roof of the proposed building. The outlet of this tank will be to the existing sewer network at a maximum discharge rate of 1.0l/s. The existing drainage network will need to be surveyed to confirm position and depth. A pump may be required at the outlet of the geocellular tank if the existing drainage is shallow. The area draining into this tank is 200m$^2$. The position of the tank as shown in Appendix D does not allow infiltration due to the proximity to the proposed building. Full or partial infiltration may be possible for this tank but infiltration tests are needed to confirm this. If infiltration is feasible the tank will need to be located at a distance greater than 5m from the proposed building and any offsite drainage connection amended to suit.
- The runoff from the additional four car park spaces on the west side of the access road and the two carpark spaces west of the mews will be treated and attenuated using permeable pavement Type A (full infiltration). The size of this area is 122m$^2$. The very slow rates of infiltration are enough to prevent the site from flooding (as shown in Appendix C – Calculations). The proposed nominal depth of 0.4m provides enough storage when assuming the most conservative infiltration coefficient for the soil.

8.3 The design strategy illustrated in Appendix D successfully limits the peak flow runoff from the 1:100 year +40% climate change to 1.0l/s. It provides a total volume of storage of 17.6m$^3$ (obtained from the sum of the storage volumes of the geocellular tanks – 4.7m$^3$ and the pervious pavements – 12.9m$^3$). Nominal infiltration has been allowed for at this stage.

8.4 Calculations simulating the proposed drainage arrangement are included in Appendix 3 and show the site does not flood during all storm events up to and including the 1 in 100yr + CC rainfall event.

## Runoff rates

8.5 As described within the CIRIA SuDS manual the aspiration of any development is to achieve the pre-development greenfield runoff rates or as close as feasible.

8.6 The London Borough of Richmond Upon Thames Develoment Management Plan states that "a*ny discharge should be reduced to greenfield run-off rates wherever feasible*".

8.7 Concerning small greenfield runoff rates The Planning Guidance Document "Delivering SuDS in Richmond" states that "for smaller sites these rates may not be achievable because the minimum acceptable orifice size is 20mm (if protected from blockage)". As such design runoff rates should be finalised based on a 20mm orifice diameter.

8.8 The greenfield QBar runoff rate calculated for the considered area is 0.1l/s and the 1 in 100 year greenfield rate is 0.3l/s. Calculations are provided in Appendix C. It is not deemed feasible to discharge at such low rates. Therefore, the limiting discharge was designed to provide significant betterment from the existing situation.

8.9 The limiting discharge from the geocellular tank was set to 1l/s as this is deemed as close as practically feasible to the greenfield runoff rate for the 1 in 100 year event while not causing eventual issues regarding the size of the outlet.

8.10 The proposed permeable pavement of 122m$^2$ will discharge by full infiltration into the ground.

| SURFACE WATER DISCHARGE RATES SUMMARY | | | | | | |
|---|---|---|---|---|---|---|
| | Impermeable/Hardstanding Area (m²) | Green Area (m²) | Discharge Rates (l/s) | | | |
| | | | 1 year | $Q_{BAR}$ | 30 year | 100 year |
| Greenfield Site | 0 | 548 | 0.1 | 0.1 | 0.2 | 0.3 |
| Existing Hard surface runoff rates | 123 | 425 | 1.7 | | 4.2 | 5.4 |
| Limiting Discharge for Proposed Site | 272 | 276 | 1.0 | 1.0 | 1.0 | 1.0 |
| Betterment | | | **41%** | | **76%** | **81%** |

*Table 9 – Surface Water Discharge Rates Summary*

## Interception Storage

8.11 Preliminary calculations have been carried out for a typical rainfall depth of 5 mm to store the volume owing to these very frequent storms.

8.12 As per *CIRIA 753 'The SUDS Manual' storage relating to* 80% of runoff from the first 5mm of a rainfall event should be achieved for summer rainfall events. **Based on the size of the impervious area of the site and the Runoff Percentage, the Interception Storage is 1.4 m³.**

8.13 The permeable pavement provides infiltration which is an effective way of delivering interception.

8.14 The tank can only be partially lined (up to 5m away from the building) to allow for nominal infiltration if the water table is shown to be deep enough.

## Long Term Storage

8.15 Long term storage is not taken into account as the obtained approach limits all peak runoff rates to a value close to the greenfield runoff rate.

## Attenuation Storage

8.16 Attenuation storage is needed to temporarily store water during periods when the runoff rates from the development site exceed the allowable discharge rates from the site.

8.17 Rainfall depths for the 1 in 100 years Return Period plus 40% of climate change were produced using the *Microdrainage* software in order to estimate the largest volume, *critical storm*, for typical storm durations for the proposed site limiting the discharge rate up to a rate of 1.0 l/s. See summary calculations in Appendix 3, Calculations.

8.18 Thus, it meets with the minimum standards required by the DEFRA - Non-statutory technical standards for sustainable drainage systems (March 2015) to avoid the flood risk within the development in a 1 in 100 year rainfall event.

8.19 In terms of storage, for a 100 years storm event with an allowance for climate change, **the Attenuation Storage Volume required for the whole site is 17.6m³**. See Appendix 3.

8.20 The half drain duration of the proposed permeable pavement is currently long. This is heavily dependent on the actual infiltration coefficient of the soil and as such potentially better coefficient than the assumed 0.36m/hr will decrease this duration. If after infiltration tests are conducted, the half drain duration is still calculated to be more than 24 hours, this is considered a minor residual risk since the area that is being drained to the permeable pavement is relatively small.

## Design Exceedance

8.21 In the event of drainage system failure under extreme rainfall events or blockage, flooding may occur within the site. In the event of the development's drainage system failure, the runoff flow will be dictated by topography on site. This will not impact on the site or nearby dwellings. Design of external ground levels will need to be undertaken at detailed design stage to finalise these routes but some indicative flow paths have been indicated on the outline strategy drawings.

8.22 It is advised that the finished floor level of the proposed buildings should be 300mm above surrounding ground levels where feasible, to mitigate against any potential surface water flows.

# 9. Conclusions

9.1 This study has been undertaken in accordance with the principles set out in NPPF. We can conclude that, providing the development adheres to the conditions advised in the conclusions of this report, the said development proposals can be accommodated without increasing flood risk within the locality in accordance with objectives set by Central Government and the EA.

9.2 The strategy for drainage of this site is to infiltrate the area of the proposed new 5 car park spaces through a Type A permeable pavement. The runoff from the roof is to be discharged to the sewer network utilising a geocellular tank with managed offsite flows controlled by Hydrobrake, or similar flow control, as necessary. Infiltration rates are to be confirmed but local geology suggests some infiltration may be feasible.

9.3 Initial calculations indicate a storage requirement of approximately 17.6 $m^3$, being properly managed by the proposed SuDS train. This can be accommodated through the proposed SuDS train including a geocellular tank to provide 4.7m3 and permeable pavement sub-base to provide 12.9$m^3$ of storage

9.4 The treatment provided by the permeable paving is suitable to offer acceptable contamination treatment to runoff prior to infiltration. The runoff from the roof is considered uncontaminated and sediment traps are deemed sufficient to provide the required treatment prior to discharge to the sewer.

9.5 The findings and recommendations of this report are for the use of the client who commissioned the assessment, and no responsibility or liability can be accepted for the use of the report or its findings by any other person or for any other purpose.

9.6 This report is not intended to offer a full detailed design solution but to show that water runoff can be accommodated and managed on site. Further detailed design and regulatory approval may be necessary.

Dr. J. B. Butler
B.Sc., M.Phil., PhD.
Ambiental Technical Solutions Ltd. February 2019

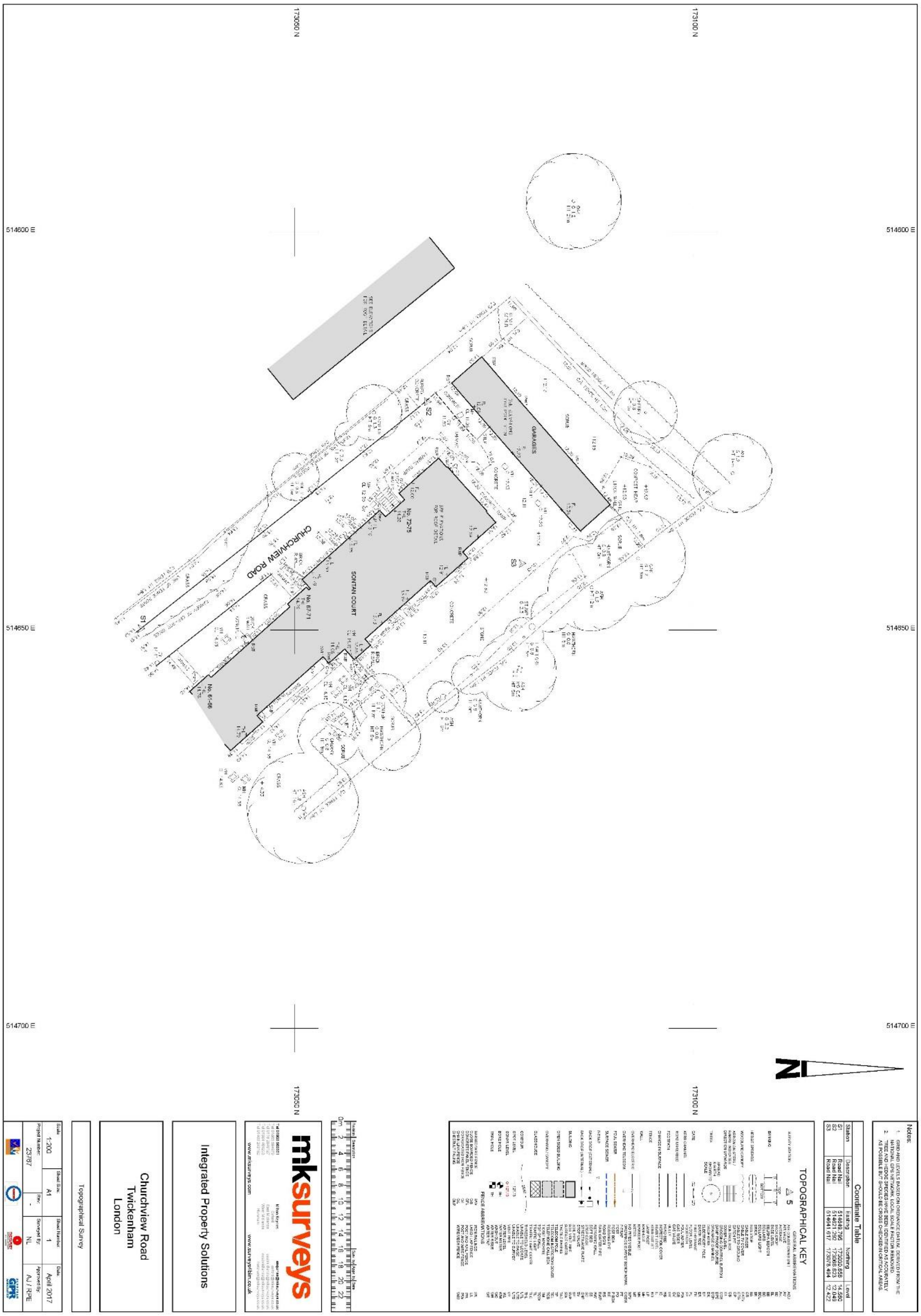
# Appendix A – Plans

Appendix A, Figure 1 – Site Location (Source: UK & European Property Developments Ltd)

Appendix A, Figure 2 – Existing Topographical Map 1of2 (Source: MK Surveys)

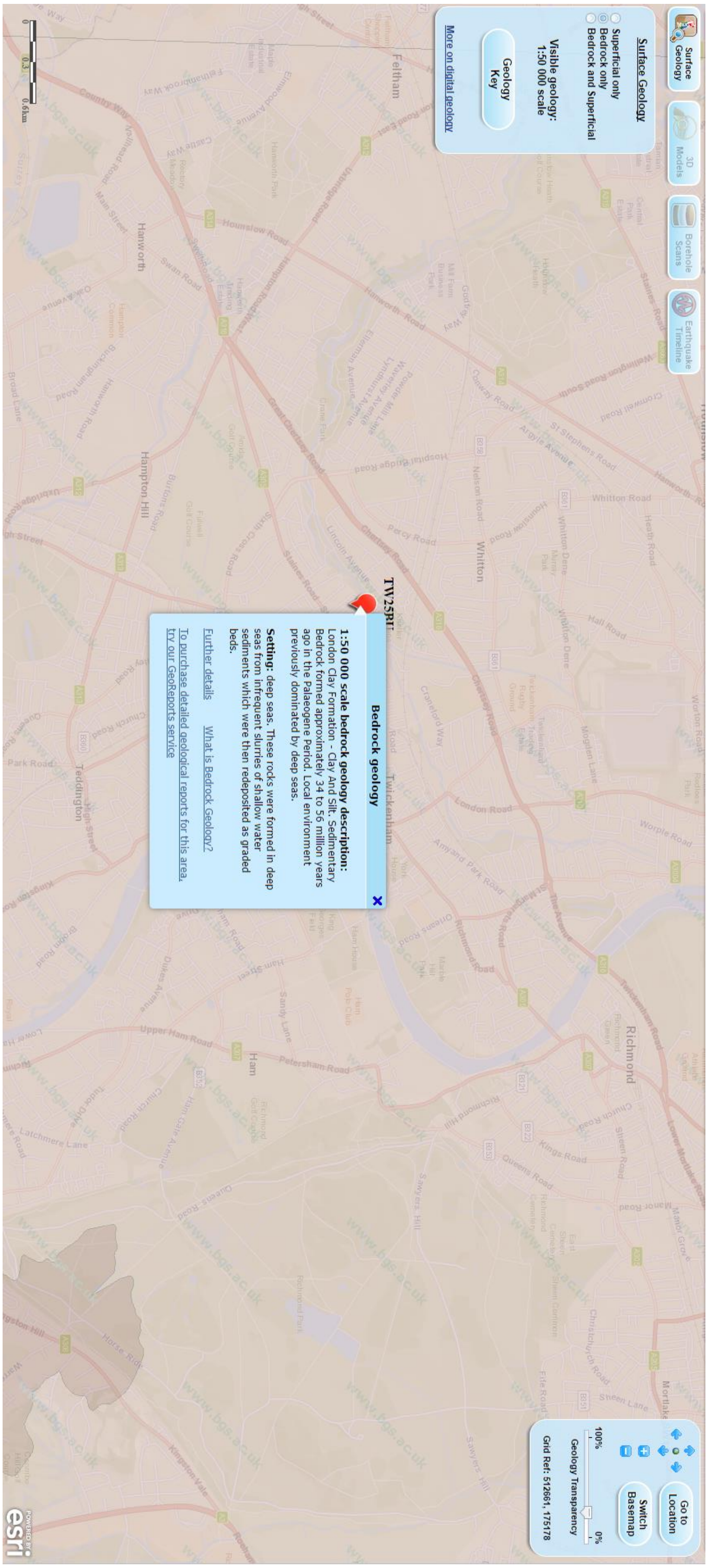
Appendix A, Figure 3 – Proposed plan


*Appendix A, Figure 1 – Site Location (Source: UK & European Property Developments Ltd)*

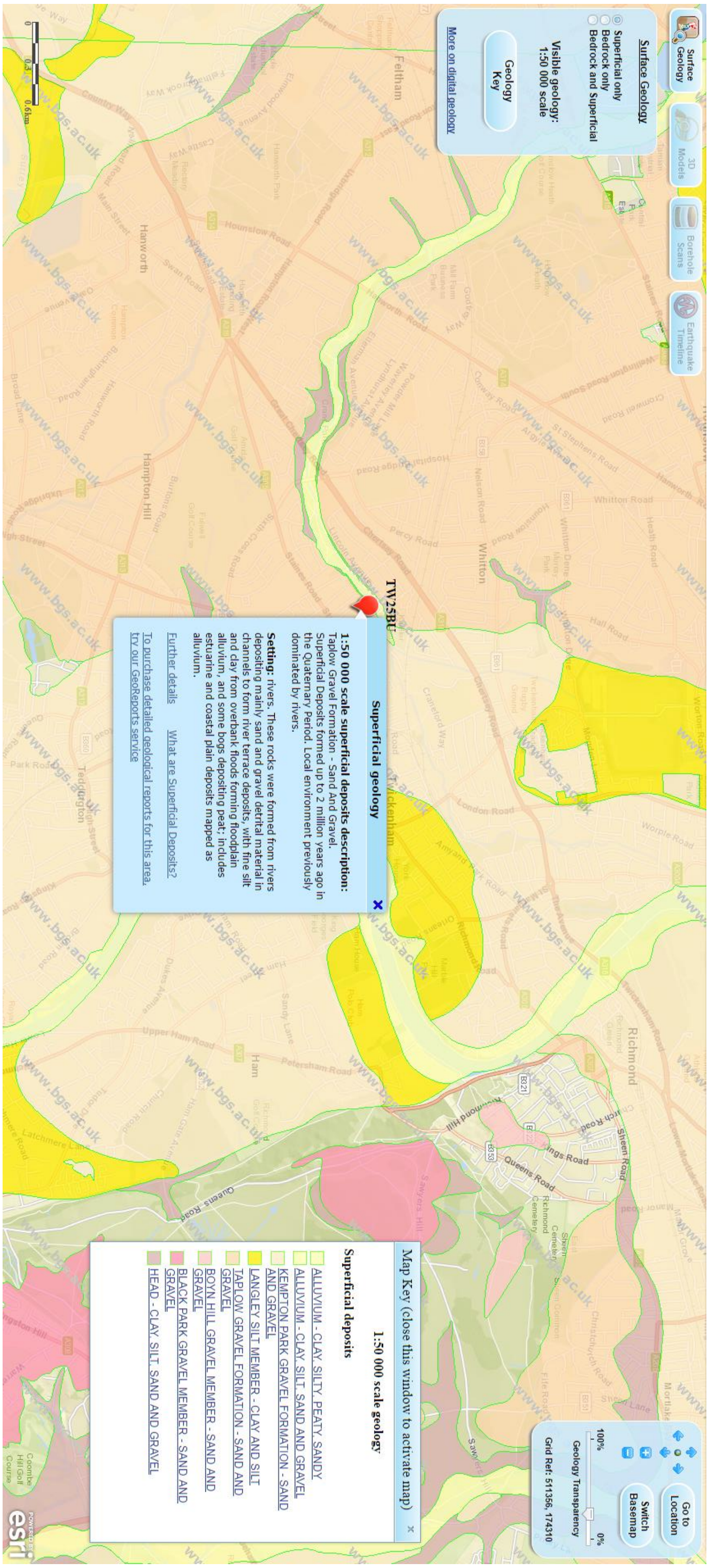



*Appendix A, Figure 2 – Existing Topographical Map 1of2 (Source: MK Surveys)*

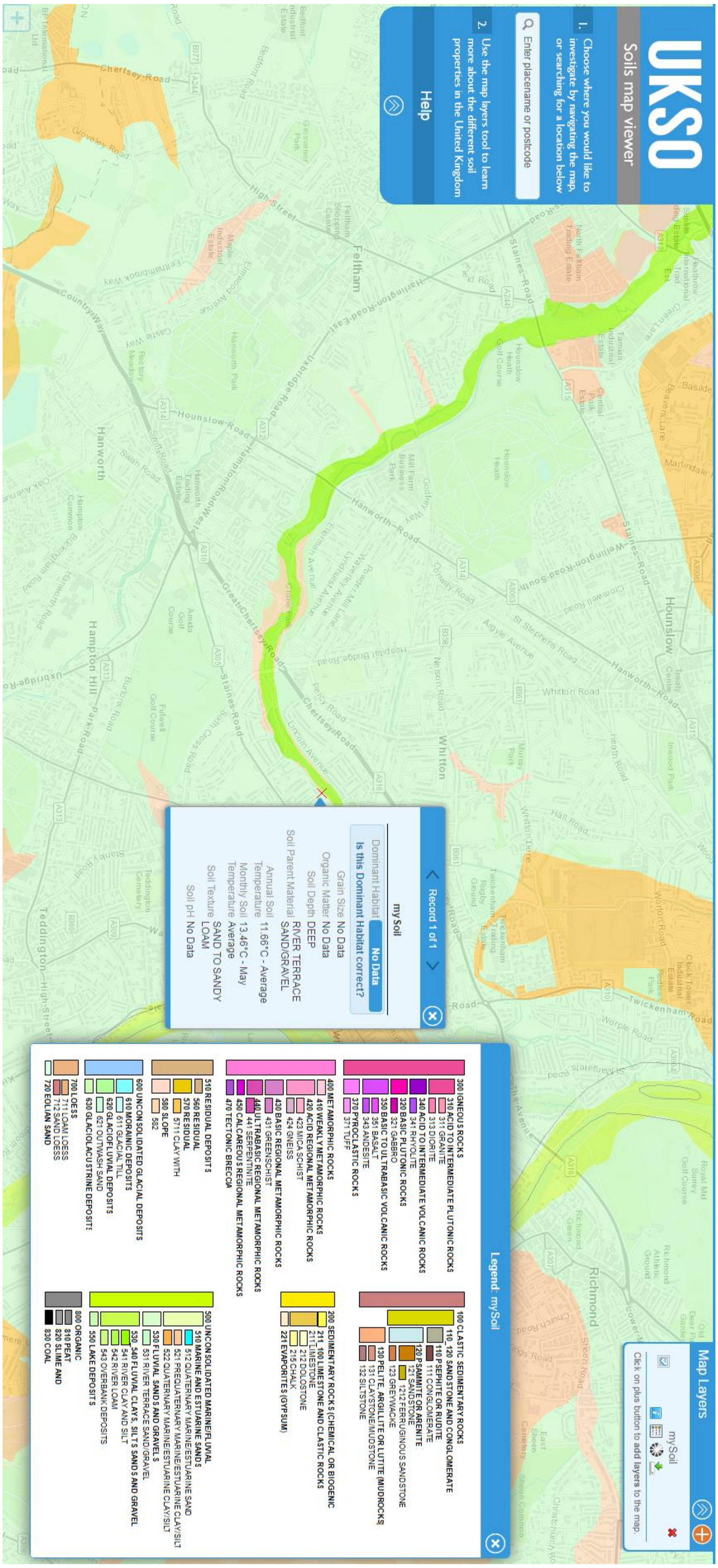
# Appendix B – Site Geology Maps and Ground Investigation




*Appendix B, Figure 1 – Bedrock Geology*

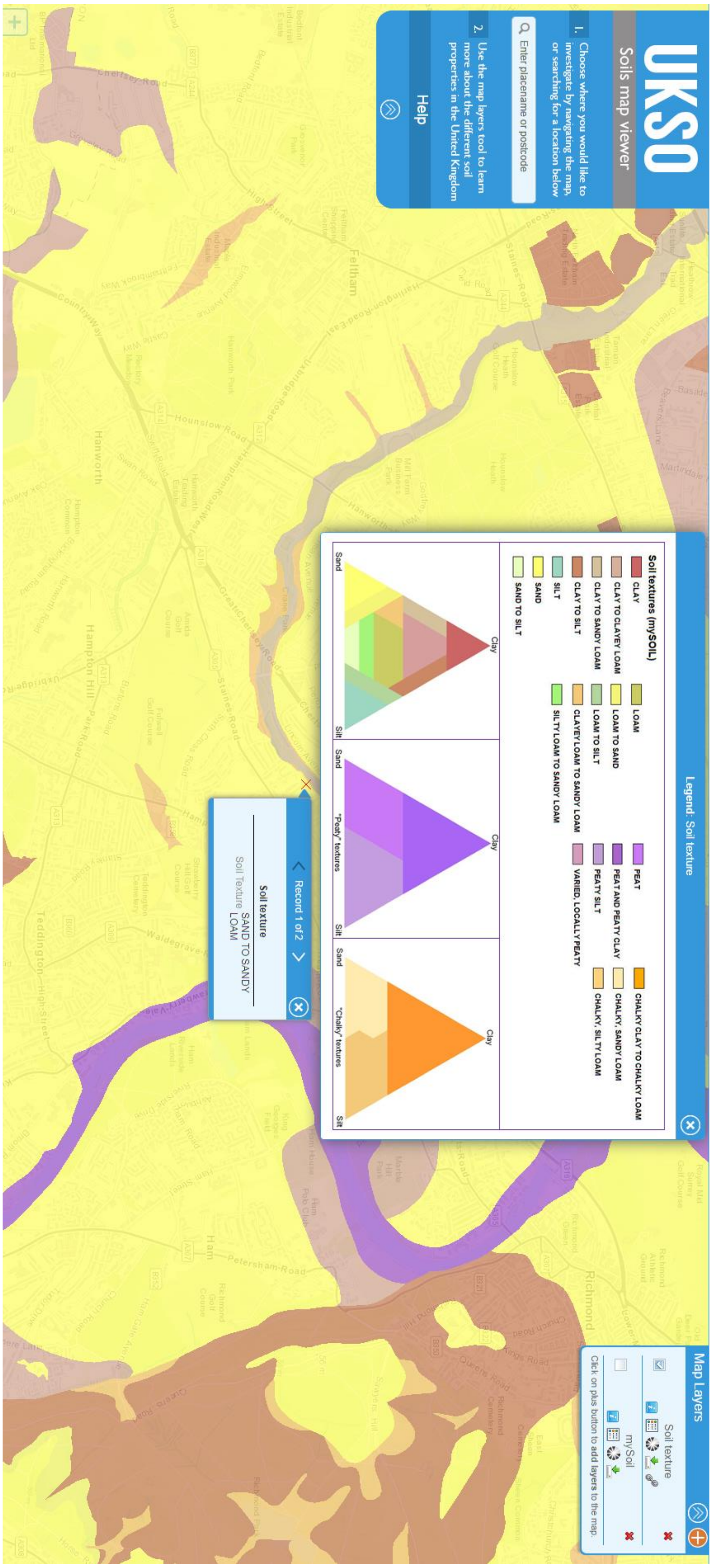



*Appendix B, Figure 2 - Superficial Deposits*

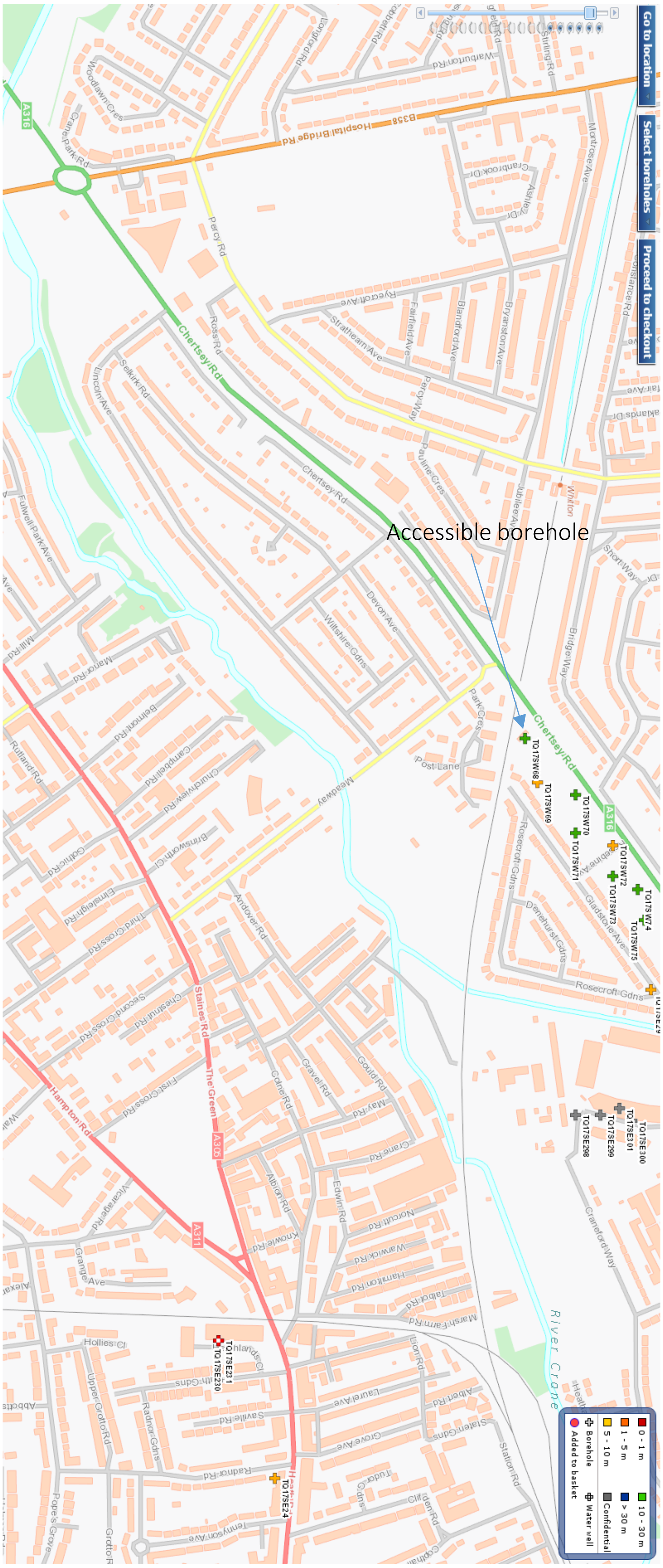



*Appendix B, Figure 3 - Soil Parental Material*

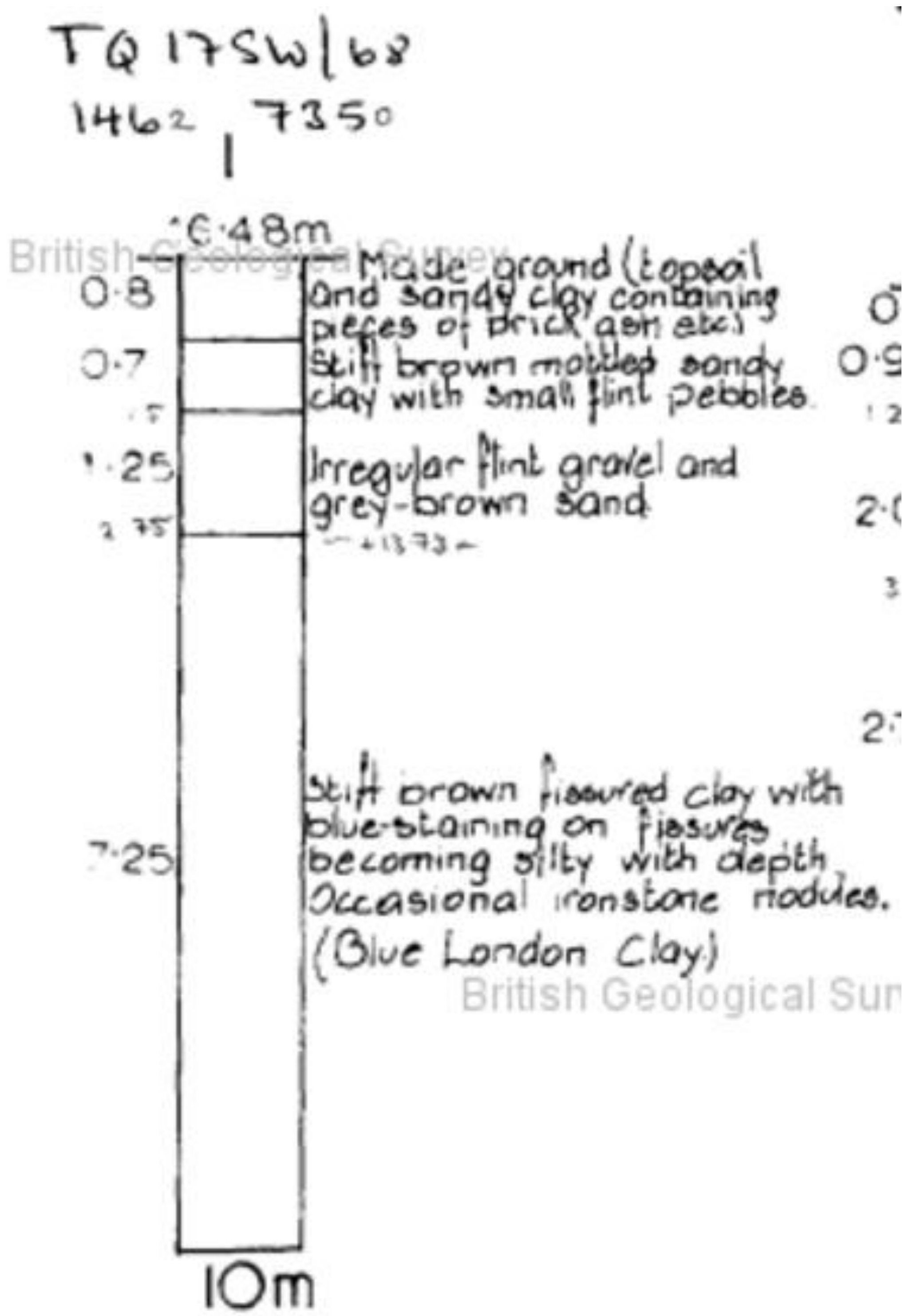
ambiental



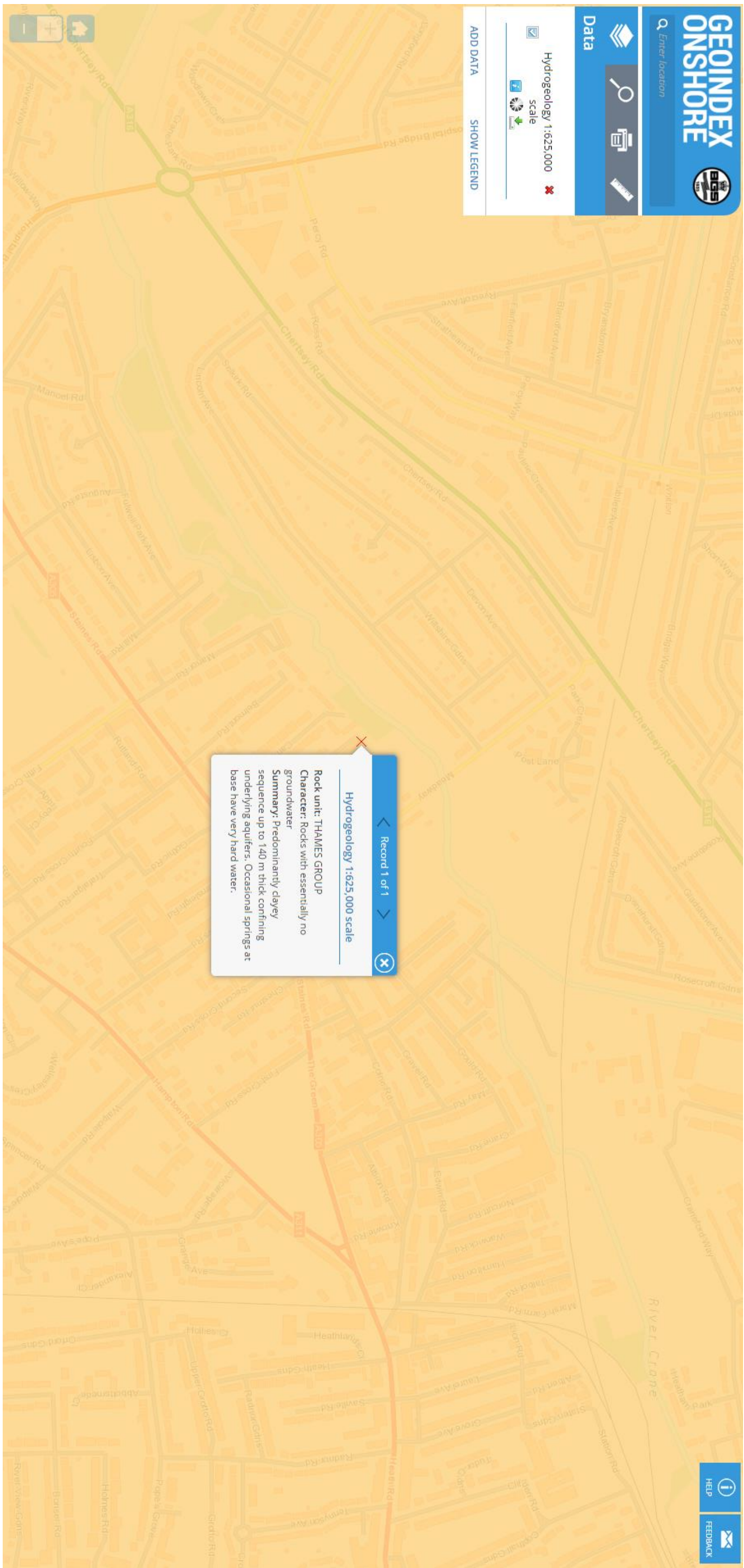

*Appendix B, Figure 4 - Soil Texture*



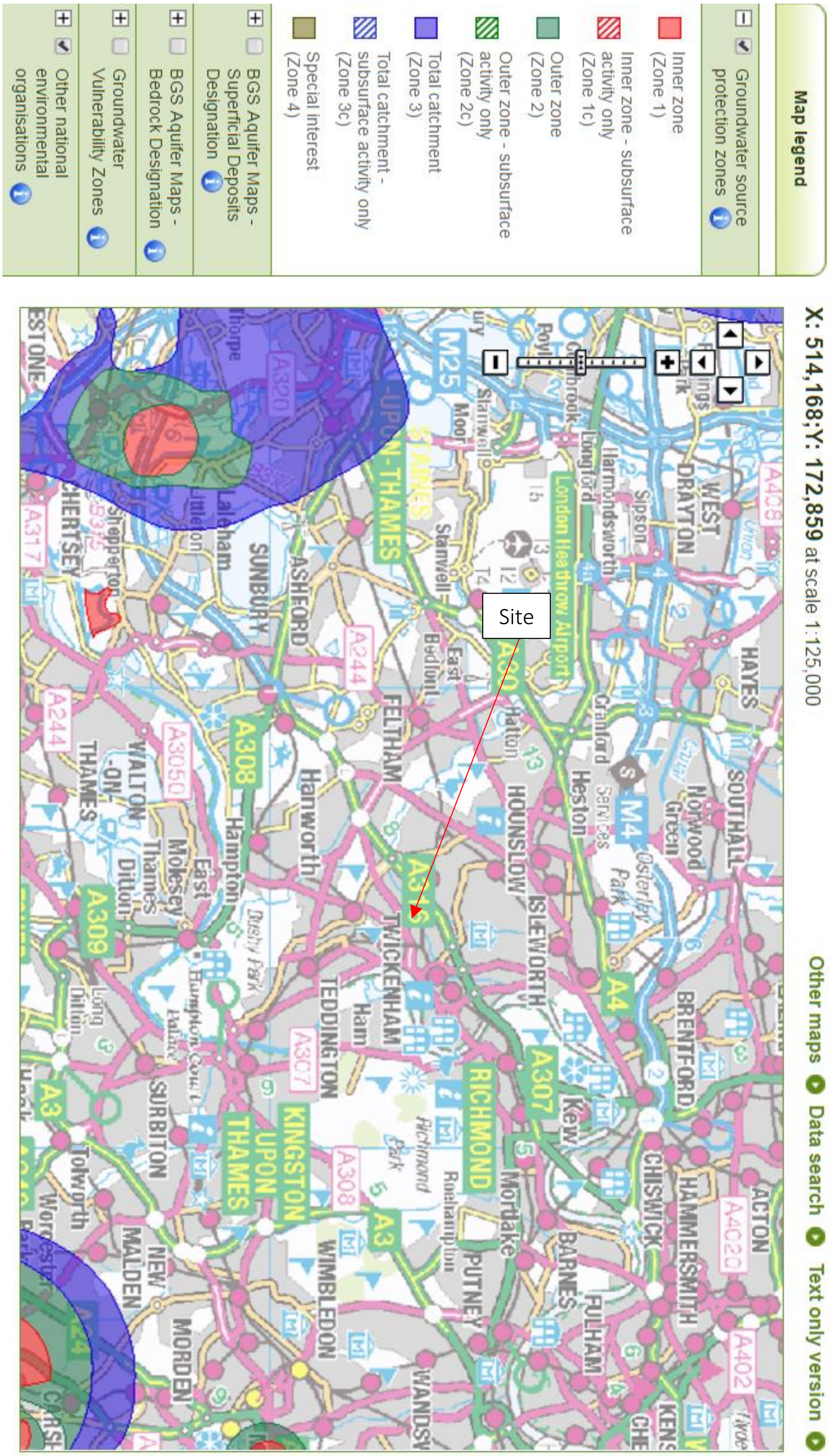

*Appendix B, Figure 5 - Boreholes Map*



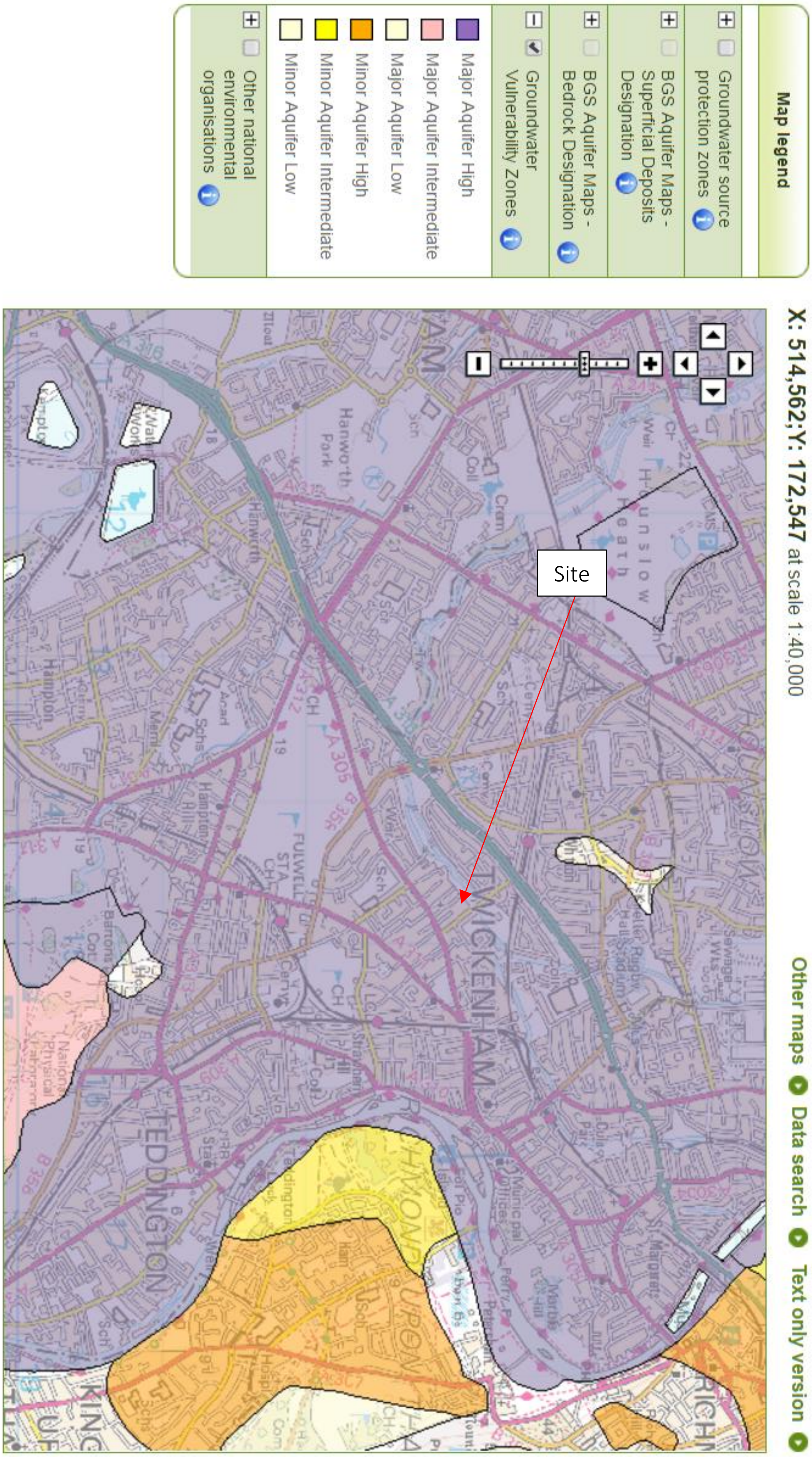

*Appendix B, Figure 6 - Borehole TQ79SW2 (Groundwater conditions not recorded)*




*Appendix B, Figure 7 - Hydrogeology*




*Appendix B, Figure 8 - Groundwater Source Protection Zones*




*Appendix B, Figure 9 - Groundwater Vulnerability Zones*

# Appendix C – Calculations

Appendix C – Greenfield Peak Runoff

Appendix C – Existing Runoff Rate

Appendix C –Geocellular Tank Storage Calculations

Appendix C – Permeable Pavement Storage Calculations

## ICP SUDS Mean Annual Flood

Input

| | | | |
|---|---|---|---|
| Return Period (years) | 100 | Soil | 0.300 |
| Area (ha) | 0.054 | Urban | 0.000 |
| SAAR (mm) | 599 | Region Number | Region 6 |

| **Results** | **l/s** |
|---|---|
| QBAR Rural | 0.1 |
| QBAR Urban | 0.1 |
| | |
| Q100 years | 0.3 |
| | |
| Q1 year | 0.1 |
| Q30 years | 0.2 |
| Q100 years | 0.3 |

## 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 (%) | 95 |
| M5-60 (mm) | 20.000 | Add Flow / Climate Change (%) | 0 |
| Ratio R | 0.400 | Minimum Backdrop Height (m) | 0.200 |
| Maximum Rainfall (mm/hr) | 50 | Maximum Backdrop Height (m) | 1.500 |
| Maximum Time of Concentration (mins) | 30 | Min Design Depth for Optimisation (m) | 1.200 |
| 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) |
|---|---|---|---|
| 0-4 | 0.011 | 4-8 | 0.001 |

Total Area Contributing (ha) = 0.012

Total Pipe Volume (m³) = 0.994

### Network Design Table for Storm

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.000 | 20.000 | 0.500 | 40.0 | 0.011 | 4.00 | 0.0 | 0.600 | o | 225 | Pipe/Conduit | |
| 1.001 | 5.000 | 0.063 | 79.4 | 0.000 | 0.00 | 0.0 | 0.600 | o | 225 | 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) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.000 | 50.00 | 4.16 | 11.000 | 0.011 | 0.0 | 0.0 | 0.0 | 2.07 | 82.5 | 1.5 |
| 1.001 | 50.00 | 4.22 | 10.500 | 0.011 | 0.0 | 0.0 | 0.0 | 1.47 | 58.4 | 1.5 |

### Free Flowing Outfall Details for Storm

| Outfall Pipe Number | Outfall Name | C. Level (m) | I. Level (m) | Min I. Level (m) | D,L (mm) | W (mm) |
|---|---|---|---|---|---|---|
| 1.001 | | 11.000 | 10.437 | 0.000 | 500 | 0 |

## Simulation Criteria for Storm

| | | | |
|---|---|---|---|
| Volumetric Runoff Coeff | 0.750 | Additional Flow - % of Total Flow | 0.000 |
| Areal Reduction Factor | 1.000 | MADD Factor * 10m³/ha Storage | 2.000 |
| Hot Start (mins) | 0 | Inlet Coeffiecient | 0.800 |
| Hot Start Level (mm) | 0 | Flow per Person per Day (l/per/day) | 0.000 |
| Manhole Headloss Coeff (Global) | 0.500 | Run Time (mins) | 60 |
| Foul Sewage per hectare (l/s) | 0.000 | Output Interval (mins) | 1 |

| | | | |
|---|---|---|---|
| Number of Input Hydrographs | 0 | Number of Storage Structures | 0 |
| Number of Online Controls | 0 | Number of Time/Area Diagrams | 0 |
| Number of Offline Controls | 0 | Number of Real Time Controls | 0 |

## Synthetic Rainfall Details

| | | | |
|---|---|---|---|
| Rainfall Model | FSR | Profile Type | Summer |
| Return Period (years) | 100 | Cv (Summer) | 0.750 |
| Region | England and Wales | Cv (Winter) | 0.840 |
| M5-60 (mm) | 20.000 | Storm Duration (mins) | 30 |
| Ratio R | 0.400 | | |

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

### Simulation Criteria

| | | | |
|---|---|---|---|
| Areal Reduction Factor | 1.000 | Additional Flow - % of Total Flow | 0.000 |
| Hot Start (mins) | 0 | MADD Factor * 10m³/ha Storage | 2.000 |
| Hot Start Level (mm) | 0 | Inlet Coeffiecient | 0.800 |
| Manhole Headloss Coeff (Global) | 0.500 | Flow per Person per Day (l/per/day) | 0.000 |
| Foul Sewage per hectare (l/s) | 0.000 | | |

| | | | |
|---|---|---|---|
| Number of Input Hydrographs | 0 | Number of Storage Structures | 0 |
| Number of Online Controls | 0 | Number of Time/Area Diagrams | 0 |
| Number of Offline Controls | 0 | Number of Real Time Controls | 0 |

### Synthetic Rainfall Details

| | | | |
|---|---|---|---|
| Rainfall Model | FSR | Ratio R | 0.400 |
| Region | England and Wales | Cv (Summer) | 0.750 |
| M5-60 (mm) | 20.000 | Cv (Winter) | 0.840 |

| | | | |
|---|---|---|---|
| Margin for Flood Risk Warning (mm) | 275.0 | DVD Status | OFF |
| Analysis Timestep | Fine | Inertia Status | OFF |
| DTS Status | ON | | |

| | |
|---|---|
| Profile(s) | Summer and Winter |
| Duration(s) (mins) | 15, 30, 60, 120, 240, 360, 480, 960, 1440 |
| Return Period(s) (years) | 1, 30, 100 |
| Climate Change (%) | 0, 0, 0 |

| PN | US/MH Name | Storm | Return Period | Climate Change | First (X) Surcharge | First (Y) Flood | First (Z) Overflow | Overflow Act. | Water Level (m) |
|---|---|---|---|---|---|---|---|---|---|
| 1.000 | 1 | 15 Summer | 1 | +0% | | | | | 11.023 |
| 1.001 | 2 | 15 Winter | 1 | +0% | | | | | 10.532 |

| PN | US/MH Name | Surcharged Depth (m) | Flooded Volume (m³) | Flow / Cap. | Overflow (l/s) | Pipe Flow (l/s) | Status | Level Exceeded |
|---|---|---|---|---|---|---|---|---|
| 1.000 | 1 | -0.202 | 0.000 | 0.02 | | 1.7 | OK | |
| 1.001 | 2 | -0.193 | 0.000 | 0.05 | | 1.7 | OK | |

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

### Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 0
Number of Online Controls 0 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

### Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.400
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 275.0 DVD Status OFF
Analysis Timestep Fine Inertia Status OFF
DTS Status ON

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 0

| PN | US/MH Name | Storm | Return Period | Climate Change | First (X) Surcharge | First (Y) Flood | First (Z) Overflow | Overflow Act. | Water Level (m) |
|---|---|---|---|---|---|---|---|---|---|
| 1.000 | 1 | 15 Winter | 30 | +0% | | | | | 11.034 |
| 1.001 | 2 | 15 Summer | 30 | +0% | | | | | 10.551 |

| PN | US/MH Name | Surcharged Depth (m) | Flooded Volume (m³) | Flow / Cap. | Overflow (l/s) | Pipe Flow (l/s) | Status | Level Exceeded |
|---|---|---|---|---|---|---|---|---|
| 1.000 | 1 | -0.191 | 0.000 | 0.06 | | 4.2 | OK | |
| 1.001 | 2 | -0.174 | 0.000 | 0.12 | | 4.2 | OK | |

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

### Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 0
Number of Online Controls 0 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

### Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.400
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 275.0 DVD Status OFF
Analysis Timestep Fine Inertia Status OFF
DTS Status ON

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 0

| PN | US/MH Name | Storm | Return Period | Climate Change | First (X) Surcharge | First (Y) Flood | First (Z) Overflow | Overflow Act. | Water Level (m) |
|---|---|---|---|---|---|---|---|---|---|
| 1.000 | 1 | 15 Winter | 100 | +0% | | | | | 11.040 |
| 1.001 | 2 | 15 Summer | 100 | +0% | | | | | 10.559 |

| PN | US/MH Name | Surcharged Depth (m) | Flooded Volume (m³) | Flow / Cap. | Overflow (l/s) | Pipe Flow (l/s) | Status | Level Exceeded |
|---|---|---|---|---|---|---|---|---|
| 1.000 | 1 | -0.185 | 0.000 | 0.07 | | 5.4 | OK | |
| 1.001 | 2 | -0.166 | 0.000 | 0.15 | | 5.4 | OK | |

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

Half Drain Time : 42 minutes.

| Storm Event | Max Level (m) | Max Depth (m) | Max Infiltration (l/s) | Max Control (l/s) | Max Σ Outflow (l/s) | Max Volume (m³) | Status |
|---|---|---|---|---|---|---|---|
| 15 min Summer | 11.826 | 0.226 | 0.0 | 1.0 | 1.0 | 3.2 | O K |
| 30 min Summer | 11.873 | 0.273 | 0.0 | 1.0 | 1.0 | 3.9 | O K |
| 60 min Summer | 11.888 | 0.288 | 0.0 | 1.0 | 1.0 | 4.1 | O K |
| 120 min Summer | 11.869 | 0.269 | 0.0 | 1.0 | 1.0 | 3.8 | O K |
| 180 min Summer | 11.835 | 0.235 | 0.0 | 1.0 | 1.0 | 3.4 | O K |
| 240 min Summer | 11.802 | 0.202 | 0.0 | 1.0 | 1.0 | 2.9 | O K |
| 360 min Summer | 11.746 | 0.146 | 0.0 | 1.0 | 1.0 | 2.1 | O K |
| 480 min Summer | 11.708 | 0.108 | 0.0 | 1.0 | 1.0 | 1.5 | O K |
| 600 min Summer | 11.684 | 0.084 | 0.0 | 1.0 | 1.0 | 1.2 | O K |
| 720 min Summer | 11.671 | 0.071 | 0.0 | 0.9 | 0.9 | 1.0 | O K |
| 960 min Summer | 11.658 | 0.058 | 0.0 | 0.8 | 0.8 | 0.8 | O K |
| 1440 min Summer | 11.645 | 0.045 | 0.0 | 0.6 | 0.6 | 0.6 | O K |
| 2160 min Summer | 11.636 | 0.036 | 0.0 | 0.4 | 0.4 | 0.5 | O K |
| 2880 min Summer | 11.632 | 0.032 | 0.0 | 0.3 | 0.3 | 0.4 | O K |
| 4320 min Summer | 11.626 | 0.026 | 0.0 | 0.2 | 0.2 | 0.4 | O K |
| 5760 min Summer | 11.623 | 0.023 | 0.0 | 0.2 | 0.2 | 0.3 | O K |
| 7200 min Summer | 11.621 | 0.021 | 0.0 | 0.2 | 0.2 | 0.3 | O K |
| 8640 min Summer | 11.619 | 0.019 | 0.0 | 0.1 | 0.1 | 0.3 | O K |
| 10080 min Summer | 11.618 | 0.018 | 0.0 | 0.1 | 0.1 | 0.3 | O K |
| 15 min Winter | 11.858 | 0.258 | 0.0 | 1.0 | 1.0 | 3.7 | O K |

| Storm Event | Rain (mm/hr) | Flooded Volume (m³) | Discharge Volume (m³) | Time-Peak (mins) |
|---|---|---|---|---|
| 15 min Summer | 138.153 | 0.0 | 3.9 | 17 |
| 30 min Summer | 90.705 | 0.0 | 5.1 | 31 |
| 60 min Summer | 56.713 | 0.0 | 6.4 | 48 |
| 120 min Summer | 34.246 | 0.0 | 7.7 | 82 |
| 180 min Summer | 25.149 | 0.0 | 8.5 | 116 |
| 240 min Summer | 20.078 | 0.0 | 9.0 | 148 |
| 360 min Summer | 14.585 | 0.0 | 9.8 | 208 |
| 480 min Summer | 11.622 | 0.0 | 10.5 | 264 |
| 600 min Summer | 9.738 | 0.0 | 10.9 | 318 |
| 720 min Summer | 8.424 | 0.0 | 11.4 | 374 |
| 960 min Summer | 6.697 | 0.0 | 12.0 | 492 |
| 1440 min Summer | 4.839 | 0.0 | 13.1 | 734 |
| 2160 min Summer | 3.490 | 0.0 | 14.1 | 1100 |
| 2880 min Summer | 2.766 | 0.0 | 14.9 | 1468 |
| 4320 min Summer | 1.989 | 0.0 | 16.1 | 2180 |
| 5760 min Summer | 1.573 | 0.0 | 17.0 | 2896 |
| 7200 min Summer | 1.311 | 0.0 | 17.7 | 3664 |
| 8640 min Summer | 1.129 | 0.0 | 18.3 | 4368 |
| 10080 min Summer | 0.994 | 0.0 | 18.8 | 5104 |
| 15 min Winter | 138.153 | 0.0 | 4.3 | 17 |

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

| Storm Event | Max Level (m) | Max Depth (m) | Max Infiltration (l/s) | Max Control (l/s) | Max Σ Outflow (l/s) | Max Volume (m³) | Status |
|---|---|---|---|---|---|---|---|
| 30 min Winter | 11.913 | 0.313 | 0.0 | 1.0 | 1.0 | 4.5 | O K |
| 60 min Winter | 11.929 | 0.329 | 0.0 | 1.0 | 1.0 | 4.7 | O K |
| 120 min Winter | 11.901 | 0.301 | 0.0 | 1.0 | 1.0 | 4.3 | O K |
| 180 min Winter | 11.849 | 0.249 | 0.0 | 1.0 | 1.0 | 3.5 | O K |
| 240 min Winter | 11.796 | 0.196 | 0.0 | 1.0 | 1.0 | 2.8 | O K |
| 360 min Winter | 11.716 | 0.116 | 0.0 | 1.0 | 1.0 | 1.7 | O K |
| 480 min Winter | 11.675 | 0.075 | 0.0 | 1.0 | 1.0 | 1.1 | O K |
| 600 min Winter | 11.663 | 0.063 | 0.0 | 0.8 | 0.8 | 0.9 | O K |
| 720 min Winter | 11.655 | 0.055 | 0.0 | 0.7 | 0.7 | 0.8 | O K |
| 960 min Winter | 11.645 | 0.045 | 0.0 | 0.6 | 0.6 | 0.6 | O K |
| 1440 min Winter | 11.637 | 0.037 | 0.0 | 0.4 | 0.4 | 0.5 | O K |
| 2160 min Winter | 11.630 | 0.030 | 0.0 | 0.3 | 0.3 | 0.4 | O K |
| 2880 min Winter | 11.626 | 0.026 | 0.0 | 0.2 | 0.2 | 0.4 | O K |
| 4320 min Winter | 11.622 | 0.022 | 0.0 | 0.2 | 0.2 | 0.3 | O K |
| 5760 min Winter | 11.619 | 0.019 | 0.0 | 0.1 | 0.1 | 0.3 | O K |
| 7200 min Winter | 11.618 | 0.018 | 0.0 | 0.1 | 0.1 | 0.3 | O K |
| 8640 min Winter | 11.616 | 0.016 | 0.0 | 0.1 | 0.1 | 0.2 | O K |
| 10080 min Winter | 11.615 | 0.015 | 0.0 | 0.1 | 0.1 | 0.2 | O K |

| Storm Event | Rain (mm/hr) | Flooded Volume (m³) | Discharge Volume (m³) | Time-Peak (mins) |
|---|---|---|---|---|
| 30 min Winter | 90.705 | 0.0 | 5.7 | 30 |
| 60 min Winter | 56.713 | 0.0 | 7.1 | 52 |
| 120 min Winter | 34.246 | 0.0 | 8.6 | 90 |
| 180 min Winter | 25.149 | 0.0 | 9.5 | 126 |
| 240 min Winter | 20.078 | 0.0 | 10.1 | 158 |
| 360 min Winter | 14.585 | 0.0 | 11.0 | 214 |
| 480 min Winter | 11.622 | 0.0 | 11.7 | 260 |
| 600 min Winter | 9.738 | 0.0 | 12.3 | 318 |
| 720 min Winter | 8.424 | 0.0 | 12.7 | 376 |
| 960 min Winter | 6.697 | 0.0 | 13.5 | 492 |
| 1440 min Winter | 4.839 | 0.0 | 14.6 | 734 |
| 2160 min Winter | 3.490 | 0.0 | 15.8 | 1100 |
| 2880 min Winter | 2.766 | 0.0 | 16.7 | 1464 |
| 4320 min Winter | 1.989 | 0.0 | 18.0 | 2204 |
| 5760 min Winter | 1.573 | 0.0 | 19.0 | 2872 |
| 7200 min Winter | 1.311 | 0.0 | 19.8 | 3672 |
| 8640 min Winter | 1.129 | 0.0 | 20.5 | 4248 |
| 10080 min Winter | 0.994 | 0.0 | 21.0 | 5088 |

## Rainfall Details

| | | | |
|---|---|---|---|
| Rainfall Model | FSR | Winter Storms | Yes |
| Return Period (years) | 100 | Cv (Summer) | 0.750 |
| Region | England and Wales | Cv (Winter) | 0.840 |
| M5-60 (mm) | 20.000 | Shortest Storm (mins) | 15 |
| Ratio R | 0.400 | Longest Storm (mins) | 10080 |
| Summer Storms | Yes | Climate Change % | +40 |

## Time Area Diagram

Total Area (ha) 0.015

| **Time From:** | **(mins) To:** | **Area (ha)** |
|---|---|---|
| 0 | 4 | 0.015 |

## Time Area Diagram

Total Area (ha) 0.000

| **Time From:** | **(mins) To:** | **Area (ha)** |
|---|---|---|
| 0 | 4 | 0.000 |

## Model Details

Storage is Online Cover Level (m) 12.300

## Cellular Storage Structure

Invert Level (m) 11.600 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

| Depth (m) | Area ($m^2$) | Inf. Area ($m^2$) | Depth (m) | Area ($m^2$) | Inf. Area ($m^2$) |
|---|---|---|---|---|---|
| 0.000 | 15.0 | 0.0 | 0.401 | 0.0 | 0.0 |
| 0.400 | 15.0 | 0.0 | | | |

## Hydro-Brake® Optimum Outflow Control

| | |
|---|---|
| Unit Reference | MD-SHE-0055-1000-0400-1000 |
| Design Head (m) | 0.400 |
| Design Flow (l/s) | 1.0 |
| Flush-Flo™ | Calculated |
| Objective | Minimise upstream storage |
| Application | Surface |
| Sump Available | Yes |
| Diameter (mm) | 55 |
| Invert Level (m) | 11.600 |
| Minimum Outlet Pipe Diameter (mm) | 75 |
| Suggested Manhole Diameter (mm) | 1200 |

| Control Points | Head (m) | Flow (l/s) |
|---|---|---|
| Design Point (Calculated) | 0.400 | 1.0 |
| Flush-Flo™ | 0.117 | 1.0 |
| Kick-Flo® | 0.273 | 0.8 |
| Mean Flow over Head Range | - | 0.9 |

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

| Depth (m) | Flow (l/s) | Depth (m) | Flow (l/s) | Depth (m) | Flow (l/s) | Depth (m) | Flow (l/s) |
|---|---|---|---|---|---|---|---|
| 0.100 | 1.0 | 1.200 | 1.6 | 3.000 | 2.5 | 7.000 | 3.8 |
| 0.200 | 1.0 | 1.400 | 1.8 | 3.500 | 2.7 | 7.500 | 3.9 |
| 0.300 | 0.9 | 1.600 | 1.9 | 4.000 | 2.8 | 8.000 | 4.0 |
| 0.400 | 1.0 | 1.800 | 2.0 | 4.500 | 3.0 | 8.500 | 4.1 |
| 0.500 | 1.1 | 2.000 | 2.1 | 5.000 | 3.2 | 9.000 | 4.3 |
| 0.600 | 1.2 | 2.200 | 2.2 | 5.500 | 3.3 | 9.500 | 4.4 |
| 0.800 | 1.4 | 2.400 | 2.2 | 6.000 | 3.5 | | |
| 1.000 | 1.5 | 2.600 | 2.3 | 6.500 | 3.6 | | |

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

Half Drain Time exceeds 7 days.

| Storm Event | Max Level (m) | Max Depth (m) | Max Infiltration (l/s) | Max Volume (m³) | Status |
|---|---|---|---|---|---|
| 15 min Summer | 12.225 | 0.075 | 0.0 | 2.7 | O K |
| 30 min Summer | 12.254 | 0.104 | 0.0 | 3.8 | O K |
| 60 min Summer | 12.284 | 0.134 | 0.0 | 4.9 | O K |
| 120 min Summer | 12.314 | 0.164 | 0.0 | 6.0 | O K |
| 180 min Summer | 12.332 | 0.182 | 0.0 | 6.7 | O K |
| 240 min Summer | 12.344 | 0.194 | 0.0 | 7.1 | O K |
| 360 min Summer | 12.362 | 0.212 | 0.0 | 7.7 | O K |
| 480 min Summer | 12.374 | 0.224 | 0.0 | 8.2 | O K |
| 600 min Summer | 12.385 | 0.235 | 0.0 | 8.6 | O K |
| 720 min Summer | 12.393 | 0.243 | 0.0 | 8.9 | O K |
| 960 min Summer | 12.406 | 0.256 | 0.0 | 9.4 | O K |
| 1440 min Summer | 12.423 | 0.273 | 0.0 | 10.0 | O K |
| 2160 min Summer | 12.438 | 0.288 | 0.0 | 10.5 | O K |
| 2880 min Summer | 12.447 | 0.297 | 0.0 | 10.9 | O K |
| 4320 min Summer | 12.455 | 0.305 | 0.0 | 11.1 | O K |
| 5760 min Summer | 12.455 | 0.305 | 0.0 | 11.2 | O K |
| 7200 min Summer | 12.452 | 0.302 | 0.0 | 11.1 | O K |
| 8640 min Summer | 12.447 | 0.297 | 0.0 | 10.9 | O K |
| 10080 min Summer | 12.441 | 0.291 | 0.0 | 10.7 | O K |
| 15 min Winter | 12.236 | 0.086 | 0.0 | 3.2 | O K |

| Storm Event | Rain (mm/hr) | Flooded Volume (m³) | Time-Peak (mins) |
|---|---|---|---|
| 15 min Summer | 138.153 | 0.0 | 19 |
| 30 min Summer | 90.705 | 0.0 | 34 |
| 60 min Summer | 56.713 | 0.0 | 64 |
| 120 min Summer | 34.246 | 0.0 | 124 |
| 180 min Summer | 25.149 | 0.0 | 184 |
| 240 min Summer | 20.078 | 0.0 | 244 |
| 360 min Summer | 14.585 | 0.0 | 364 |
| 480 min Summer | 11.622 | 0.0 | 484 |
| 600 min Summer | 9.738 | 0.0 | 604 |
| 720 min Summer | 8.424 | 0.0 | 724 |
| 960 min Summer | 6.697 | 0.0 | 964 |
| 1440 min Summer | 4.839 | 0.0 | 1444 |
| 2160 min Summer | 3.490 | 0.0 | 2164 |
| 2880 min Summer | 2.766 | 0.0 | 2884 |
| 4320 min Summer | 1.989 | 0.0 | 4324 |
| 5760 min Summer | 1.573 | 0.0 | 5760 |
| 7200 min Summer | 1.311 | 0.0 | 7200 |
| 8640 min Summer | 1.129 | 0.0 | 8376 |
| 10080 min Summer | 0.994 | 0.0 | 8776 |
| 15 min Winter | 138.153 | 0.0 | 19 |

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

| Storm Event | Max Level (m) | Max Depth (m) | Max Infiltration (l/s) | Max Volume (m³) | Status |
|---|---|---|---|---|---|
| 30 min Winter | 12.268 | 0.118 | 0.0 | 4.3 | O K |
| 60 min Winter | 12.302 | 0.152 | 0.0 | 5.6 | O K |
| 120 min Winter | 12.336 | 0.186 | 0.0 | 6.8 | O K |
| 180 min Winter | 12.356 | 0.206 | 0.0 | 7.5 | O K |
| 240 min Winter | 12.370 | 0.220 | 0.0 | 8.0 | O K |
| 360 min Winter | 12.389 | 0.239 | 0.0 | 8.8 | O K |
| 480 min Winter | 12.404 | 0.254 | 0.0 | 9.3 | O K |
| 600 min Winter | 12.416 | 0.266 | 0.0 | 9.7 | O K |
| 720 min Winter | 12.425 | 0.275 | 0.0 | 10.1 | O K |
| 960 min Winter | 12.440 | 0.290 | 0.0 | 10.6 | O K |
| 1440 min Winter | 12.460 | 0.310 | 0.0 | 11.3 | O K |
| 2160 min Winter | 12.478 | 0.328 | 0.0 | 12.0 | O K |
| 2880 min Winter | 12.489 | 0.339 | 0.0 | 12.4 | O K |
| 4320 min Winter | 12.500 | 0.350 | 0.0 | 12.8 | O K |
| 5760 min Winter | 12.503 | 0.353 | 0.0 | 12.9 | O K |
| 7200 min Winter | 12.502 | 0.352 | 0.0 | 12.9 | O K |
| 8640 min Winter | 12.498 | 0.348 | 0.0 | 12.8 | O K |
| 10080 min Winter | 12.493 | 0.343 | 0.0 | 12.6 | O K |

| Storm Event | Rain (mm/hr) | Flooded Volume (m³) | Time-Peak (mins) |
|---|---|---|---|
| 30 min Winter | 90.705 | 0.0 | 34 |
| 60 min Winter | 56.713 | 0.0 | 64 |
| 120 min Winter | 34.246 | 0.0 | 124 |
| 180 min Winter | 25.149 | 0.0 | 182 |
| 240 min Winter | 20.078 | 0.0 | 242 |
| 360 min Winter | 14.585 | 0.0 | 362 |
| 480 min Winter | 11.622 | 0.0 | 480 |
| 600 min Winter | 9.738 | 0.0 | 598 |
| 720 min Winter | 8.424 | 0.0 | 718 |
| 960 min Winter | 6.697 | 0.0 | 954 |
| 1440 min Winter | 4.839 | 0.0 | 1428 |
| 2160 min Winter | 3.490 | 0.0 | 2140 |
| 2880 min Winter | 2.766 | 0.0 | 2848 |
| 4320 min Winter | 1.989 | 0.0 | 4236 |
| 5760 min Winter | 1.573 | 0.0 | 5600 |
| 7200 min Winter | 1.311 | 0.0 | 6984 |
| 8640 min Winter | 1.129 | 0.0 | 8296 |
| 10080 min Winter | 0.994 | 0.0 | 9576 |

## Rainfall Details

| | | | |
|---|---|---|---|
| Rainfall Model | FSR | Winter Storms | Yes |
| Return Period (years) | 100 | Cv (Summer) | 0.750 |
| Region | England and Wales | Cv (Winter) | 0.840 |
| M5-60 (mm) | 20.000 | Shortest Storm (mins) | 15 |
| Ratio R | 0.400 | Longest Storm (mins) | 10080 |
| Summer Storms | Yes | Climate Change % | +40 |

## Time Area Diagram

Total Area (ha) 0.013

| **Time From:** | **(mins) To:** | **Area (ha)** |
|---|---|---|
| 0 | 4 | 0.013 |

## Time Area Diagram

Total Area (ha) 0.000

| **Time From:** | **(mins) To:** | **Area (ha)** |
|---|---|---|
| 0 | 4 | 0.000 |

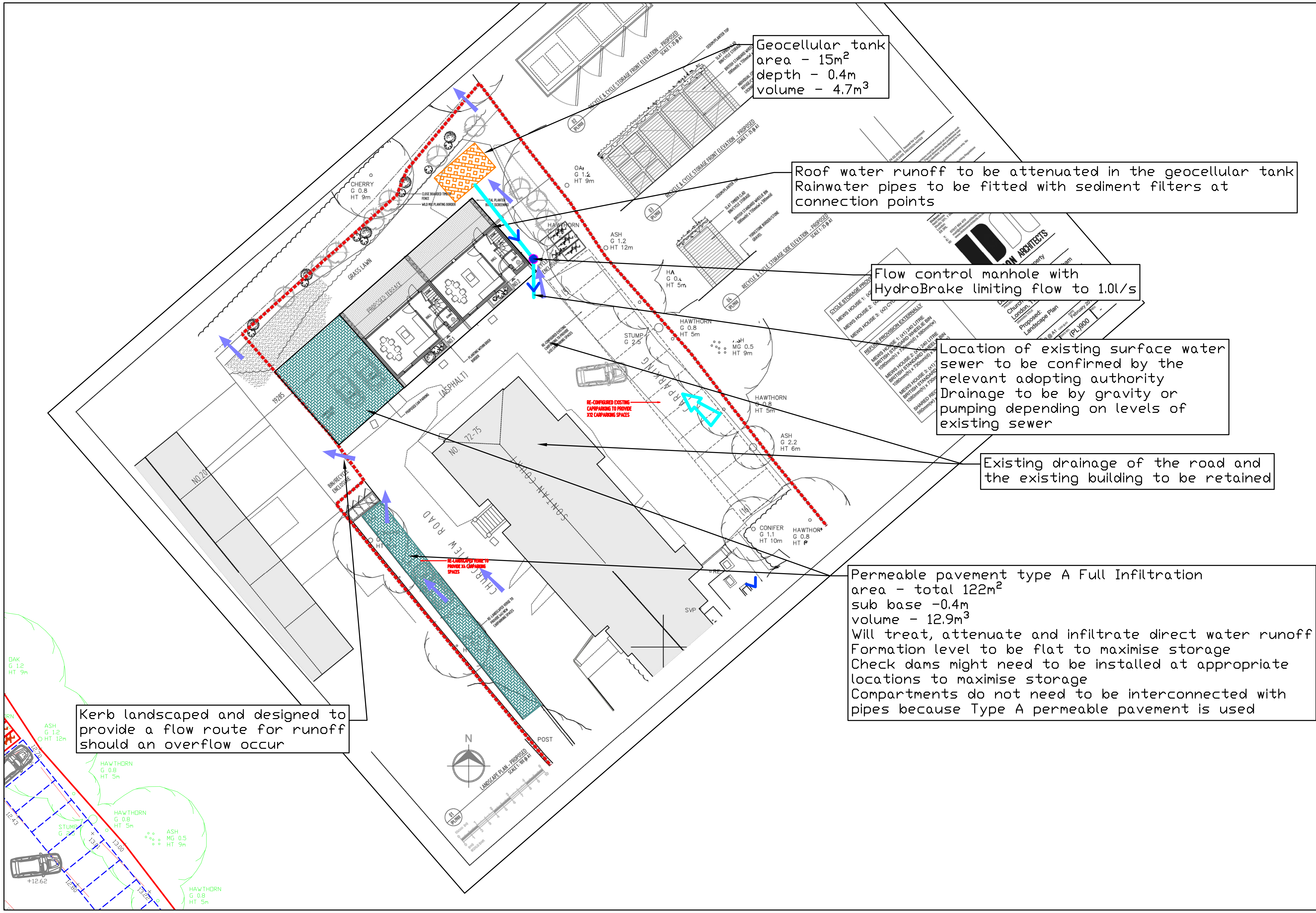
## Model Details

Storage is Online Cover Level (m) 12.700

## Porous Car Park Structure

| | | | |
|---|---|---|---|
| Infiltration Coefficient Base (m/hr) | 0.00036 | Width (m) | 10.0 |
| Membrane Percolation (mm/hr) | 1000 | Length (m) | 12.2 |
| Max Percolation (l/s) | 33.9 | Slope (1:X) | 0.0 |
| Safety Factor | 2.0 | Depression Storage (mm) | 5 |
| Porosity | 0.30 | Evaporation (mm/day) | 3 |
| Invert Level (m) | 12.150 | Membrane Depth (m) | 150 |

# Appendix D – Proposed Drainage Strategy




**Proposed Schedule of Maintenance for Below Ground Drainage**

| Item | Visual Inspection | Cleanse / De-sludge | CCTV Survey | Comments |
|---|---|---|---|---|
| Foul Drainage System (pipework, chambers etc.) | 5 years | 10 years | 10 years | Cleansing to be carried as necessary |
| Surface Water Drainage System (pipework, chambers etc.) | 5 years | 10 years | 10 years | Cleansing to be carried as necessary |
| Gullies/Channels | 1 year | 1 year | N/A | Cleansing to be carried as necessary |
| Petrol interceptor: Chamber & Alarm | 1 year | 1 year | N/A | Maintenance in accordance with manufacturer's instructions |
| Soakaways and catchpits | 1 year | | N/A | Cleansing to be carried as necessary |
| Permeable Tarmac Paving | 1 year | 'Swept' clean of debris every 2 years. | N/A | Jetwash or suction roadsweep permeable tarmac as performance levels reduce. |
| Permeable Paving | 1 year | 'Swept' clean of debris every 2 years. | N/A | Lift blocks and remove sand bedding and replace and re-bed paving – refer to individual manufacturers recommendations. |

**Project**
SURFACE WATER DRAINAGE STRATEGY
CHURCHVIEW ROAD - CONTRACT 3193

**Drawing**
Preliminary Surface Water Drainage Strategy Layout

| | | | |
|---|---|---|---|
| Drawn by: | Bojidar Boiadjiev | Date: | 23/05/2017 |
| Checked by: | Mark Naumann | Date: | 23/05/2017 |
| Approved by: | Mark Naumann | Date: | 23/05/2017 |
| Amended by: | Bojidar Boiadjiev | Date: | 18/02/2019 |

**Drawing No.** Lower Level — **Revision** A

**Drawing Scale:** NOT TO SCALE

**Client**
Metropolis Planning and Design


**Ambiental Technical Solutions**
The Sussex Innovation Centre
Science Park Square,
Falmer, Brighton
BN1 9SB
Tel +44 (0) 01273 704441
www.ambiental.co.uk
drainage@ambiental.co.uk


| Rev | By | Chkd | Apprvd | Date | Description |
|---|---|---|---|---|---|
| | | | | | |
| | | | | | |

**Legend**
- Site Boundary
- Private Drain (Not Adoptable)
- Pattern Hatch Indicative of SuDS Permeable Pavement Type A
- Pattern Hatch Indicative of SuDS Geocellular Tank
- Type 2 Manhole
- Design Flow Exceedance Flow Route

1. GENERAL
(i) THIS DRAWING IS NOT TO BE SCALED, WORK TO FIGURED DIMENSIONS ONLY, CONFIRMED ON SITE.
(ii) THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTURAL DRAWINGS, DETAILED SPECIFICATIONS WHERE APPLICABLE AND ALL ASSOCIATED DRAWINGS IN THIS SERIES.
(iii) ANY DISCREPANCY ON THIS DRAWING IS TO BE REPORTED IMMEDIATELY TO THE PARTNERSHIP FOR CLARIFICATION.
(iv) THE CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY WORKS AND FOR THE STABILITY OF THE WORKS IN PROGRESS.

PRELIMINARY DRAWING
DRAWING FOR INFORMATION ONLY.
NOT FOR CONSTRUCTION.

2. BELOW GROUND DRAINAGE – GENERAL INFORMATION
(i) PIPEWORK TO BE UPVC-U PIPES TO BS 4660 : 2000 AND INSPECTION CHAMBERS TO BS 7158 : 2001.
(ii) ALL ADOPTABLE DRAINAGE TO BE CONSTRUCTED IN ACCORDANCE WITH 'SEWERS FOR ADOPTION' 7TH EDITION AND THE RELEVANT COUNCIL DESIGN GUIDE.
(iii) ALL PRIVATE SURFACE WATER SEWERS TO BE LAID AT 1 IN 80 UNLESS OTHERWISE STATED ON THE DRAWING.
(iv) ALL PRIVATE FOUL WATER SEWERS TO BE LAID AT 1 IN 40 AT THE HEAD OF PIPE RUNS AND 1 IN 80 ELSEWHERE UNLESS OTHERWISE STATED.
(v) ALL PRIVATE FOUL SEWER PIPES TO BE 100MM DIAMETER UNLESS OTHERWISE STATED ON THE DRAWING. ALL PRIVATE SURFACE WATER SEWER PIPES TO BE 100MM DIAMETER FROM DOWNPIPES AND 150MM DIAMETER ELSEWHERE UNLESS OTHERWISE STATED ON THE DRAWING.
(w) ALLOW FOR RODDING ACCESS ABOVE GROUND WHERE RAINWATER DOWNPIPES DO NOT HAVE A DIRECT CONNECTION TO AN INSPECTION CHAMBER.
(vii) EXISTING SEWER PIPE TO BE RE-USED TO BE SURVEYED AND LEVELLED PRIOR TO COMMENCEMENT OF THE DRAINAGE WORKS AND REFURBISHED IF NECESSARY.
(viii) CONNECTIONS TO AN ADOPTED SEWER ONLY TO BE MADE FOLLOWING APPROVAL FROM THE RELEVANT ADOPTING AUTHORITY.
(ix) ALL DRAINS, SEWER PIPES AND MANHOLES TO BE CLEANED AND TESTED FOR WATER TIGHTNESS ON COMPLETION OF CONSTRUCTION.

# Appendix E – Information

## Surface Water Runoff Calculation Method

Rainfall data has been extracted from the FEH CD-ROM for several storm duration events for a number of return periods, including 1:1.01 year, 1:10 year and 1:100 year storm events. These return periods are industry standard, however it is important to be aware that return periods less than 1:2 years are not considered reliable and should not be used in detailed design calculations.

The 1:100 year with an allowance for climate change has been based on a 40% increase to the 1:100 year rainfall intensity and not the rainfall depth. This is to provide the most conservative runoff rates for the site possible.

Greenfield runoff rates have been calculated using The Institute of Hydrology Report 124 Marshall and Bayliss, 1994 method, as recommended in the SuDS Manual CIRIA (C753). In keeping with standard practice, the calculations are based on calculating the Greenfield runoff rates for a 50 Ha site and then factored to account for the actual site size.

Impermeable runoff rates have been calculated using the Modified Rational Method for the impermeable surfaces on site only.

These runoff rates have then been combined to provide the most accurate runoff rate possible for both the existing and proposed site.