

Flood Risk Assessment and Drainage Strategy

Harefield Hospital, Hill End Road Harefield UB9 6JH

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Prepared For:



Guy's and St Thomas'
NHS Foundation Trust



FLOOD RISK ASSESSMENT AND DRAINAGE STRATEGY

HAREFIELD HOSPITAL, HILL END ROAD HAREFIELD UB9 6JH

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Executive Summary

Meinhardt have undertaken this Flood Risk Assessment (FRA) and prepared this Drainage Strategy on behalf of Royal Brompton & Harefield Hospital which is part of Guy and St Thomas NHS Foundation Trust to support the planning application for the replacement of the existing generators at Harefield Hospital, Hill End Road, Harefield, UB9 6JH, within the London Borough of Hillingdon. The Site comprises an area of 0.031 hectares (ha) and is located on a pocket of land within the wider Harefield Hospital site within the Royal Borough of Hillingdon. The OS Grid Reference for the approximate centre of the Site is National Grid Reference 505167, 190856.

The proposed development includes the removal of existing generators serving Substations 2 and 3, and installation of new replacement generators with associated works.

This FRA investigates flood risk on site and in the area and outlines the mitigation measures proposed to ensure the sustainable and safe development of the Site in line with the requirements of the National Planning Policy Framework (NPPF), Environment Agency's (EA) Climate Change allowances guidance and Standing Advice, and the Flood Risk and Coastal Change Planning Practice Guidance (PPG).

The Site is located in Flood Zone 1 where the probability of river or sea flooding is less than 0.1% (1 in 1000) chance in any given year. Therefore, the probability of tidal or fluvial flooding is assessed as negligible.

Available baseline information indicates that within the Site there are isolated spots at low risk of surface water flooding. However, a surface water drainage strategy has been designed in line with the most recent EA Climate Change Guidance to consider and manage the impact of a 1:100 year plus climate change rainfall event and is presented within this report. The surface water drainage strategy considers surface water runoff management: the solutions proposed ensure that for the 100 year plus 40% climate change allowance event, surface water will be accommodated within the Site and therefore prevent potential exceedance flows off-site.

The Site's surface water will be managed in approximately 30% porous infiltration trenches located beneath and around the proposed equipment in line with London Plan Drainage hierarchy requirements subject to Local Planning Authority approval.

The potential for implementing Sustainable Drainage Systems (SuDS) has been evaluated for the site. As previously noted, the proposed development involves replacing existing generators and installing new units within Glass Reinforced Plastic (GRP) enclosures. This configuration significantly limits the effective integration of green infrastructure elements, such as blue roofs and rainwater harvesting systems, within the GRP enclosures. Given the characteristics of the development, oil spills are possible during the refilling of the generators. Mitigation measures as proprietary treatment systems, bunding and spill kits will be considered as the design develops.

Other potential sources of flooding have been investigated and have been deemed negligible.

1 Introduction

1.1 Project Brief

Meinhardt have been appointed by Guy's and St Thomas NHS Foundation Trust to undertake this Flood Risk Assessment (FRA) and Drainage Strategy to accompany the planning application for the replacement of existing generators at the Harefield Hospital site (hereafter referred to as the 'Proposed Development') located at Harefield Hospital Hill End Road, Harefield UB9 6JH, within the London Borough of Hillingdon (hereafter referred to as the 'Site'). This report relates solely to the above Site and should not be used for any other reasons other than for which the report was originally prepared.

1.2 Assessment Methodology

The below methodology has been carried out to investigate flood risk on Site and in the surrounding area and to identify suitable mitigation measures required to ensure the sustainability and safety of the Proposed Development over its lifetime.

The report has been produced in line with the requirements of the NPPF and the Environment Agency (EA) Standing Advice, as well as the requirements of London Borough of Hillingdon as the Lead Local Flood Authority (LLFA).

The following documents and policies have been reviewed to inform this report:

- LLFA – Strategic Flood Risk Assessment (SFRA) (2015)¹;
- LLFA – Preliminary Flood Risk Assessment (PFRA) (2011)²;
- LLFA – Surface Water Management Plan (SWMP) (2013)³;
- LLFA – Local Flood Risk Management Strategy (LFRMS) (2015)⁴;
- LLFA – Local Plan (2012)⁵;
- LLFA – SuDS Design & Evaluation Guide (2018)⁶;
- Greater London Authority – The London Plan, (2021)⁷;
- EDI Surveys Ltd – Topographical Survey (2009);
- British Geological Society (BGS) Online Viewer, 1:50,000 Bedrock and Superficial deposits⁸;
- DEFRA's online Magic Map⁹; and
- GOV.UK's online Long-Term Flood Maps¹⁰.

The local policies discussed in Section 2 have also been considered to inform this assessment.

Meinhardt does not guarantee that the advice provided by this report will guarantee the availability of flood insurance. The insurance industry carries out their own assessments to properties in terms of determining the insurability for flood risk. Those intending to undertake development in areas which may be at risk of flooding are advised to contact their insurers or the Association of British Insurers (ABI) to seek further guidance prior to commencement.

¹ London Borough of Hillingdon (2015). London Borough of Hillingdon Strategic flood risk assessment.

² London Borough of Hillingdon (2011). London Borough of Hillingdon Preliminary flood risk assessment.

³ London Borough of Hillingdon (2013). London Borough of Hillingdon Surface water Management Plan.

⁴ London Borough of Hillingdon (2015). London Borough of Hillingdon Local Flood Risk Management Strategy.

⁵ London Borough of Hillingdon (2012). London Borough of Hillingdon Local Flood Risk Management Strategy

⁶ London Borough of Hillingdon SuDS D&E Guide (2018) Scans.bgs.ac.uk. 2024. Page 1-150 [online] Available at: <<https://online.flipbuilder.com/mccloy.consulting/ztpy/mobile/index.html#p=2>> [Accessed 28 August 2024].

⁷ Greater London Authority (2021).

⁸ Mapapps.bgs.ac.uk. 2024. Geology of Britain viewer | British Geological Survey (BGS). [online] Available at: [Accessed 28 August 2024].

⁹ Magic.defra.gov.uk. 2024. Magic Map Application. [online] Available at: [Accessed 28 August 2024].

¹⁰ GOV.UK. 2022. Check the long-term flood risk for an area in England. [online] Available at: [Accessed 02 August 2024].

Although this report was prepared using the degree of skill and care ordinarily exercised by engineers practicing under similar circumstances, please note that Meinhardt cannot take responsibility for errors in the information provided by third parties.

2 Planning Policy and Guidance

The below policy and guidance are considered relevant to this FRA and Drainage Strategy and have been used to inform this assessment.

2.1 National and Regional Policy

National Planning Policy Framework

The National Planning Policy Framework (NPPF) was updated and published in December 2024 with the aim of protecting the environment and to promote sustainable growth. There is an overarching objective to strive towards sustainable development and ensuring that this is a key element in every proposal.

The following paragraphs from the NPPF are considered relevant to this assessment:

Paragraph 170: Requires that “Inappropriate development in areas at risk of flooding should be avoided by directing development away from areas at highest risk (whether existing or future). Where development is necessary in such areas, the development should be made safe for its lifetime without increasing flood risk elsewhere”;

Paragraph 181: Explains that “When determining any planning applications, local planning authorities should ensure that flood risk is not increased elsewhere. Where appropriate, applications should be supported by a site-specific flood-risk-assessment.”

Paragraph 182: Recommends that “Applications which could affect drainage on or around the site should incorporate sustainable drainage systems to control flow rates and reduce volumes of runoff, and which are proportionate to the nature and scale of the proposal. These should provide multifunctional benefits wherever possible, through facilitating improvements in water quality and biodiversity, as well as benefits for amenity. Sustainable drainage systems provided as part of proposals for major development should:

- a) take account of advice from the lead local flood authority;
- b) have appropriate proposed minimum operational standards;
- c) have maintenance arrangements in place to ensure an acceptable standard of operation for the lifetime of the development;

The London Plan

The London Plan: Spatial Development Strategy for Greater London (March 2021):

The London Plan sets out the new spatial development strategy for Greater London and it sets out an integrated economic, environmental, transport and social framework for the development of London over the next 20-25 years. The following policies are considered relevant to this assessment:

Policy SI 5 Water Infrastructure - “Development proposals should seek to improve the water environment and ensure that adequate water infrastructure capacity is provided. Development plans should promote the protection and improvement of the water environment in line with the Thames River Basin Management Plan and should take account of Catchment Plans whilst supporting wastewater treatment infrastructure investment to accommodate London’s growth and climate change impacts. Development proposals should minimise the use of mains incorporating measures such as smart metering, water saving and recycling measures, including retrofitting, to help to achieve a maximum mains consumption of 105 litres or less per head per day”

Policy SI 12 Flood Risk Management - "Development proposals must comply with the flood risk assessment and management requirements set out in the NPPF and the associated technical Guidance on flood risk"; and

Policy SI 13 Sustainable Drainage - "Development should utilise sustainable urban drainage systems (SUDS) unless there are practical reasons for not doing so, and should aim to achieve greenfield run-off rates whilst ensuring that surface water run-off is managed as close to its source as possible in line with the below drainage hierarchy:

- 1) rainwater use as a resource (for example rainwater harvesting, blue roofs for irrigation)
- 2) rainwater infiltration to ground at or close to source
- 3) rainwater attenuation in green infrastructure features for gradual release (for example green roofs, rain gardens)
- 4) rainwater discharge direct to a watercourse (unless not appropriate)
- 5) controlled rainwater discharge to a surface water sewer or drain
- 6) controlled rainwater discharge to a combined sewer".

Development proposals for impermeable paving should be refused where deemed inappropriate such as in small areas of front gardens and driveways.

Drainage should be designed and implemented to address water use efficiency, water quality, biodiversity, amenity and recreation.

London Borough of Hillingdon Local Plan.

London Borough of Hillingdon Local Plan (2012) Part 1 Strategic Policies:

"The two sections of the Local Plan form the council's future development strategy for the borough. It sets out a framework and detailed policies to guide planning decisions and it's the starting point for considering whether planning applications should be approved."

The following policies are considered relevant to this assessment:

Policy EM6 Flood Risk Management – "The Council will require new development to be directed away from Flood Zones 2 and 3 in accordance with the principles of the National Planning Policy Framework (NPPF). The subsequent Hillingdon Local Plan: Part 2 -Site Specific Allocations LDD will be subjected to the Sequential Test in accordance with the NPPF. Sites will only be allocated within Flood Zones 2 or 3 where there are overriding issues that outweigh flood risk. In these instances, policy criteria will be set requiring future applicants of these sites to demonstrate that flood risk can be suitably mitigated. The Council will require all development across the borough to use sustainable urban drainage systems (SUDS) unless demonstrated that it is not viable. The Council will encourage SUDS to be linked to water efficiency methods. The Council may require developer contributions to guarantee the long-term maintenance and performance of SUDS is to an appropriate standard."

Policy EM7 Biodiversity and Geological Conservation item 7 - "Hillingdon's biodiversity and geological conservation will be preserved and enhanced with particular attention given to: The use of sustainable drainage systems that promote ecological connectivity and natural habitats."

London Borough of Hillingdon Local Flood Risk Management Strategy (2015):

The Local Flood Risk Management Strategy "provides an overview of the assessment of flood risks already undertaken in other documents and sets out a framework for the management of local flood risk in the London Borough of Hillingdon, by the Council and other relevant bodies. This strategy is supported by the already published Flood Risk Management Portfolio of documentation which contains other critical legal documents such as the Preliminary Flood Risk Assessment, Strategic Flood Risk Assessment and Surface

Water Management Plan Parts 1 and 2. These provide the evidence base for this strategy with more detailed information on the flood risks in Hillingdon and actions.”

London Borough of Hillingdon Surface Water Management Plan Parts 1 and 2 (2013):

“The Surface Water Management Plan (SWMP) is produced by the Council as the Lead Local Flood Authority (LLFA). Its purpose is to set out the Council’s surface water management strategy, identifying options and actions for managing flood risk. In this context surface water flooding comprises flooding from sewers, drains and groundwater, together with runoff from land, small water courses and ditches occurring as a result of heavy rainfall.”

London Borough of Hillingdon Strategic Flood Risk Assessment (2015):

This document Initially prepared in 2008, and updated in 2015, “collated all known evidence of flood risk and forms part of the evidence base for the vision and objectives of the Local Plan. It provides an understanding of all flood risks at that point in time. There may be additional information on flood risk collected subsequently.”

London Borough of Hillingdon Preliminary Flood Risk Assessment (2011):

The preliminary flood risk assessment is “aimed at providing a high-level overview of flood risk from all sources within the Borough, including consideration of surface water, groundwater, ordinary watercourses and canals. The PFRA includes a: summary of information on significant historic floods; summary of information on future flood risks based primarily on the Environment Agency’s national datasets; as well as a spreadsheet containing information for reporting to the European Commission.”

London Borough of Hillingdon SuDS Design & Evaluation Guide (2018):

The London Borough of Hillingdon SuDS Design & Evaluation Guide (2018) “promotes the idea of integrating SuDS into the fabric of development using the available landscape spaces as well as the construction profile of buildings.” It is intended for designers such that “SuDS designers will need to meet its requirements when submitting proposals to the Local planning authority (LPA).”

2.2 Climate Change Guidance

The EA guidance “Flood risk assessments: climate change allowances” was issued in 2016 (last updated 27th May 2022) and provides the most recent information on expected changes in rainfall, river flows and sea levels as a consequence of climate change.

Sea level allowances by river basin district for each epoch is provided in Table 2 of the guidance document.

The guidance provides peak river flow allowances by management catchments, the sub-catchments of the river basin districts. The peak flows are based on percentiles, a percentile is a measure used to describe the proportion of possible scenarios that fall below an allowance level (e.g. a 50% percentile means that the allowance has 50% chances of not being exceeded).

The most recent update to the guidance, updates the Peak Rainfall Allowances, which takes into account UKCP Local 2.2km projections and supporting research FUTURE-DRAINAGE. These are also now provided by management catchments instead of at a national scale and an online “Peak Rainfall Allowances Map” is now provided. Peak rainfall allowances are provided for 1% annual exceedance probability (AEP) events and for 3.3% AEP events.

The guidance states how to apply peak rainfall allowances, using the central allowance for development with a lifetime up to 2100 and the upper end allowance for development with a lifetime from 2100 to 2125. Residential development is to have a minimum lifetime of a 100 years, whilst commercial a minimum of 60 years.

The following approach should be used for drainage design:

- Development with a lifetime beyond 2100 - Design your development so that for the upper end allowance in the 1% annual exceedance probability event:
 - There is no increase in flood risk elsewhere
 - Your development will be safe from surface water flooding
- Development with a lifetime of between 2061 and 2100 - For development with a lifetime between 2061 and 2100 take the same approach but use the central allowance for the 2070s epoch (2061 to 2125).
- Development with a lifetime up to 2060 - For development with a lifetime up to 2060, take the same approach but use the central allowance for the 2050s epoch (2022 to 2060).

Exceptions:

In some locations the allowance for the 2050s epoch is higher than that for the 2070s epoch. If so, and development has a lifetime beyond 2061, use the higher of the two allowances.

3 Existing Site

3.1 Site Location and Description

The Site is located within Harefield Hospital, in the London Borough of Hillingdon. The site for the proposed generator installation is part of the wider Harefield hospital site. The Hospital site premises are bound by Rickmansworth Road to the east and Park Lane to the south. The OS Grid Reference for the approximate centre of the Site is National Grid Reference 505177, 190833.

The Site for generator installation consists of three distinct sections, totalling an approximate area of 0.031 hectares (ha). Presently, the Site features existing generators mounted on concrete foundations, along with open areas and trees that will require removal. The concrete foundations will need to be enlarged to accommodate the installation of new generators, bulk fuel tanks, and generator control units housed within GPR enclosures. These generators are designated to support Substations, 2, and 3, as illustrated in the figure below. For a visual representation of the Site's location, please refer to **Figure 1**.



Figure 1: Indicative Site Location Plan.

3.2 Topography

Based on the topographical and utilities survey that was carried out by EDI Surveys in December 2009 the levels vary across the site. Levels for the overall hospital site generally fall from Northeast to Southwest, with levels ranging between 89.99AOD and 81.71 AOD. However levels are generally flat around the the area for the proposed location of the 2 x generators for substation 2 and 3. Refer to **Appendix A** for the Topographical and Utilities Survey.

3.3 Existing Drainage

A Topographic and utilities survey was carried out by EDI surveys in December 2009, refer to **Appendix A**. The sites for generator installations are part of the wider hospital site with existing drainage services. The survey shows that the existing site is served by a network of private foul and surface water drainage which convey foul and surface water to various discharge locations surrounding the site. At the time of writing this report, there were no records from Thames Water available indicating the extent of public sewers around the site. The available documentation has been reviewed and the existing utilities on the site are summarised below:

3.3.1 Public Sewers

- There were no Thames Water sewer records at the time of writing this report to confirm the existence of public sewer close to the site.

3.3.2 Private On Site Drainage

Site Earmarked for the installation of 2 x new generators, 1 x bulk fuel tank, and 1 x generator control panels for substation 2 and 3:

- The Utilities survey shows that there is a 100mmØ surface water sewer to the west of the site running from Northeast to south-west Towards the Heart Science Centre. Given the indicated flow direction, it is presumed that this sewer discharges into the soak-away located in the open area between the Heart Science Centre and the Temporary Wards, specifically to the north of the Heart Science Centre. Additional surveys will be necessary to confirm the precise path of this sewer.
- The Utilities Survey shows that there is a 150mmØ surface water sewer to the south of the site running from southeast to northwest to discharge into a soak-away located in the open area between the Heart Science Center and the Temporary Wards, just to the northern part of the Heart Science Centre. Further surveys will be required to confirm if there are additional sewers discharging elsewhere beyond the soak-away.
- The utilities surveys shows that there is 100mmØ foul water sewer to the west of the site running from northeast to southwest and it appears to connect to the 225mmØ sewer to the south of the site, which appears to be exiting the site presumably to discharge into a public network. However, there were no Thames Water records available for the site at the time of writing this report and additional surveys will be required to confirm if this sewer discharges into a public sewer network.

3.4 Existing Watercourses, Waterbodies and Flood Defences

The River Colne is located approximately 1.1 km to the southwest of the site, while Lynsters lake is located approximately 1.4 km to the west of the site.

There are no known artificial watercourses in the vicinity of the Site.

3.5 Geological and Hydrogeological Setting

Error! Reference source not found. provides a summary of British Geological Survey (BGS) Geological Map 'Geology of Britain' online maps. In addition, a publicly available BGS borehole logs TQ09SW76¹¹ located approximately 300m northwest of the Site, TQ09SE67¹² located approximately 260m north of the site, and

¹¹ Scans.bgs.ac.uk. 2024. Page 2 | Borehole TQ09SE76 | Borehole Logs. [online] Available at: < <https://api.bgs.ac.uk/sobi-scans/v1/borehole/scans/items/577384> > [Accessed 27 August 2024].

¹² Scans.bgs.ac.uk. 2024. Page 11 | Borehole TQ09SE67 | Borehole Logs. [online] Available at: < <https://api.bgs.ac.uk/sobi-scans/v1/borehole/scans/items/577029> > [Accessed 27 August 2024].

TQ09SE65¹³ located approximately 300m southeast of the site were reviewed and the information included in Table 1 below.

Geological Unit/Stratum	Aquifer Designation	Typical Depth to Top Of Stata (m bgl)	Typical Strata Description
Sudbury Formation (Superficial Deposits)	Secondary A Aquifer	4	Sand and gravel, locally with lenses of silt, clay or peat and organic material. Rests unconformably on bedrock geology.
Drift		3.9	Gravel and Red sands
Reading beds		3.0	Yellow clay, blue and brown clay, dry sand with clay and pebbles (filled up again as the water drained away through this bed)
Lambeth formation (Bedrock)	Secondary A Aquifer	39	Variable sequences mainly of clay, some silty or sandy, with some sands and gravels, minor limestones and lignites and occasional sandstone and conglomerate.

Table 1: Geological Distribution

The BGS, 1:50 00 Artificial group mapping do not show made ground presence on the site.

A **Secondary A aquifer** is defined by the EA as “permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers.”

A review of publicly available BGS borehole, TQ09SE65³, located approximately 300m southeast of the Site identified a water strike at 1.4m bgl, however- the well was found to be dry during dry seasons.

Based on the EA’s online Groundwater Vulnerability Zones Map the Site is located within a “Medium to Low Groundwater” Vulnerability Zone (GWVZ).

According to the DEFRA’s online Magic Map for Source Protection Zones, the Site is located within a Source Protection Zone (Zone II – Outer protection Zone).

Evidence of soakaways has been identified across the broader hospital site, as indicated in the Topographic and Utilities survey performed by EDI Survey Ltd (2009), see **Appendix A** and an infiltration test conducted on the site on 07.10.24 established a soil infiltration rate of 0.000094 m/s.

¹³ Scans.bgs.ac.uk. 2024. Page 2 | Borehole TQ09SE65 | Borehole Logs. [online] Available at:< <https://api.bgs.ac.uk/sobi-scans/v1/borehole/scans/items/577382>> [Accessed 27 August 2024].

4 Proposed Development

4.1 Proposed Development Description

The proposed development includes the removal of existing generators serving Substations 2 and 3, and installation of new replacement generators with associated works:

Substation 2 and 3

The proposal is to strip out and remove the redundant existing generators and replace them with two new containerised 1000kVA generators to provide N+1 resilience to substation 2 and 3 in addition to a bulk fuel tank and a switch gear room to comply with HTM 06- 01 requirements.

4.2 Impact of Climate change on the Proposed Development

As the Site is located within Flood Zone 1 where the probability of fluvial and tidal flooding is lower than 0.1% in any year, fluvial and tidal flooding is not expected to become a significant issue when the impact of climate change is taken into account.

The impact of climate change on surface water runoff at the Site in the surrounding areas could present a higher risk of surface water flooding, however, the Drainage Strategy produced as part of this report has been designed to accommodate surface runoff during all events up to and including the 100-year plus a 40% climate change allowance.

Based on the EA's climate change allowance guidance, the attenuation requirements for the proposed drainage strategy will be determined using the central estimates but will be modelled with a freeboard that can accommodate the upper end estimates to prevent potential exceedance flows off-site.

5 Flood Risk

5.1 Relevant Maps and Data

Online UK Government Flood Maps have been used to identify flood risk from fluvial, tidal, surface water, and artificial sources.

The LLFA's SFRA, SWMP, LFRMS and the PFRA have also been reviewed and relevant maps are included in **Appendix B**.

5.2 Tidal and Fluvial Flooding

Based on the online GOV.UK Flood Map for Planning, the Site is located in Flood Zone 1 where the probability of river or sea flooding is less than 0.1% (1 in 1000) chance in any given year. The closest water body is Rive Colne located approximately 1.1km to the west of the site. As depicted in **Figure 2**, the site is in an area at negligible risk to tidal and fluvial flooding.

Additionally, the SWMP Phase 2 Risk assessment identified 17 Critical Drainage Areas (CDAs) as depicted in **Appendix B** (Figure i in the Surface water Management Plan Part 1- Evidence base). One of the defining parameters for CDA classification particularly related to Tidal and fluvial water flooding is as follows:

"An area located in River Valleys (current and historical) - Across the study area, the areas particularly susceptible to overland flow are formed by narrow corridors associated with topographical valleys which represent the routes of the 'lost' watercourses within London".

As depicted in **Appendix B** the site is in Harefield, is not part of a critical drainage area within the London Borough of Hillingdon and therefore is not at risk of fluvial flooding.

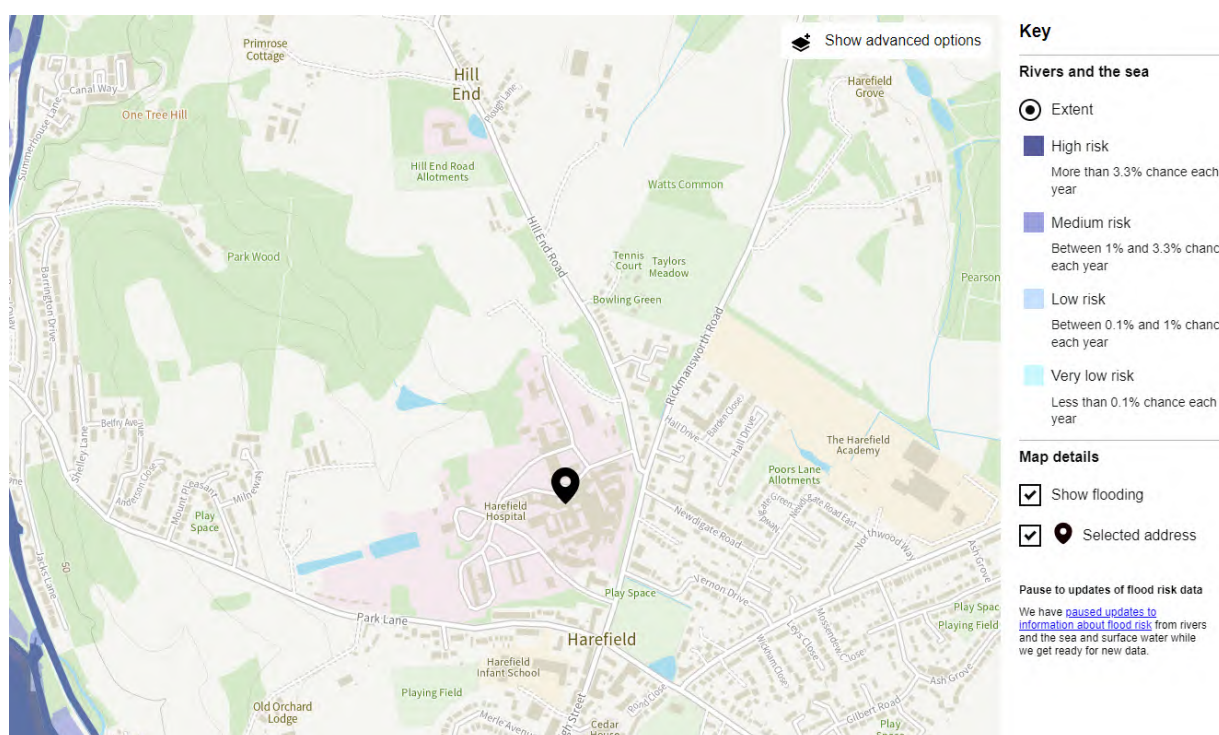


Figure 2: Extract from GOV.UK's Long Term Flood Risk Map for Tidal and fluvial Flooding.

Based on the information available, the probability of fluvial and tidal flooding can be assessed as **negligible**.

5.3 Surface Water Flooding

The GOV.UK Long Term Flood Risk Maps provide a general indication of areas that may be at risk of surface water flooding. The maps take a broad account of underground drainage and typical storms which are likely to cause flooding. The map identifies that there is at a very low risk of surface water flooding across most of the Site, refer to **Figure 3**.

The long-term flood map shows that the site is predominantly within a low-risk zone for surface water flooding. This is the general trend across the wider Harefield hospital site. However, the long-term flood map also shows there is a continuous band of low-risk localised flood spots located towards the east of the site where the flood depth could be up to 300mm. Topographical levels generally are generally flat on the site for the proposed development. However overall levels on the wider hospital site generally fall from northeast to southwest meaning in the event of a flood, flows are likely to flow away from the proposed development site following the natural topography of the overall site.

Additionally, the SWMP Phase 2 Risk assessment identified 17 Critical Drainage Areas (CDAs) as depicted in **Appendix B** (Figure i in the Surface water Management Plan Part 1- Evidence base). Two of the defining parameters for CDA classification in the report particularly related to surface water flooding are as follows:

- Topographical Low Points – areas which are at topographical low points throughout the Borough which result in small, discrete areas of deep surface water ponding; and
- Topographical Low-lying Areas - areas such as underpasses, subways and lowered roads beneath railway lines are more susceptible to surface water flooding;

As highlighted in in section 5.2, the site in Harefield is not within a critical drainage area, further minimising the risk of surface water flooding.

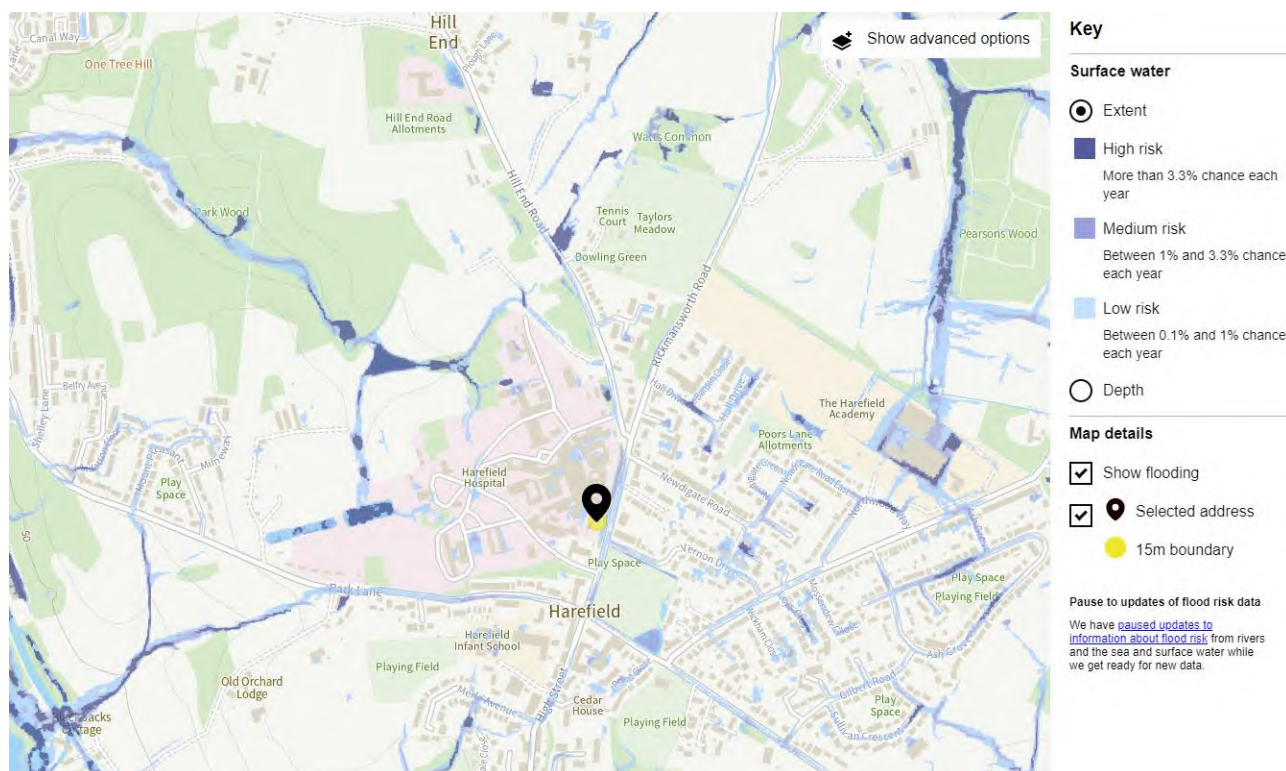


Figure 3: Extract from GOV.UK's Long Term Flood Risk Map for Surface Water Flooding

Based on the available information and modelling, localised surface water flooding is possible on the site. However, the probability is considered to be low. Based on the available information, the risk of surface water flooding on the site has been assessed as **Low**.

5.4 Artificial Sources of Flooding

According to the online GOV.UK's Long Term Flood Risk Map, the Site is not shown to be within the maximum extent of flooding from any reservoir.

Since spring 2009 all reservoirs, under the 1975 Reservoir Act, require the development of a Flood Plan and associated inundation mapping along with governing their maintenance and management. As such the risk of flooding from a reservoir is low and is being managed at a strategic level under the 1975 Reservoir Act.

No other artificial sources relevant to the Site have been noted in the London Borough of Hillingdon flood risk documentation review undertaken.

Based on the available information, the probability of flooding from artificial sources can be assessed as **negligible**.

5.5 Groundwater Flooding

Groundwater flooding is caused by the emergence of water originating from sub-surface permeable strata. In a groundwater flooding event, water levels rise sufficiently to intersect the surface and inundate low lying land.

Based on Figure 10.1 of London borough of Hillingdon's Phase 2 Risk Assessment detailed in the Surface water Management plan evidence base (**Appendix B**), No ground water flooding incidents have been recorded in close proximity to the site, and according to the BGS 1:625,000 Hydrogeology Map, the site is not located above consolidated aquifers and permeable superficial deposits where increased potential for elevated ground water is envisaged.

Additionally, the SWMP Phase 2 Risk assessment identified 17 Critical Drainage Areas (CDAs) as depicted in **Appendix B** (Figure i in the Surface water Management Plan Part 1- Evidence base). In the report, one of the defining parameters for a CDA related to groundwater is as follows:

"A discrete geographic area (usually a hydrological catchment) where multiple and interlinked sources of flood risk (surface water, groundwater, sewer, main river and/or tidal) cause flooding in one or more Local Flood Risk Zones during severe weather thereby affecting people, property or local infrastructure".

As highlighted in section 5.2 the site is not within a critical drainage area, further minimizing the risk of groundwater flooding on the site.

Groundwater conditions beneath the site will be confirmed during future SI which are to be completed at the post planning stage.

Based on the information available and the assessment undertaken, the probability of groundwater flooding impacting the Site can be assessed as **negligible**.

5.6 Sewers Flooding

The risk of flooding to the site from overloaded sewers and surrounding drainage is based on the capacity of the network, operational / maintenance of the network and topography in the vicinity of the Site. When sewer flooding occurs sewage can be released back out of the drainage network through the top of manholes, drains or low-lying fittings such as toilets and showers.

Based on Figure 9.1 of London borough of Hillingdon's Phase 2 Risk Assessment detailed in the Surface water Management plan - Evidence base (**Appendix B**) the site is located in an area with a low recorded historical sewer flooding incidents (i.e. 1-5) depicted in green.

Additionally, the SWMP Phase 2 Risk assessment identified 17 Critical Drainage Areas (CDAs) as depicted in **Appendix B** (Figure i in the Surface water Management Plan Part 1- Evidence base). Two of the defining parameters for CDA classification in the report particularly related to sewers flooding are as follows:

- Sewer Flood Risk – areas where extensive and deep surface water flooding is likely to be the influence of sewer flooding mechanisms alongside pluvial and groundwater sources.

As highlighted in section 5.2 the site is not in a critical drainage area further minimizing the risk of sewer sewers flooding on the site.

The sewer flooding mapping by the London Borough of Hillingdon was conducted in liaison with Thames Water. Despite the LLFA model's failure to predict future sewer flooding, it is likely that the areas facing sewer flooding in the past have been targeted for maintenance and therefore may not be at greatest risk in the future.

Based on the information available the probability of sewer flooding impacting the Site can be assessed as **negligible**.

7 Flood Risk Mitigation Measures

The below flood mitigation measures are required to manage the risk of flooding identified at the site. The proposed mitigation measures are based on best practice and the EA guidance and advice. Additionally, the scheme aims to comply with DEFRA's guidance on Improving the Flood Performance of New Buildings so that the ground level in particular can quickly recover from flood events.

As discussed in the previous sections the only source of flooding at the site was identified as surface water flooding.

Any potential risk linked to surface water runoff generated is limited through the design of the Drainage Strategy, with attenuation provided to cater for the 1:100 year + 40% climate change event. The management of surface water runoff is discussed in the proposed Drainage Strategy below.

The proposed drainage strategy includes a range of design and management solutions with general design guidance outlined below.

7.1 Sitewide Levels

To reduce the risk of surface water flooding on the rooms on site, these have been designed such that the rooms are elevated above the ground level as shown in Figure 4. Additionally, it is suggested that the external ground levels be designed promote positive drainage towards the infiltration trenches. This design then protects the enclosures from surface water flows during storms of higher return periods. Additionally, it is advisable for the Site's external levels to be maintained at a slightly higher elevation than the adjacent roads to minimize the likelihood of surface water flow from paths encroaching onto the Site. The Levels design for this site will be done by others.

8 Drainage Strategy

8.1 Sustainable Surface Water Drainage Strategy

The proposed surface water has been designed in accordance with best practise and guidance to ensure that surface water is managed effectively to limit the risk off-site as well as on site.

The drainage strategy has been based on London Plan policy SI 13 which states that surface water runoff from a Site shall discharge to one of the following in order of priority:

Discharge priority (<i>in descending order</i>)	Application to the Site
Rainwater use as a resource (for example rainwater harvesting, blue roofs for irrigation).	The proposed development includes the replacement of existing generators. In this arrangement, the effective integration of green infrastructure, like blue roofs and rainwater collection systems, on GRP enclosures at the Site may not be feasible.
Rainwater infiltration to ground at or close to source.	Infiltration testing was conducted onsite on 07.10.24 and an infiltration rate of 0.000094 m/s was established for the site. The proposed strategy for surface water drainage involves the use of a gravel layer beneath and around the equipment for infiltration of surface water flows and management of surface water close to point of generation in line with the Lond Plan policy drainage hierarchy. Although, infiltration is typically implemented at distances greater than 5m from foundations or structures, Appendix H includes a geotechnical investigation conducted on the site confirming that infiltration within distances less than 5m from the structure is acceptable for this site. It is also noted that the wider hospital site's surface water drainage system appears to be discharging into a soakaway situated between the Heart Science Centre and the temporary wards, located to the north of the Heart Science Centre. Additionally, there exist opportunities for the implementation of filter strips aimed at water quality management within the site.
Rainwater attenuation in green infrastructure features for gradual release (for example green roofs, rain gardens).	Given the configuration of the proposed generator replacement, the implementation of green infrastructure, such as green roofs and rainwater harvesting systems on GRP enclosures, may not be feasible on the site. However, there are potential opportunities to integrate rain gardens within the site, and the feasibility of such integration will be evaluated in subsequent phases of the design process. Additionally, rainwater attenuation through geo-cellular attenuation tanks is deemed unsuitable for the site due to space restrictions and congested existing utilities on the site.

Rainwater discharge direct to a watercourse (unless not appropriate)	The site is not situated near any existing water courses. The nearest watercourse, River Colne, is approximately 1.1 km to the southwest of the site. Consequently, discharging into a watercourse is not a viable option.
Controlled rainwater discharge to a surface water sewer or drain.	The existing hospital site is currently served by several private surface water sewers, as outlined in section 3.3.2. Nonetheless, since the proposal is to manage surface water drainage discharge onsite through infiltration to the ground as highlighted above, controlled discharge of rainwater to a surface water sewer or drain will not be considered for this site.
Controlled rainwater discharge to a combined sewer	There are no combined water sewers in close proximity to the site. Consequently, discharge of surface water flows into a combined sewer is not deemed feasible for this site.

Proposed drainage strategy for the site incorporating new generators for Substation 2 and 3.

The designated area for the installation of two generators a switch room and a fuel tank will utilise a system of interlinking infiltration trenches around and below the proposed equipment. The surface water collected from the equipment and surrounding areas within the redline boundary will flow to the gravel infiltration trenches for subsequent infiltration into the ground. Please refer to Figure 4 below for the proposed drainage gravel below slab arrangement. Although infiltration is typically implemented at distances greater than 5m from a structure or foundation, **Appendix H** includes a geotechnical investigation conducted by Lucion Engineering confirming that infiltration at distances less than 5m from the structure or foundation is acceptable for this site.

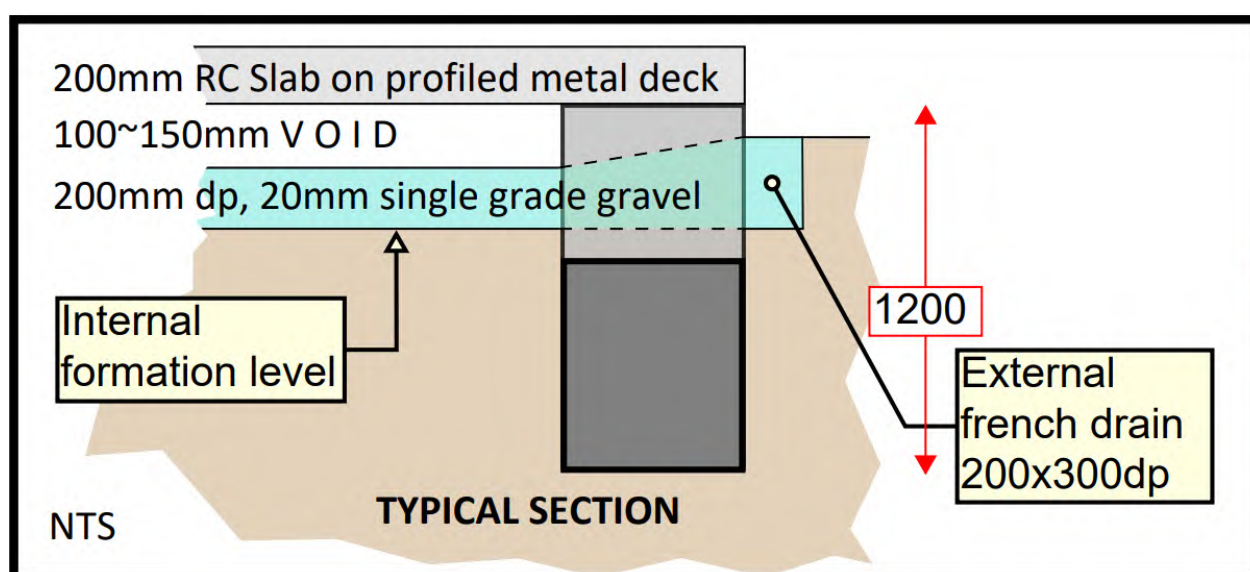


Figure 4: Proposed drainage gravel beneath slab arrangement.

Levels onsite will be designed by others to promote positive flow towards the infiltration areas as detailed in **Appendix C**. This levels design proposed by others should also ensure that no flows from the boundary

encroach onto surrounding areas onsite. Additionally, the proposed porous gravel will provide temporary attenuation in line with the London Plan Policy requirements. Please refer to **Appendix C** for further details on the proposed drainage strategy.

Given the characteristics of the development, oil spills are possible during the refilling of the generators. Mitigation measures as proprietary treatment systems, bunding and spill kits will be considered as the design develops.

As mentioned previously, there are no local watercourses within proximity of the Site therefore the option to discharge into local watercourses is not considered suitable for this site.

Please refer to **Appendix D** for the London Borough of Hillingdon SuDS Proforma issued to the LLFA. Contact was made with the London borough of Hillingdon as the LLFA, but their confirmation was still pending at the time of writing of this report. However, the proposed drainage strategy includes management of surface water flows within the proposed development in line with the London Plan Policy drainage hierarchy as well as the wider hospital site drainage strategy where in surface water flows are discharged into the soakaway located between the heart science centre and the temporary wards; It is there unlikely that the proposed strategy will be unacceptable to the LLFA.

8.1.1 Surface Water Modelling

When undertaking the modelling for the surface water drainage strategy the following assumptions were made:

- Causeway flow has been used for all hydraulic design;
- FEH rainfall data has been used when determining existing runoff rates;
- HR Wallingford Greenfield Runoff Rate Estimation Tool (www.uksuds.com) was used to determine the proposed greenfield runoff rates;
- CV values are set at 1.0 (100%) for hardstanding areas (roads/roofs) within Causeway flow, for conservative measures at this stage, at the detailed design this figure will be reviewed in consultation with the LLFA;
- The Site area is approximately 0.031ha;
- The existing Site has been considered to be 100% impermeable; and
- The proposed drainage area of the site is considered to be 0.031ha.
- Design infiltration rate of 0.000094 m/s.
- The proposed drainage network has been designed for the 1 in 100 year plus 40% climate change event.

In accordance with best practise as well as guidance from the Environment Agency guidance and Sewers for Adoption 8th Edition the proposed drainage strategy has been designed to ensure that:

- There will be no surcharging of the drainage system for a 1 in 2-year storm;
- The drainage can be surcharged with no flooding for a 1 in 30-year storm; and
- The drainage can flood on site for a 1 in a 100-year storm with a 40% climate change allowance provided the flood water remains on site and does not flood habitable areas or affect safe ingress and egress to the site for occupiers.

All surface water drainage options outlined in this report adhere to these principles.

Modelling results are available in **Appendix E**.

8.1.2 Existing Surface Water Network Results

The existing positively drained area discharging to the proposed infiltration trenches is 0.031ha. Causeway Flow was used to determine the existing surface water discharge rates. Results are summarised in **Table 2**.

Table 2: Exiting Surface Water Discharge Rates

Return Period (years)	Existing Surface Water Discharge Rates (l/s)
1 in 2	0.5
1 in 30	1.4
1 in 100	1.8
1 in 100 + 40%	N/A

Pre-development discharge

Site Makeup	Brownfield	OK
Brownfield Method	MRM	Cancel
Contributing Area (ha)	0.031	
PIMP (%)	100	
CV	1.000	
Time of Concentration (mins)	5.00	
Betterment (%)	0	
	Calc	

Return Period (years)	Q (l/s)
1	
2	0.5
30	1.4
100	1.8

Note: FEH point descriptors can be downloaded from fehweb.ceh.ac.uk

Only XML file format can be used

FEH-22 is the current FEH data and this should be used for new development

ReFH2 legacy – Doesn't contain the new BFIHOST19 descriptor
ReFH2 – Contains the new BFIHOST19 descriptor

8.1.3 Proposed Surface Water Network Results

A previously highlighted, the surface water drainage discharge will be managed onsite in accordance with the requirements of the London Plan policy SI 13. Causeway flow has been utilised to determine the required infiltration area to accommodate up to and including the 1:100 year +40%cc storm event. **Figure 4** shows the approximate area earmarked for proposed generator location on the site. The results of this modelling are provided within **Appendix E**, and infiltration requirements are summarised in **Table 4**.



Figure 5: Catchment areas and discharge point locations.

Table 3: Proposed Infiltration Requirements

Infiltration device	Total Drainage Area (Ha)	Soil Infiltration Rate (l/s)	Effective storage volume for 1:100yr+40% CC (m³)	Plan Area Required (m²)
Drainage gravel (porosity approximately = 0.30)	0.031	0.000094 m/s	8.261	126.204

Although the above strategy identifies the general principles of surface water drainage management at the Site, assumptions and design solutions proposed will need to be re-assessed at the detailed design stage and any necessary approvals sought.

8.1.4 Surface Water Quality

The assessment for the generator site for substation 2 and 3 is consistent with the site's pollution risk management strategy, utilizing a basic qualitative method as described in the CIRIA SuDS Manual C753. This process involves assessing the potential pollution hazard levels relevant to the site. Given the planned bulk fuel tanks intended to support the generators, the presence of non-residential fuel is expected, which consequently leads to a high pollution hazard classification, as indicated in Table 26.2 of the CIRIA SuDS Manual C753 and illustrated in Figure 4 below.

TABLE 26.2 Pollution hazard indices for different land use classifications				
Land use	Pollution hazard level	Total suspended solids (TSS)	Metals	Hydrocarbons
Residential roofs	Very low	0.2	0.2	0.05
Other roofs (typically commercial/ industrial roofs)	Low	0.3	0.2 (up to 0.8 where there is potential for metals to leach from the roof)	0.05
Individual property driveways, residential car parks, low traffic roads (eg cul de sacs, homezones and general access roads) and non-residential car parking with infrequent change (eg schools, offices) ie < 300 traffic movements/day	Low	0.5	0.4	0.4
Commercial yard and delivery areas, non-residential car parking with frequent change (eg hospitals, retail), all roads except low traffic roads and trunk roads/motorways ¹	Medium	0.7	0.6	0.7
Sites with heavy pollution (eg haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways ¹	High	0.8 ²	0.8 ²	0.9 ²

Figure 6: Extract from CIRIA C753: Pollution Hazard Indices.

Given the significant pollution hazard risk anticipated during the refuelling operations for the tanks, it is recommended to implement proprietary systems such as proprietary treatment systems, bunding and spill kits will be considered as the design develops at this site before any infiltration into the ground. This recommendation is based on Table 26.4 from the CIRIA SuDS Manual C753, as illustrated in Figure 5. The selected mitigation strategies exhibit higher indices in comparison to the pollution hazard indices, demonstrating their effectiveness in pollutant removal prior to the discharge of surface water into the existing network. Table 26.4 has been chosen because the proposed drainage strategy discharges into a proposed infiltration trenches situated around the equipment within the area earmarked for the installation of new generators for sub-station 2 and 3.

TABLE 26.4 Indicative SuDS mitigation indices for discharges to groundwater				
Characteristics of the material overlying the proposed infiltration surface, through which the runoff percolates ¹		TSS	Metals	Hydrocarbons
A layer of dense vegetation underlain by a soil with good contaminant attenuation potential ² of at least 300 mm in depth ³		0.6 ⁴	0.5	0.6
A soil with good contaminant attenuation potential ² of at least 300 mm in depth ³		0.4 ⁴	0.3	0.3
Infiltration trench (where a suitable depth of filtration material is included that provides treatment, ie graded gravel with sufficient smaller particles but not single size coarse aggregate such as 20 mm gravel) underlain by a soil with good contaminant attenuation potential ² of at least 300 mm in depth ³		0.4 ⁴	0.4	0.4
Constructed permeable pavement (where a suitable filtration layer is included that provides treatment, and including a geotextile at the base separating the foundation from the subgrade) underlain by a soil with good contaminant attenuation potential ² of at least 300 mm in depth ³		0.7	0.6	0.7
Bioretention underlain by a soil with good contaminant attenuation potential ² of at least 300 mm in depth ³		0.8 ⁴	0.8	0.8
Proprietary treatment systems ^{5, 6}		These must demonstrate that they can address each of the contaminant types to acceptable levels for inflow concentrations relevant to the contributing drainage area.		

Figure 7: Extract from CIRIA C753: Indicative SuDS mitigation Indices.

8.2 Offsite Effects

The surface water drainage strategy demonstrates that the drainage network at the Site is designed to accommodate runoff during all events up to and including the 1 in 100-year plus 40% climate change allowance and therefore prevents potential exceedance flows off-site, refer to **Appendix F** for the exceedance flow routes plan.

The existing network shown in the Topographic and utilities survey conducted by EDI surveys Ltd appears to be discharging into a soak-away located between the Heart science centre and the temporary wards and there no offsite effects are expected as a result of the additional flows arising from the proposed development. The development will not cause effects elsewhere in terms of fluvial and tidal flood risk due to its location within Flood Zone 1.

8.3 Residual Flood Risk

The only likely risk of flooding from the Proposed Development was identified as surface water flooding, all other potential sources of flooding were considered negligible. As discussed in the previous sections, the proposed design solutions detailed in the proposed drainage strategy minimise any residual risk linked to potential extreme rainfall events.

It has been recommended that the Site's external levels remain slightly elevated compared to the surrounding roads to reduce risk of the surface water flow paths in the surrounding roads entering the site.

The proposed drainage strategy demonstrates that the drainage network at the Site is designed to accommodate runoff during all events up to and including the 1 in 100-year plus a 40% climate change allowance preventing potential exceedance flows off-site. Drainage exceedance routes have also been considered and allowed for as part of the development of parameters to ensure that any surface water runoff exceeding the drainage network capacity would naturally flow away. Refer to **Appendix F** for exceedance flow routes plan.

8.4 Foul Water Drainage Strategy

The development does not include any foul water flows from the site, therefore the foul water on the site will be retained as existing and as highlighted in section 3.3, the utilities surveys conducted by EDI surveys Ltd in December 2009 shows that there is 225mmØ foul water sewer to the southwest of the site, running from north to south. This sewer appears to be exiting the site presumably to discharge into a public network.

9 Drainage Maintenance and Management strategy

The Drainage Maintenance and Management Strategy (**Table 5**) has been produced based on the information provided within the SUDS Manual; it is expected that the maintenance for the entire scheme will be under the overall management responsibility of Guy's and St Thomas NHS Trust as they hold ownership of the generators. However, WPBS will manage the maintenance on the client's behalf, in which a suitably qualified maintenance specialist will undertake the work.

Table 4: Proposed Drainage Maintenance Strategy

Drainage Feature	Regular Maintenance	Occasional/Remedial Maintenance	Monitoring
Pipes	Initial inspection should be provided as post construction CCTV survey.		N/A.
	Inspect for evidence of poor operation via water level in chambers. If required, take remedial action.		3-monthly, 48 hours after large storms.
	Rod through poorly performing runs as initial remediation		As required.
	If continued poor performance jet and CCTV survey poorly performing runs.		As required.
	Seek advice as to remediation techniques suitable for the type of performance issue and location.		As required if above does not improve performance.
		Check and remove large vegetation growth near pipe runs.	6 monthly.
Drainage channels/Gullies	Inspections will include gratings; covers including their locking bolts; sumps and sump buckets; exposed concrete surround and adjacent surfacing. Check for accumulation of debris and silt and cleaned as necessary. Gratings, frames and all associated locking parts to be checked for damage. Exposed concrete and adjacent surfacing to be checked for cracking and general damage. Check condition of inlet and outlet pipes, flow controls, baffles and isolation structures	Channel cleaning will be by flushing with water or high-pressure jetting (no boiling water or cleaning agent will be used). All silt buckets and sumps will be cleaned out replaced back into the units ensuring they are correctly fitted. All channel surfaces and joints will be checked and repaired as necessary. Repair/rehabilitation of inlets, outlet, overflows and vents, as required.	Inspect every 4 months or after large storm.

Catchpit Manholes/Inspection Chambers	Check for accumulation of debris and silt and cleaned as necessary. Covers and frames to be checked for damage. Exposed concrete and adjacent surfacing to be checked for cracking and general damage. Check condition of inlet and outlet pipes, flow controls, baffles and isolation structures	Clean as necessary. All manhole and inspection chamber covers and frames to be replaced as necessary. Repair exposed concrete and surfacing as necessary. Repair/rehabilitation of inlets, outlet, overflows and vents, as required.	Inspect every 6 months or after large storm.
Infiltration trenches and Blue Roof	Inspection chambers will be checked every 6 months for the accumulation of debris and silt and cleaned as necessary. Remove debris from the catchment surface where it may cause risk to performance (monthly).		Inspect every 6 months or after large storm.
Proprietary treatment system	Remove litter and debris and inspect for sediment, oil and grease accumulation; six monthly Change the filter media; as recommended by manufacturer Remove sediment, oil, grease and floatables; as necessary – indicated by system inspections or immediately following significant spill	Replace malfunctioning parts or structures; as required	Inspect for evidence of poor operation; six monthly Inspect filter media and establish appropriate replacement frequencies; six monthly Inspect sediment accumulation rates and establish appropriate removal frequencies; monthly during the first half year of operation, then every six months

10 Conclusion

This FRA and DS has been prepared on behalf of Guy's and St Thomas NHS Trust to support the planning application for the generator replacement project that will include the installation of 2 x new generators, a bulk fuel tank and a control room for substations 2 and 3.

The EA and the online GOV.UK Flood Map for Planning identify that the Site is in Flood Zone 1 where the probability of river or sea flooding is less than 0.1% (1 in 1000) chance in any given year. Meaning that the probability of tidal or fluvial flooding can therefore be assessed as negligible.

Surface Water could in theory be a source of flood risk due to the localised flood risk spots present in on the Site. This however has been mitigated by the design of an outline drainage strategy which demonstrates the ability to accommodate surface water runoff during all events up to and including the 100 year plus an 40% climate change allowance to prevent potential exceedance flows off-site. As such the Proposed Development will reduce the probability of surface water flooding at the Site and in the surroundings.

Other potential sources of flooding have been investigated and are deemed negligible to low. Considering the above, it can be concluded that there are no flood risk, surface or foul water drainage grounds on which to refuse this application.

The proposed surface water drainage strategy optimises the use of SuDS in line with best practice, national and local policy. It is proposed that surface water runoff will be managed locally to infiltrate into the ground. The surface water flows from the area earmarked for generator location for substation 2 and 3 will drain into a proposed infiltration trenches located beneath and around the equipment with levels onsite being designed to promote positive flow towards the infiltration trenches. Levels design to be done by others.

Furthermore, the potential for implementing Sustainable Drainage Systems (SuDS) has been evaluated for the site. As previously noted, the proposed development involves replacing existing generators and installing new units within Glass Reinforced Plastic (GRP) enclosures. This configuration significantly limits the effective integration of green infrastructure elements, such as blue roofs and rainwater harvesting systems, within the GRP structures. Nevertheless, there are viable opportunities to incorporate rain gardens on the site. The feasibility of this integration will be examined in subsequent phases of the design process.



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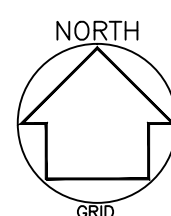
www.meinhardt.co.uk

Appendix A – Topographic and Utilities Survey.

[illegible]

SHEET LAYOUT

		11596/T/01A-11	11596/T/02A-11
		11596/T/03A-11	11596/T/04A-11
11596/T/05A-11		11596/T/06A-11	11596/T/07A-11
11596/T/08A-11	11596/T/09A-11	11596/T/10A-11	11596/T/11A-11



NOTES:-

1. The accuracy and content of this drawing are dependent on the original specification and the user shall ensure that the EDI should be consulted before use of other scales or use in CAD form.
2. Where underground services are shown, all reasonable care has been taken within the scope of the original specification and requirement. Before use of this information the user shall consult EDI and satisfy themselves of the completeness and accuracy of such details before undertaking any works. Due to the nature of this work and the limitations imposed by ground conditions and the detection equipment no guarantee can be given that all services have been correctly identified.

All reasonable care has been taken in the survey data represented in this drawing but discrepancies must be reported to E2 immediately.

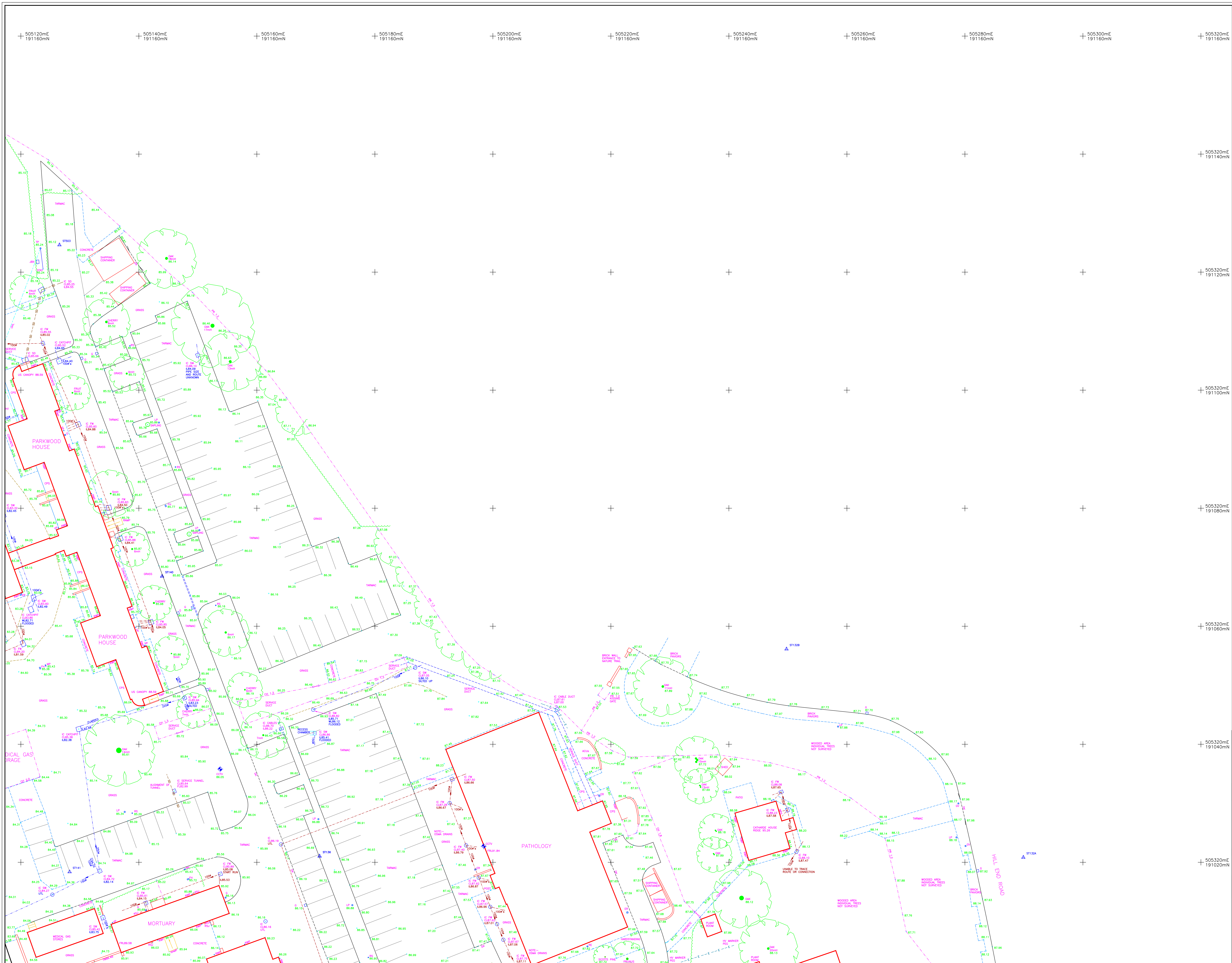
It is intended to produce the best possible results within the specification and cost constraints of our clients. Any comments are most welcome.

All levels and over/under dimensions are in millimetres described.

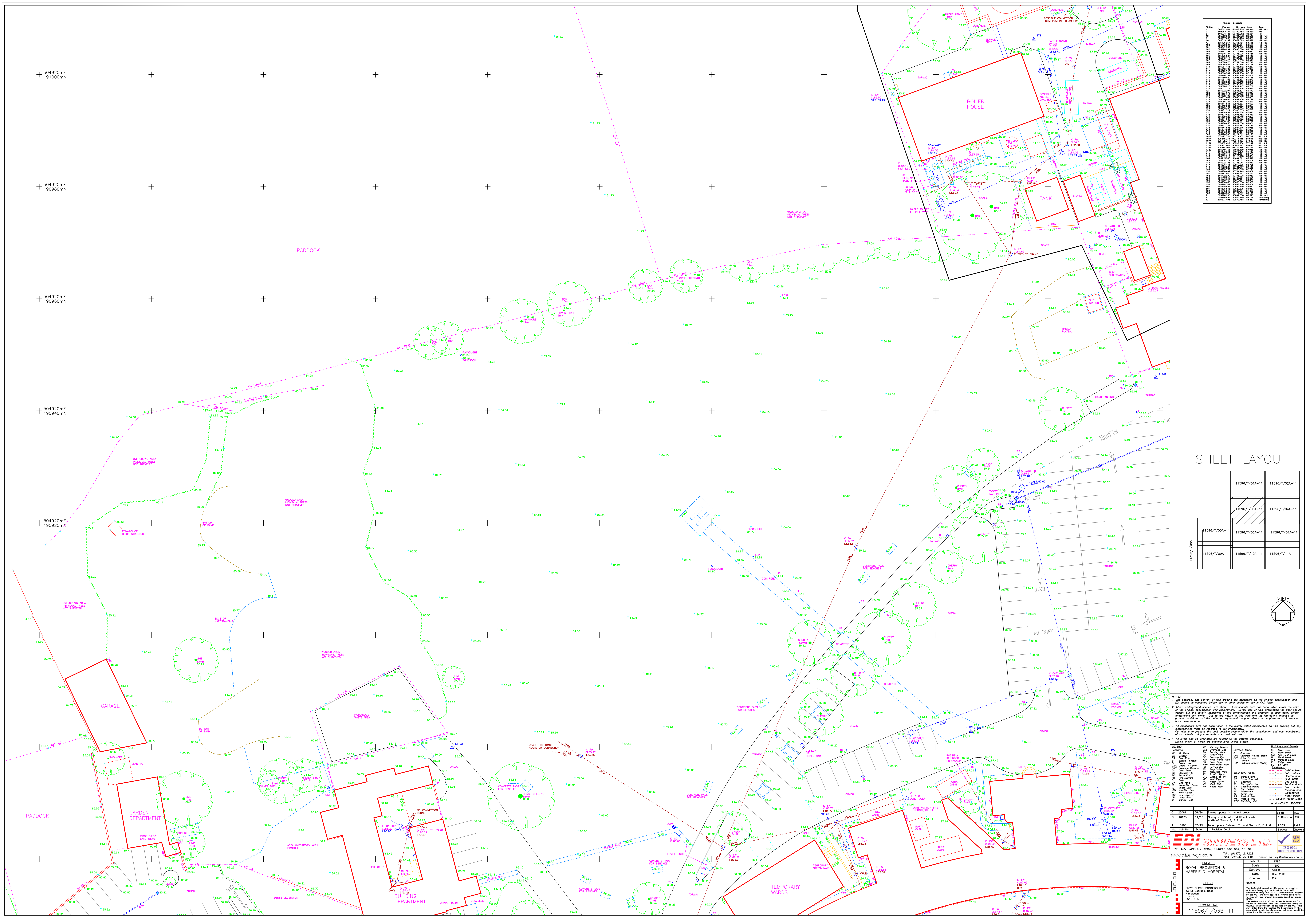
Levels above sea level are shown: level means average.

AutocAD 2013

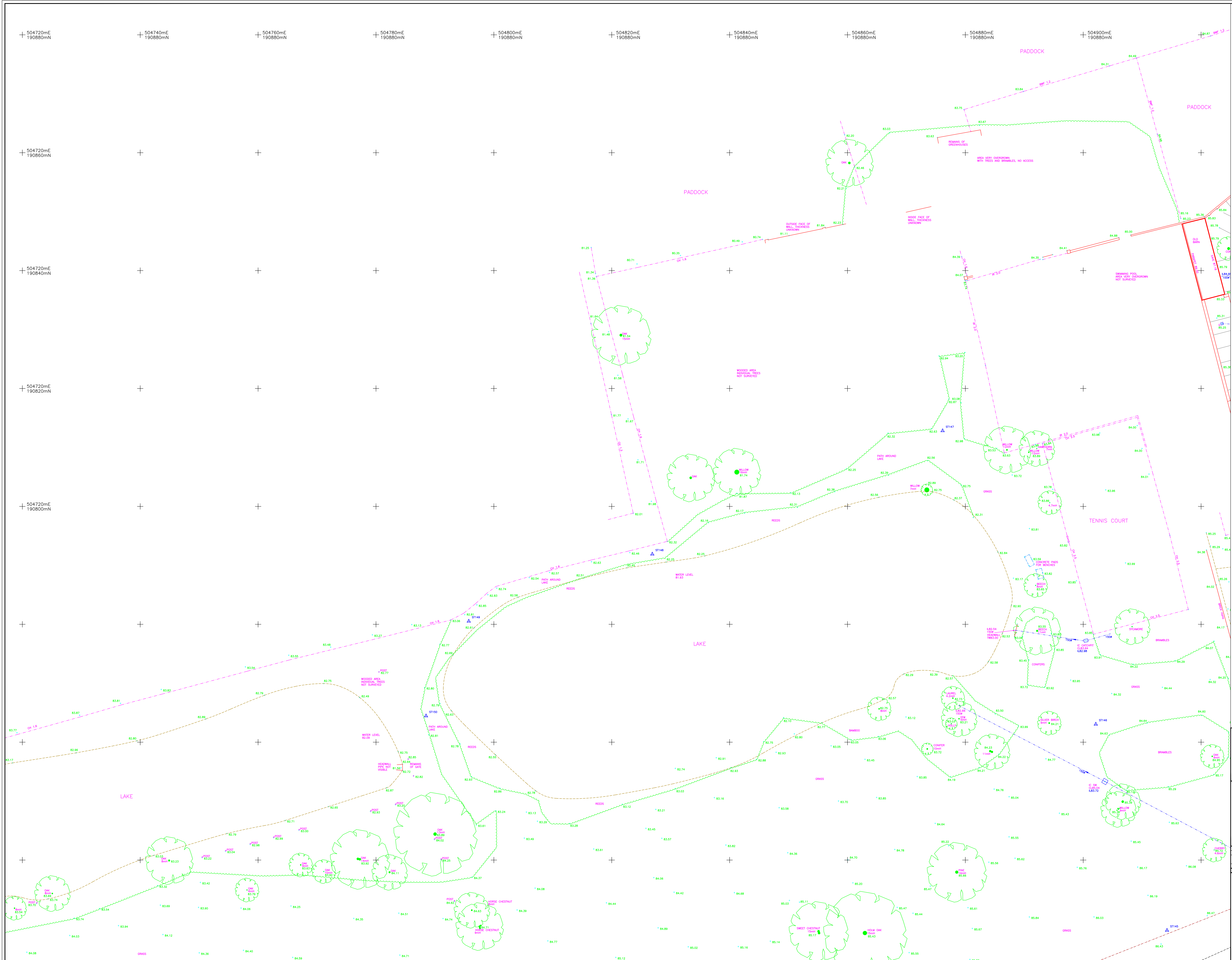
[illegible]



Station Schedule			
Station	Chainage	Level	Notes
1	0+00.00	85.10	Start of Road
2	0+05.00	85.12	
3	0+10.00	85.15	
4	0+15.00	85.18	
5	0+20.00	85.20	
6	0+25.00	85.22	
7	0+30.00	85.25	
8	0+35.00	85.28	
9	0+40.00	85.30	
10	0+45.00	85.32	
11	0+50.00	85.35	
12	0+55.00	85.38	
13	0+60.00	85.40	
14	0+65.00	85.42	
15	0+70.00	85.45	
16	0+75.00	85.48	
17	0+80.00	85.50	
18	0+85.00	85.52	
19	0+90.00	85.55	
20	0+95.00	85.58	
21	1+00.00	85.60	
22	1+05.00	85.62	
23	1+10.00	85.65	
24	1+15.00	85.68	
25	1+20.00	85.70	
26	1+25.00	85.72	
27	1+30.00	85.75	
28	1+35.00	85.78	
29	1+40.00	85.80	
30	1+45.00	85.82	
31	1+50.00	85.85	
32	1+55.00	85.88	
33	1+60.00	85.90	
34	1+65.00	85.92	
35	1+70.00	85.95	
36	1+75.00	85.98	
37	1+80.00	86.00	
38	1+85.00	86.02	
39	1+90.00	86.05	
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75	3+70.00	86.95	
76	3+75.00	86.98	
77	3+80.00	87.00	
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79	3+90.00	87.05	
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81	4+00.00	87.10	
82	4+05.00	87.12	
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88	4+35.00	87.28	
89	4+40.00	87.30	
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113	5+60.00	87.90	
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120	5+95.00	88.08	
121	6+00.00	88.10	
122	6+05.00	88.12	
123	6+10.00	88.15	
124	6+15.00	88.18	
125	6+20.00	88.20	
126	6+25.00	88.22	
127	6+30.00	88.25	
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202	10+05.00	90.12	
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243	12+10.00	91.15	
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277	13+80.00	92.00	
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289	14+40.00	92.30	
290	14+45.00	92.32	
291	14+50.00	92.35	
292	14+55.00	92.38	
293	14+60.00	92.40	
294	14+65.00	92.42	
295	14+70.00	92.45	
296	14+75.00	92.48	
297	14+80.00	92.50	
298	14+85.00	92.52	
299	14+90.00	92.55	
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303	15+10.00	92.65	
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305	15+20.00	92.70	
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307	15+30.00	92.75	
308	15+35.00	92.78	
309	15+40.00	92.80	
310	15+45.00	92.82	
311	15+50.00	92.85	
312	15+55.00	92.88	
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314	15+65.00	92.92	
315	15+70.00	92.95	
316	15+75.00	92.98	
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318	15+85.00	93.02	
319	15+90.00	93.05	
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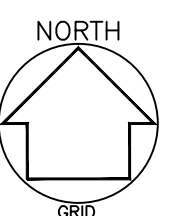


Station Schedule			
Station	Chainage	Height	Notes
1	100+00	84.10	Start of Road
2	100+10	84.15	
3	100+20	84.20	
4	100+30	84.25	
5	100+40	84.30	
6	100+50	84.35	
7	100+60	84.40	
8	100+70	84.45	
9	100+80	84.50	
10	100+90	84.55	
11	101+00	84.60	
12	101+10	84.65	
13	101+20	84.70	
14	101+30	84.75	
15	101+40	84.80	
16	101+50	84.85	
17	101+60	84.90	
18	101+70	84.95	
19	101+80	85.00	
20	101+90	85.05	
21	102+00	85.10	
22	102+10	85.15	
23	102+20	85.20	
24	102+30	85.25	
25	102+40	85.30	
26	102+50	85.35	
27	102+60	85.40	
28	102+70	85.45	
29	102+80	85.50	
30	102+90	85.55	
31	103+00	85.60	
32	103+10	85.65	
33	103+20	85.70	
34	103+30	85.75	
35	103+40	85.80	
36	103+50	85.85	
37	103+60	85.90	
38	103+70	85.95	
39	103+80	86.00	
40	103+90	86.05	
41	104+00	86.10	
42	104+10	86.15	
43	104+20	86.20	
44	104+30	86.25	
45	104+40	86.30	
46	104+50	86.35	
47	104+60	86.40	
48	104+70	86.45	
49	104+80	86.50	
50	104+90	86.55	
51	105+00	86.60	
52	105+10	86.65	
53	105+20	86.70	
54	105+30	86.75	
55	105+40	86.80	
56	105+50	86.85	
57	105+60	86.90	
58	105+70	86.95	
59	105+80	87.00	
60	105+90	87.05	
61	106+00	87.10	
62	106+10	87.15	
63	106+20	87.20	
64	106+30	87.25	
65	106+40	87.30	
66	106+50	87.35	
67	106+60	87.40	
68	106+70	87.45	
69	106+80	87.50	
70	106+90	87.55	
71	107+00	87.60	
72	107+10	87.65	
73	107+20	87.70	
74	107+30	87.75	
75	107+40	87.80	
76	107+50	87.85	
77	107+60	87.90	
78	107+70	87.95	
79	107+80	88.00	
80	107+90	88.05	
81	108+00	88.10	
82	108+10	88.15	
83	108+20	88.20	
84	108+30	88.25	
85	108+40	88.30	
86	108+50	88.35	
87	108+60	88.40	
88	108+70	88.45	
89	108+80	88.50	
90	108+90	88.55	
91	109+00	88.60	
92	109+10	88.65	
93	109+20	88.70	
94	109+30	88.75	
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97	109+60	88.90	
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99	109+80	89.00	
100	109+90	89.05	
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103	110+20	89.20	
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107	110+60	89.40	
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109	110+80	89.50	
110	110+90	89.55	
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113	111+20	89.70	
114	111+30	89.75	
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116	111+50	89.85	
117	111+60	89.90	
118	111+70	89.95	
119	111+80	90.00	
120	111+90	90.05	
121	112+00	90.10	
122	112+10	90.15	
123	112+20	90.20	
124	112+30	90.25	
125	112+40	90.30	
126	112+50	90.35	
127	112+60	90.40	
128	112+70	90.45	
129	112+80	90.50	
130	112+90	90.55	
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132	113+10	90.65	
133	113+20	90.70	
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137	113+60	90.90	
138	113+70	90.95	
139	113+80	91.00	
140	113+90	91.05	
141	114+00	91.10	
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219	121+80	95.00	
220	121+90	95.05	
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238	123+70	95.95	
239	123+80	96.00	
240	123+90	96.05	
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249	124+80	96.50	
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259	125+80	97.00	
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272	127+10	97.65	
273	127+20	97.70	
274	127+30	97.75	
275	127+40	97.80	
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278	127+70	97.95	
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328	132+70	100.45	
329	132+80	100.50	
330	132+90	100.55	
331	133+00	100.60	
332	133+10	100.65	

[illegible]

SHEET LAYOUT

Figure 1 illustrates the relationship between the number of nodes (N) and the number of processors (P) for a 2D grid. The grid is divided into four quadrants, each labeled with a processor ID and a range of node IDs. The top-left quadrant is labeled "11596/7/01A-11". The top-right quadrant is labeled "11596/7/02A-11". The bottom-left quadrant is labeled "11596/7/03A-11". The bottom-right quadrant is labeled "11596/7/04A-11". The grid is further divided into smaller sub-grids, with some sub-grids labeled "11596/7/05A-11", "11596/7/06A-11", "11596/7/07A-11", "11596/7/08A-11", "11596/7/09A-11", "11596/7/10A-11", and "11596/7/11A-11". The grid is also labeled with "11596/7/00A-11" on the left and "11596/7/01A-11" on the top.



NOTES:

1. The occupancy and content of this drawing are dependent on the original specification and the user shall be consulted before use of other scales or use in CAD form.
2. Where underground services are shown, all reasonable care has been taken within the spirit of the original specification and requirement. Before use of this information the user should ensure that the nature of the work is such that the user is not exposed to any risk in undertaking any works. Due to the nature of this work and the limitations imposed by ground conditions and the detection equipment no guarantee can be given that all services have been recorded.
3. All reasonable care has been taken in the survey data represented on this drawing but any discrepancies must be reported to EDI immediately.
4. Our aim is to produce the most accurate information within the specification and cost constraints of our clients. Any comments are most welcome.
5. All levels and co-ordinates are related to the datum described.
6. Unless otherwise stated, all dimensions are in millimetres.

[illegible]

C	2001	08/24	Survey update in marked area	1	Lrg	Rd	
C	14033	11/78	Survey update with additional level	1	In Basement	Rd	
			North of Wards E, F & G				
A	15100	03/75	Survey update from 11th and Wards E, F & G	1	1000	Rd	
N	00	00	Revision Detail	1	Survey	Rd	

EDI SURVEYS LTD.

163-165, RANGLASH ROAD, IPSWICH, SUFFOLK, IP2 6GA
www.edisurveys.co.uk

ISO 9001
REGISTERED

PROJECT

ROYAL BROMPTON &
 HAREFIELD HOSPITAL

CLIENT

FLOOR SLAB PARTNERSHIP
 31 St George's Road
 Harefield, Bucks
 London SL9 6LH

DRAWING NO.

1159672/05B-11

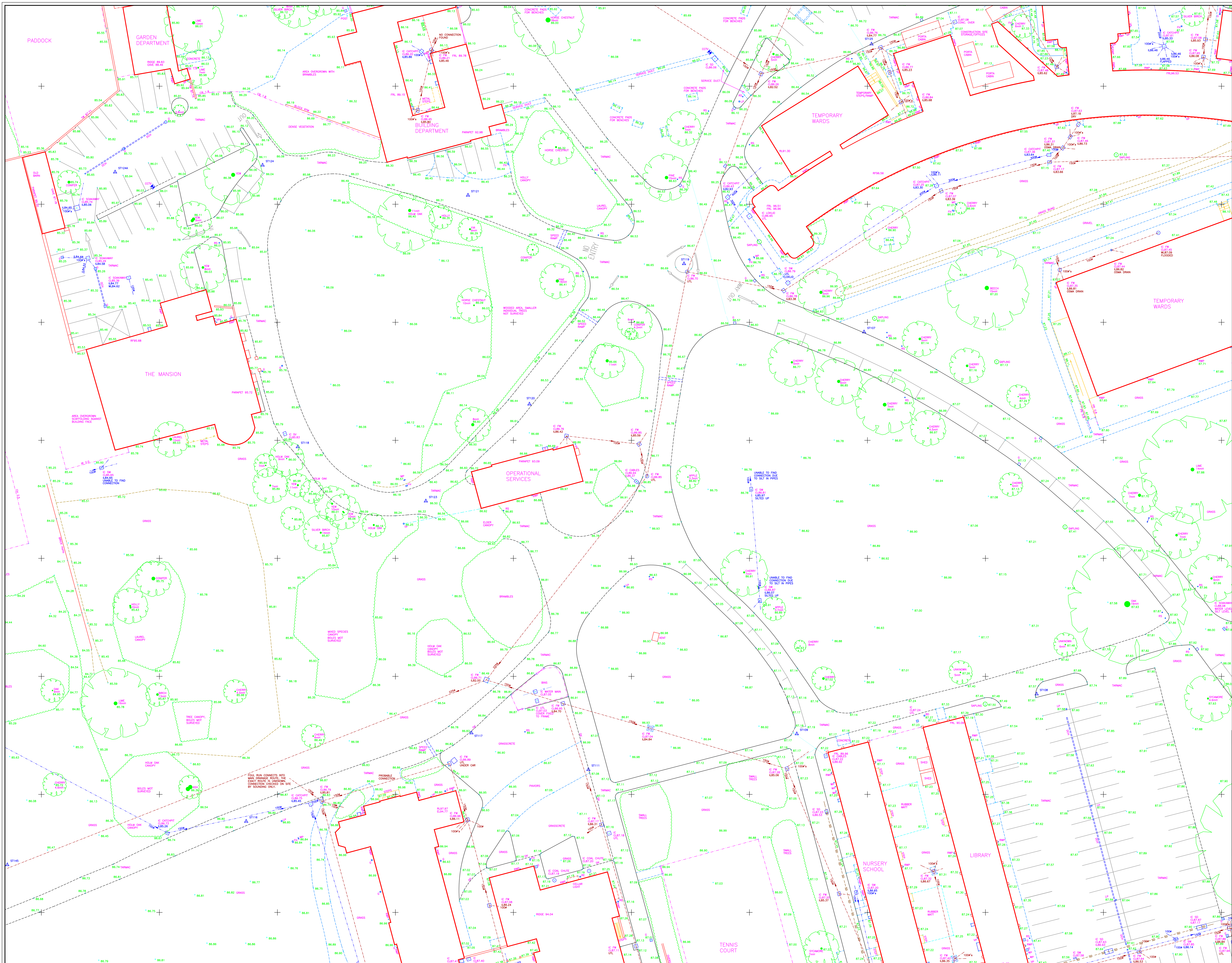
Job No. A1598

Job No. 1200

Surveyor's Name
 Date
 Checked Date: Dec. 2009

Notes:

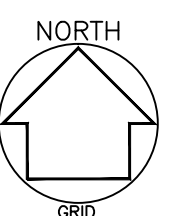
The above survey was carried out in order to build a new floor slab and to be installed from 2nd to 3rd floor. The survey was carried out in order to build a new floor slab and to be installed from 2nd to 3rd floor. The survey was carried out in order to build a new floor slab and to be installed from 2nd to 3rd floor.



Station		Elevation	
100	100.00	100.00	100.00
101	101.00	101.00	101.00
102	102.00	102.00	102.00
103	103.00	103.00	103.00
104	104.00	104.00	104.00
105	105.00	105.00	105.00
106	106.00	106.00	106.00
107	107.00	107.00	107.00
108	108.00	108.00	108.00
109	109.00	109.00	109.00
110	110.00	110.00	110.00
111	111.00	111.00	111.00
112	112.00	112.00	112.00
113	113.00	113.00	113.00
114	114.00	114.00	114.00
115	115.00	115.00	115.00
116	116.00	116.00	116.00
117	117.00	117.00	117.00
118	118.00	118.00	118.00
119	119.00	119.00	119.00
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SHEET LAYOUT

11596/T/01A-11		11596/T/02A-11	
11596/T/03A-11		11596/T/04A-11	
11596/T/05A-11		11596/T/06A-11	
11596/T/07A-11		11596/T/08A-11	
11596/T/09A-11		11596/T/10A-11	
11596/T/11A-11		11596/T/12A-11	



- NOTES:
- The accuracy and content of this drawing are dependent on the original specification and all should be checked before use of other scales or use in CAD form.
 - Where underground services are shown, all reasonable care has been taken within the limits of the original specification and equipment. Before use of this information the user should consult EDI and satisfy themselves of the completeness and accuracy of such data before undertaking any works. Due to the nature of the work and the limitations imposed by ground conditions and the detection equipment as provided can be given that all services have been recorded.
 - All reasonable care has been taken in the survey, design and construction of this drawing but any discrepancies must be reported to EDI immediately.
 - Our aim is to provide the best possible results within the specification and cost constraints of our clients. Any comments are most welcome.
 - All heads and co-ordinates are related to the datum described.
 - Gravel above of tests are shown 'near' unless stated.

LEGEND		Building Level Details	
1. Boundary Line	2. Boundary Line	1. Flow Line	2. Flow Line
3. Boundary Line	4. Boundary Line	3. Flow Line	4. Flow Line
5. Boundary Line	6. Boundary Line	5. Flow Line	6. Flow Line
7. Boundary Line	8. Boundary Line	7. Flow Line	8. Flow Line
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11. Boundary Line	12. Boundary Line	11. Flow Line	12. Flow Line
13. Boundary Line	14. Boundary Line	13. Flow Line	14. Flow Line
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97. Boundary Line	98. Boundary Line	97. Flow Line	98. Flow Line
99. Boundary Line	100. Boundary Line	99. Flow Line	100. Flow Line

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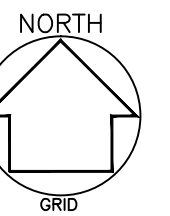
CLIENT: FLOU SLASH PARTNERSHIP

11596/T/06B-11

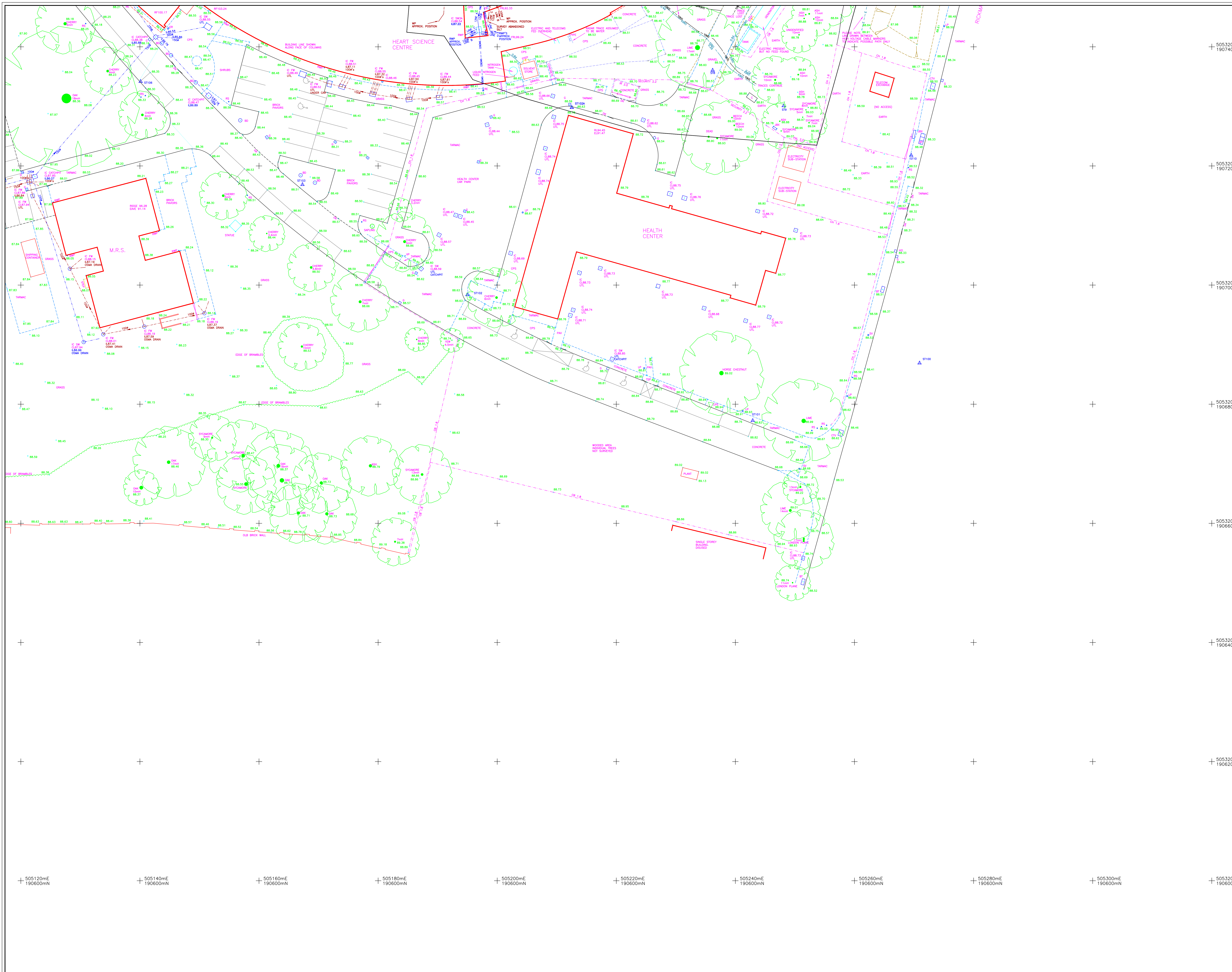
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SHEET LAYOUT

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		11596/T/03A-11	11596/T/04A-11
	11596/T/05A-11	11596/T/06A-11	11596/T/07A-11
11596/T/08A-11	11596/T/09A-11	11596/T/10A-11	11596/T/11A-11

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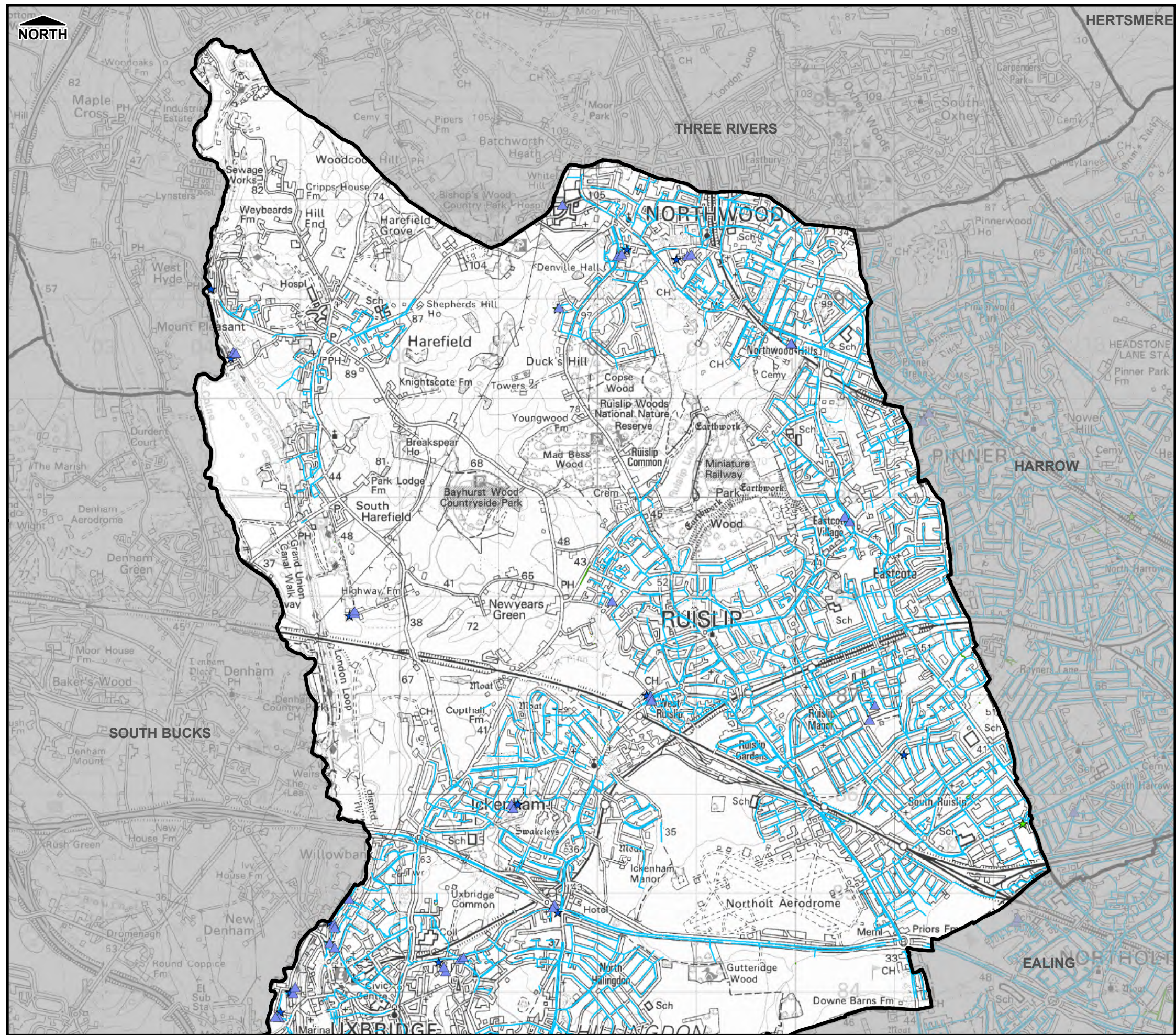
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Tel: (01473) 221660			Email: enquiry@edisurveys.co.uk											
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PROJECT	Job No: 11596													
ROYAL BROMPTON & HAREFIELD HOSPITAL	1000													
CLIENT	Surveyor A/Rose													
DATE	Date Dec. 2009													
CHECKED	Checked RUK													
NOTES: The hereditary contract of this survey is based on the following conditions: 1. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 2. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 3. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 4. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 5. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 6. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 7. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 8. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 9. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions. 10. The Surveyor shall be responsible for the accuracy of the survey, and shall be liable for any errors or omissions.														
DRAINING NO. 11596/T/08B-11														



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330	11596/T/13R-11
331	11596/T/13S-11
332	11596/T/13T-11
333	11596/T/13U-11
334	11596/T/13V-11
335	11596/T/13W-11
336	11596/T/13X-11
337	11596/T/13Y-11
338	11596/T/13Z-11
339	11596/T/14A-11
340	11596/T/14B-11
341	11596/T/14C-11
342	11596/T/14D-11
343	11596/T/14E-11
344	11596/T/14F-11
345	11596/T/14G-11
346	11596/T/14H-11
347	11596/T/14I-11
348	11596/T/14J-11
349	11596/T/14K-11
350	11596/T/14L-11
351	11596/T/14M-11
352	11596/T/14N-11
353	11596/T/14O-11
354	11596/T/14P-11
355	11596/T/14Q-11
356	11596/T/14R-11
357	11596/T/14S-11
358	11596/T/14T-11
359	11596/T/14U-11
360	11596/T/14V-11
361	11596/T/14W-11
362	11596/T/14X-11
363	11596/T/14Y-11
364	11596/T/14Z-11
365	11596/T/15A-11
366	11596/T/15B-11
367	11596/T/15C-11
368	11596/T/15D-11
369	11596/T/15E-11
370	11596/T/15F-11
371	11596/T/15G-11
372	11596/T/15H-11
373	11596/T/15I-11
374	11596/T/15J-11
375	11596/T/15K-11
376	11596/T/15L-11
377	11596/T/15M-11
378	11596/T/15N-11
379	11596/T/15O-11
380	11596/T/15P-11
381	11596/T/15Q-11
382	11596/T/15R-11
383	11596/T/15S-11
384	11596/T/15T-11
385	11596/T/15U-11
386	11596/T/15V-11
387	11596/T/15W-11
388	11596/T/15X-11
389	11596/T/15Y-11
390	11596/T/15Z-11
391	11596/T/16A-11
392	11596/T/16B-11
393	11596/T/16C-11
394	11596/T/16D-11
395	11596/T/16E-11
396	11596/T/16F-11
397	11596/T/16G-11
398	11596/T/16H-11
399	11596/T/16I-11
400	11596/T/16J-11
401	11596/T/16K-11
402	11596/T/16L-11
403	11596/T/16M-11
404	11596/T/16N-11
405	11596/T/16O-11
406	11596/T/16P-11
407	11596/T/16Q-11
408	11596/T/16R-11
409	11596/T/16S-11
410	

Appendix B – LLFA Groundwater, Sewers and Critical Drainage Area Mapping.

Filepath: L:\Environment\zwet\CS046913_DrainLondon_Tier2\Group1\ARC\mxd\GP1_Hillingdon_Fig8.1_TWSEwerNetwork.mxd



THIS DRAWING MAY BE USED ONLY FOR
THE PURPOSE INTENDED

Legend

- Borough Administrative Boundary
- Sewage Treatment Works Locations
- Pumping Stations
- Combined Sewer Overflow Locations
 - Other
 - Pumping Stations
- Combined Sewer
- Surface Water Sewer
- Other Sewers

Notes

- Foul drainage network not displayed;
- Drainage networks with a diameter less than 150mm not displayed.
- The pipe network shown on this drawing is as supplied by Thames Water. Any queries relating to connectivity should go directly to Thames Water.

London Borough of Hillingdon



Surface Water Management Plan

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Scale at A3	Date	Drawn by	Approved by
1:40,000	08/04/2011	R.MOORE	P.HLINOVSKY

Thames Water Sewer Network

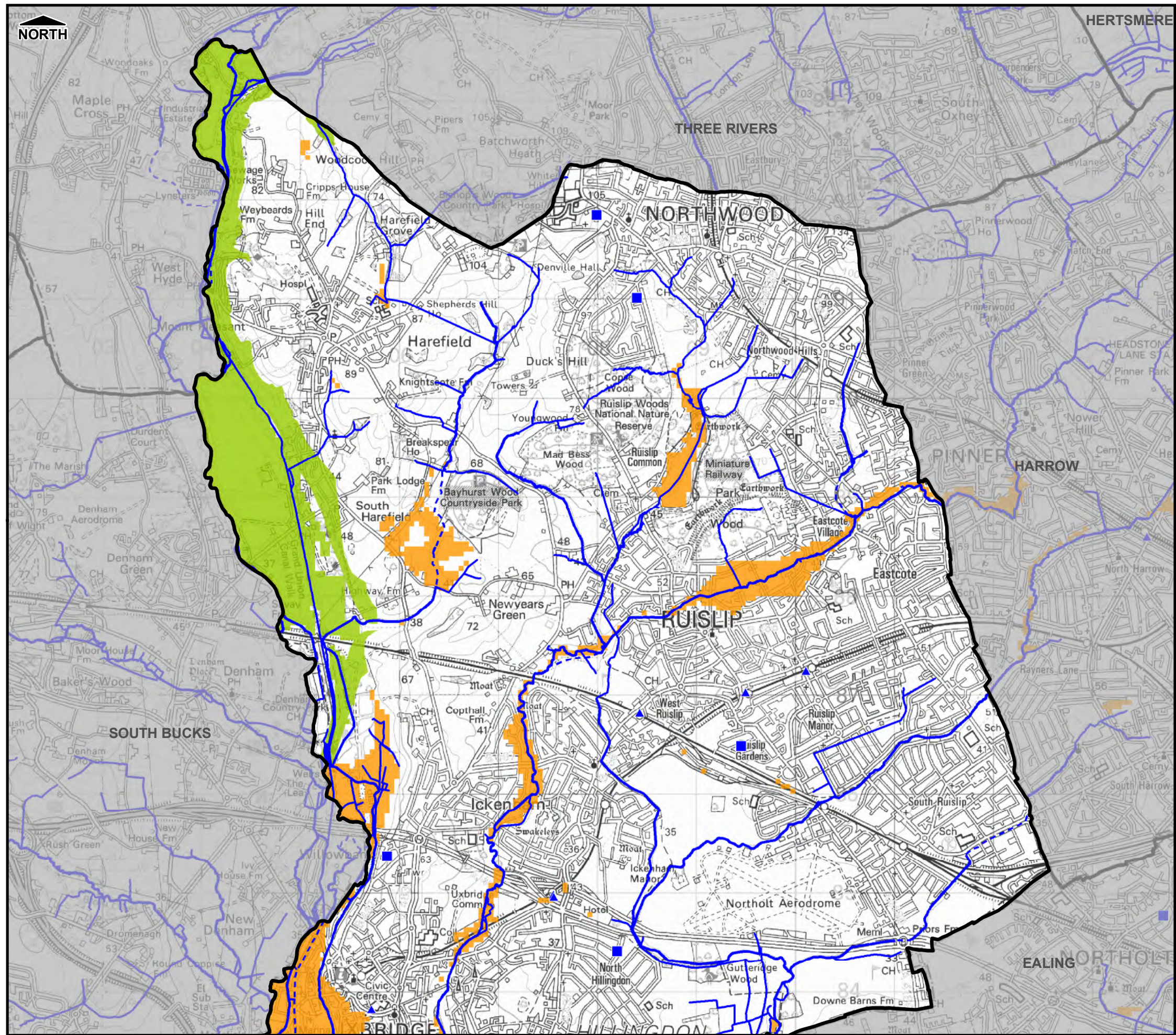
Consultants
CAPITA SYMONDS Capita Symonds
Level Seven,
52 Grosvenor Gardens,
Belgravia,
London
SW1W 0AU

Drain London Programme Board Members



FIGURE 8.1

Filepath: L:\Environment\zwet\CS046913_DrainLondon_Tier2\Group1\ARC\mxd\GP1_Hillingdon_Fig10.1_PotentialElevatedGWMMap.mxd



THIS DRAWING MAY BE USED ONLY FOR THE PURPOSE INTENDED

Legend

Borough Administrative Boundary	Culverted Watercourse (Main River)	
Main River	Groundwater Flood Incident (EA Records)	
Ordinary Watercourse	Groundwater Flood Incident (Other Records)	
Permeable Superficial Deposits	Increased Potential for Elevated Groundwater in	
Consolidated Aquifers		

- Notes**
1. The increased Potential for Elevated Groundwater map shows those areas within the London Boroughs where there is an increased potential for groundwater to rise sufficiently to interact with the ground surface or be within 2 m of the ground surface. Such groundwater rise could lead to the following consequences:
 - flooding of basements of buildings below ground level;
 - flooding of buried services or other assets below ground level;
 - inundation of farmland, roads, commercial, residential and amenity areas;
 - flooding of ground floors of buildings above ground level; and
 - overflowing of sewers and drains.
 2. Incident records shown are generally unconfirmed and may include issues such as water main bursts or non-groundwater related problems.
 3. Areas not shown to have increased potential for elevated groundwater should be considered to have a low potential for elevated groundwater – Lack of information does not imply 'no potential' of elevated groundwater in that area.
 4. Includes groundwater flood mapping provided by JBA Consulting Copyright © Jeremy Benn Associates Limited 2008 - 2011, partially derived from data supplied by the Environment Agency

London Borough of Hillingdon

Surface Water Management Plan

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Scale at A3 1:40,000	Date 05/04/2011	Drawn by R.MOORE	Approved by P.HLINOVSKY
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Increased Potential for Elevated Groundwater Map

Consultants

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Drain London Programme Board Members

GREATERLONDONAUTHORITY

FIGURE 10.1

Appendix C1 – Surface Water Modelling

Introduction

Capita Symonds has constructed seven TUFLOW hydraulic models across the London Boroughs in Group 1. The extents of the models have generally been based upon catchment boundaries and not borough boundaries to limit the amount of cross-boundary interaction between models. This was carried out to limit the dependency of one model on the results of another. Consequently, the model results for each borough are divided over a number of models and in some cases have been modelled by more than one consultant. The following table outlines the models that cover the London Borough of Hillingdon, along with the name of the final model, percentage coverage of the Borough by each model, and the names of any other Boroughs falling within the model extent. Figure 1 shows the extent of the models listed.

Table 1: Model coverage for the London Borough of Hillingdon

Consultant	Model Name	Naming Convention (100 year Flood Event)	Borough Coverage	Other Boroughs covered by the Model
Capita Symonds	Harefield	DLT2_G1HF_0100R_026	16%	-
Capita Symonds	Ruislip	DLT2_G1RS_0100R_026	20%	Harrow
Capita Symonds	Harrow	DLT2_G1HW_0100R_030	25%	Ealing, Ealing
Capita Symonds	West Drayton	DLT2_G1WD_0100R_025	17%	-
Capita Symonds	Hounslow	HO_I_100yr_5m_1p25s_03	1%	Hounslow and Ealing
Capita Symonds	Heathrow	HR_1_100yr_5m_1p5s_01	21%	Hounslow

The naming convention has generally been derived to reference the tier of work, the name of the model, the flood event being modelled and the version number. A standard naming convention was not adopted for all models built for the Drain London project, hence different conventions may have been adopted by other consultants.

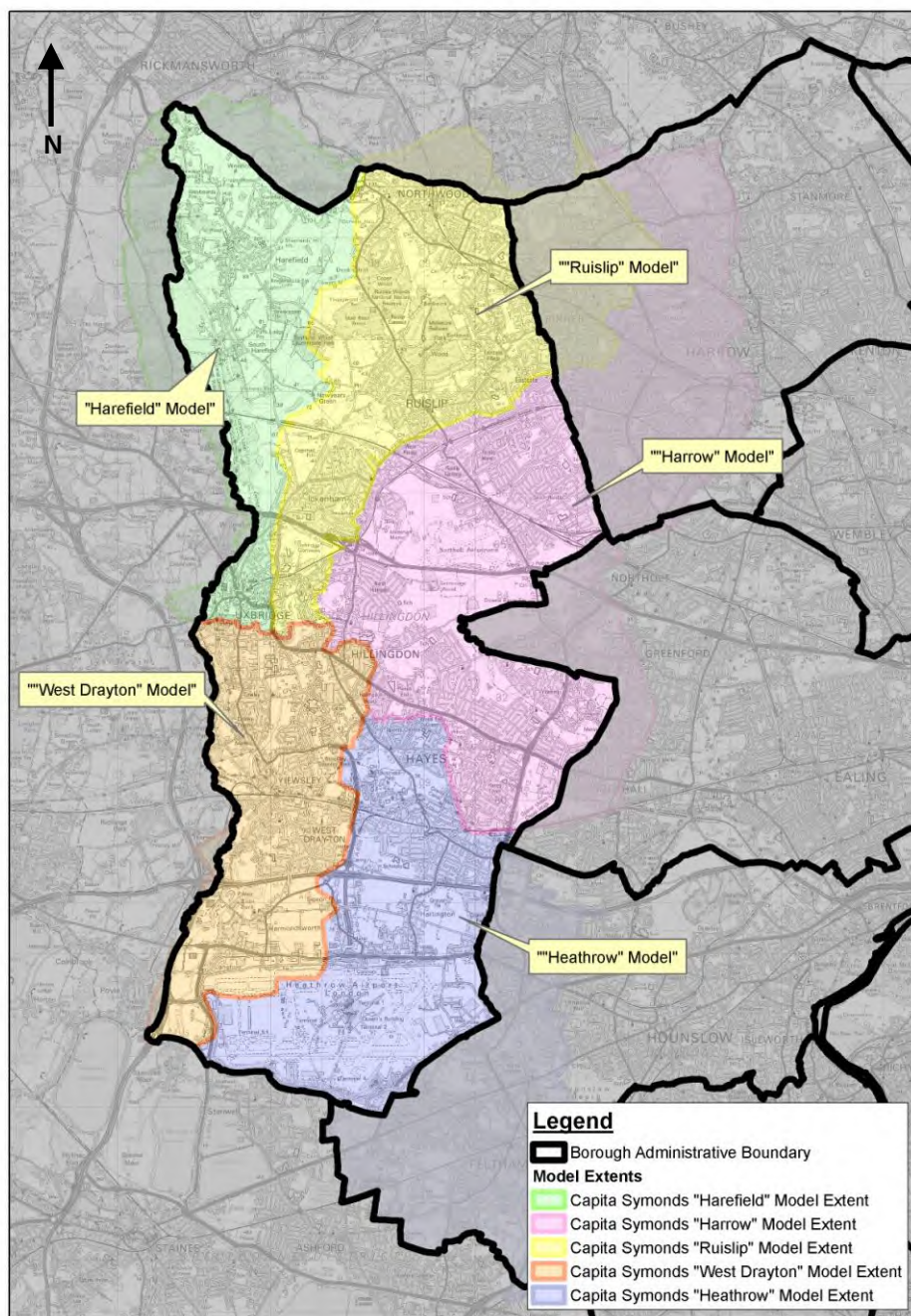


Figure 1: Model coverage for the London Borough of Hillingdon

Software Version

All models have been run using TUFLOW build 2010-10-AA-iDP as agreed by all Drain London consultants using the TUFLOW software. All models within the Borough of Hillingdon

were run on the 64bit version of this build to take advantage of the faster simulation times and more advanced handling of larger models.

Model Parameters

All hydraulic models have been constructed following the guidance outlined in the Drain London: Data and Modelling Framework V1.0 (December 2010). The following sections of this appendix describe in more detail how this guidance was applied and where amendments or additions were made.

Direct Rainfall Methodology

The Drain London modelling was designed to analyse the impact of heavy rainfall events across each London Borough by assessing flow paths, velocities and catchment response. The Drain London Data and Modelling Framework specified that the direct rainfall method should be used in the modelling approach. This method incorporates conservative allowances for the drainage network and infiltration. The following key assumptions were made to generate the model input:

- Initial Loss – None
- Infiltration Loss – None
- Allowance for Drainage System – A constant value of 6.5mm/hr was applied
- No aerial reduction factor applied
- 'Summer' rainfall profile was used

To comply with the Drain London framework requirements rainfall inputs were generated at a standard 10km grid square resolution. As specified in the framework guidance hyetographs for the following rainfall events were generated:

- 1 in 30 year
- 1 in 75 year
- 1 in 100 year
- 1 in 100 year plus climate change (+30%)
- 1 in 200 year

Total rainfall depths at each 10km grid centroid for all required return periods were extracted from the FEH CD-ROM (v3) Depth Duration Frequency (DDF) model. A comparison between the peak rainfall depths in adjacent 10km grid squares was completed to confirm the suitability of the 10km grid resolution for modelling purposes. The difference in total rainfall depths between the grid centroids for 10km grid squares was mostly less than 5%, with the maximum difference being 17%. This suggests that the 10km grid data is suitable for use in the study as a finer grid would have a minimal effect on the hyetographs.

Critical duration is a complex issue when modelling large areas for surface water flood risk. The critical duration can change rapidly even within a small area, due to the topography, land use, size of the upstream catchment and nature of the drainage systems. The ideal approach would be to model a wide range of durations. However, this is not always practical or economic when modelling large areas using 2D models which have long simulation times – such as within the Drain London study.

A high level investigation was undertaken to understand the effect of rainfall event duration on the Drain London Study area using a rapid modelling technique. The intention of the

investigation was to show variation in critical duration across the study area and thus identify whether it was possible to identify single critical durations for each sub-model. The study used the 1 in 100year hyetographs for 1, 3, 6 and 12 hour durations along with a simplified terrain model to route overland flow. The key result was that critical duration is highly variable across surface water catchments – but the influence was not sufficiently significant to justify considering multiple event durations within the Drain London Study. Therefore, a single duration of 3hrs was selected for all model runs to ensure result consistency and comparability across the Greater London area. It is strongly recommended that an analysis of possible result sensitivity to duration is considered for future studies.

Grid Size

All models within the boundary of the London Borough of Hillingdon have been constructed with a 5m grid size, within the recommended range detailed in the Data and Modelling Framework. This grid size was chosen as it represented a good balance between the degree of accuracy (i.e. ability to model overland flow paths along roads or around buildings) whilst maintaining reasonable model run (“simulation”) times. For example, refining the grid size from a 5m grid to a 2m grid is likely to increase the model simulation time from 21 hours to approximately 11 days.

Structures

Structures within the study area were generally modelled in 2D, an approach consistent with the strategic nature of the Drain London project. Structures modelled in 2D include those on watercourses and underpasses or culverts within the floodplain. The structures were modelled by using the ZSHP function in TUFLOW which allows the user to specify the object width representing the structure opening. Invert levels were determined by inspecting the LiDAR DTM with widths of structures either measured on site visits, from Google Maps, or from the LiDAR DTM.

The limitations of modelling structures in 2D, rather than as a 1D element, are that the width of the structures is limited by the grid size (i.e. structure width is a multiple of the grid size). The depth of water within the structure can also be over-estimated as rainfall is allowed to enter the structure from above and not just through the entrances of the structure. For this reason, only short structures (e.g. generally less than 40m) have been modelled in 2D.

Adjustments to Topography

When reviewing the model's representation of the LiDAR DTM, it was observed in some locations of new development that excavation pits had been captured by the DTM whereas aerial photos showed buildings. Where this occurred in critical areas of the model or where the pits were particularly large, these were manually filled in to match the elevation of surrounding areas.

Building Footprints

Building footprints have been largely represented in the model as outlined in the Data and Modelling Framework. In situations where the polygon representing the building was large or long, the use of a single elevation to represent the floor level resulted in parts of the building being raised metres above the surrounding ground level. This can therefore misrepresent the potential for the building to flood. In these cases, the building ‘polygon’ was assigned a

varying elevation such that the finished floor level remained 100mm above the ground level across the area of the polygon.

Runoff Coefficients

The runoff coefficients applied to the hydraulic models were in line with those stated in the Drain London Data and Modelling Framework. The runoff coefficients were applied to the rainfall profiles in order to represent the varying level of infiltration on each surface, therefore altering the input data directly.

Formal and Informal Defences

A GIS layer containing defences from the Environment Agency's NFCDD dataset was provided. These defences have been included in all models. Where additional data was provided by the Borough or informal defences such as walls were observed on site or through Google Maps, these were included in the model where it was thought that their presence would influence surface water flowpaths. No such defences were defined within the Borough of Hillingdon.

Model Boundaries

Downstream boundaries in the models were included where it was observed that water was able to flow outside of the model extent. This might include locations where water was found to flow into a neighbouring catchment or model. The type of downstream boundary generally used was a flow vs. stage (level) relationship, or HQ boundary. These were applied to the 2D component of the models only. The rating relationship is generated by TUFLOW automatically using a gradient provided by the modeller. The 'outflow' from the model is then applied as an 'inflow' to the downstream model.

There are a number of 2D downstream boundaries included within all models within Hillingdon; these have mainly been located on roads that make up overland flow routes for the surface water.

Cross-Boundary Issues

In some cases, it was not possible to avoid interaction with a neighbouring model due to the nature of the topography. Consequently, the extents for a few of the models with the London Borough of Hillingdon extent overlap. This was carried out to ensure that the entirety of the overland flow into the sub-catchment is modelled and that the model results would not be sensitive to the model extents. Within the overlap, the results of both models were compared. In the majority of cases, the results were found to be very similar, in both maximum depth and in flood outline. In cases where there was a notable difference, the results were analysed to determine from which catchment the surface water originated. The results from this model were then deemed to take priority. Table 4 below lists the priority of models within the overlapping areas:

Table 4: Priority levels of models within the London Borough of Hillingdon

Location of overlap	NGR	Models Involved	Model Priority
Between the junction of Airport Way and Sprout	504520, 175190	West Drayton Heathrow	West Drayton

Location of overlap	NGR	Models Involved	Model Priority
Lane and Beavers Lane Camp			
Hillingdon Golf Course	506600, 183340	Ruislip West Drayton	Ruislip

Simulation Time

All models were initially run for six hours in compliance with the Data and Modelling Framework document. The models were then assessed to determine whether this duration was suitable for each specific model. This was carried out by viewing the model results for the final few timesteps. The results were checked to determine if water depths were still increasing significantly, and whether new flowpaths were forming or existing flowpaths still propagating. If either of these conditions were found to exist, the simulation time was extended for a further hour after which the checks were repeated until none of the conditions were satisfied. The simulation times for each of the models within the London Borough of Hillingdon have been listed below in Table 5 (overleaf):

Table 5: Model simulation times

Model Name	Model Simulation Time (hrs)
DLT2 G1HF 0100R 026	6
DLT2 G1RS 0100R 026	7
DLT2 G1HW 0100R 030	6
DLT2 G1WD 0100R 025	6
HR 1 100yr 5m 1p5s 01	6
HO 1 100yr 5m 1p25s 03	6

Sensitivity Testing

The sensitivity of the model results to changes in drainage loss was tested. This was carried out for all models on the 1 in 200 year return period flood event. The original drainage loss of 6.5mm/hr was adjusted by +/-25% giving values of 8.125mm/hr and 4.875mm/hr to be used for the analysis. The two sensitivity test results were compared with the baseline results by producing a depth difference grid. This output shows the difference in depth as a result of the change in drainage loss. The model results are deemed to be sensitive to changes in the tested parameter, if the percentage change in depth is greater than the percentage change in the parameter.

As a whole, the models within the Borough of Hillingdon were not found to be sensitive to changes in drainage loss. Changes in maximum depth were less than 25% compared to the baseline results. A number of intermittent locations in the model did show a larger change in depth. These were generally located in areas where there are sudden changes in elevation, i.e. at railway cuttings.

Model Stability

Assessing the stability of a model is a critical step in understanding the robustness of a model and its ability to simulate a flood event accurately. Stability in a TUFLOW model can be

assessed by examining the cumulative error (or mass balance) of the model as well as the warnings outputted by the model during the simulation.

As can be seen in Figures 3 to 8 below, the cumulative error of the models in the Borough is largely within the recommended range of $\pm 5\%$ for the majority of the simulation. High values are reported at the beginning of the rainfall event when the model cells first wet then settle down for the remainder of the simulation. The cause and location of the high cumulative errors was investigated by examining a number of other output files provided by TUFLOW. The high values were found to occur at random locations throughout the study area for a single timestep and were not found to persistently occur at a single cell. This suggests that the high cumulative error is a consequence of the initial wetting process at the start of the rainfall event. The high cumulative error values are therefore considered to have a negligible impact on the overall model results.

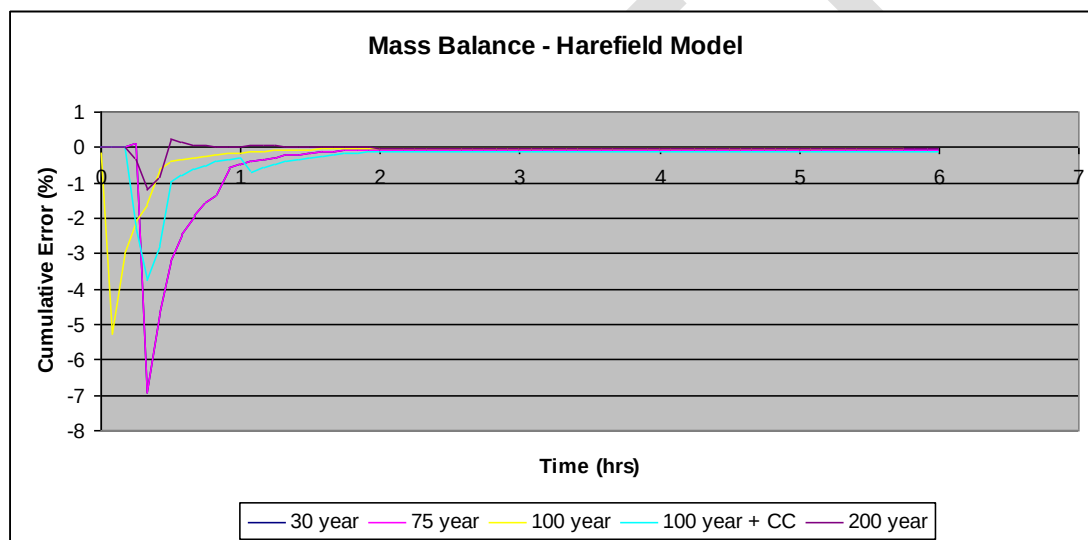


Figure 3: Mass Balance of Harefield Model

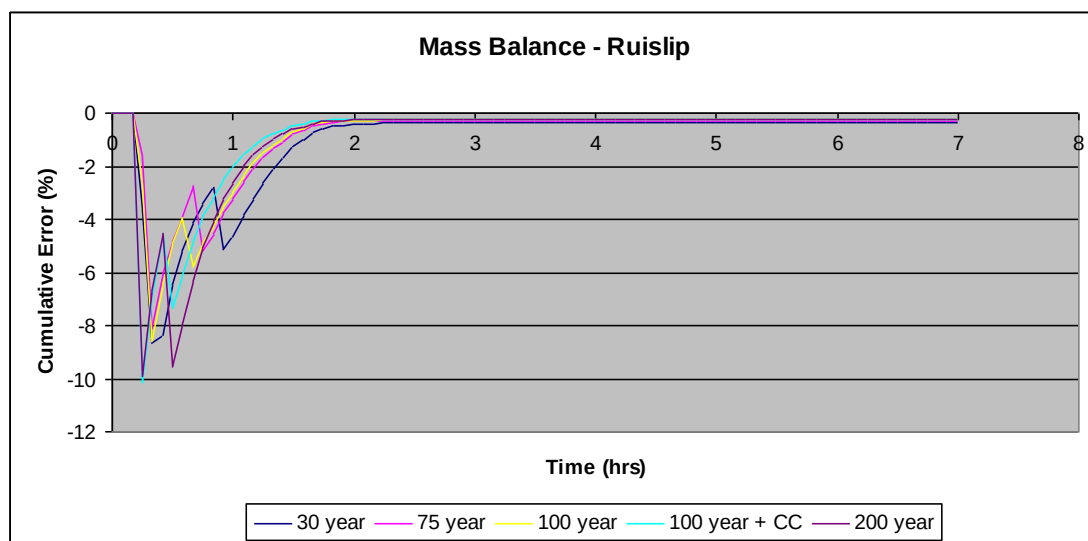


Figure 4: Mass Balance of Ruislip Model

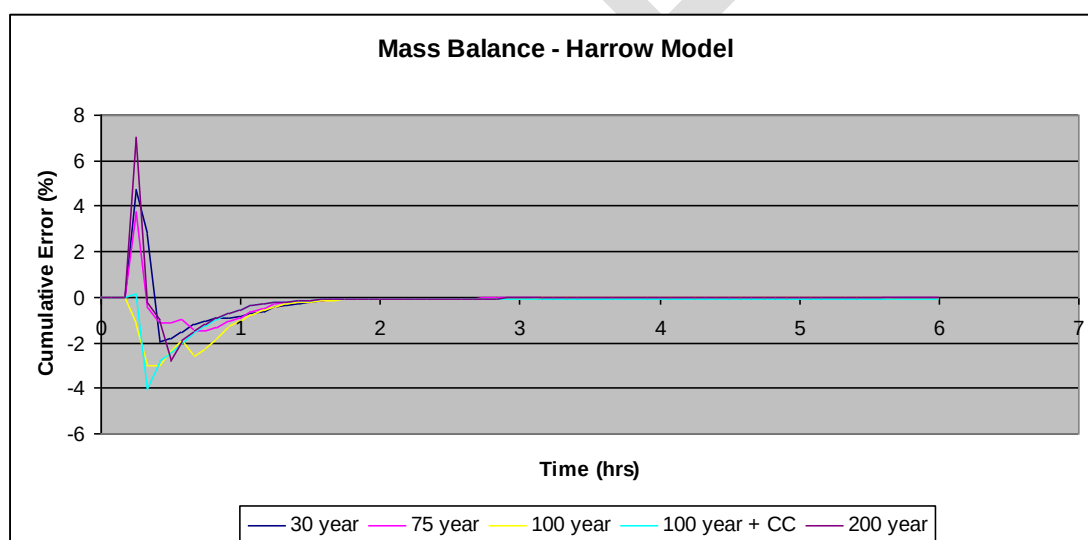


Figure 5: Mass Balance of Harrow Model

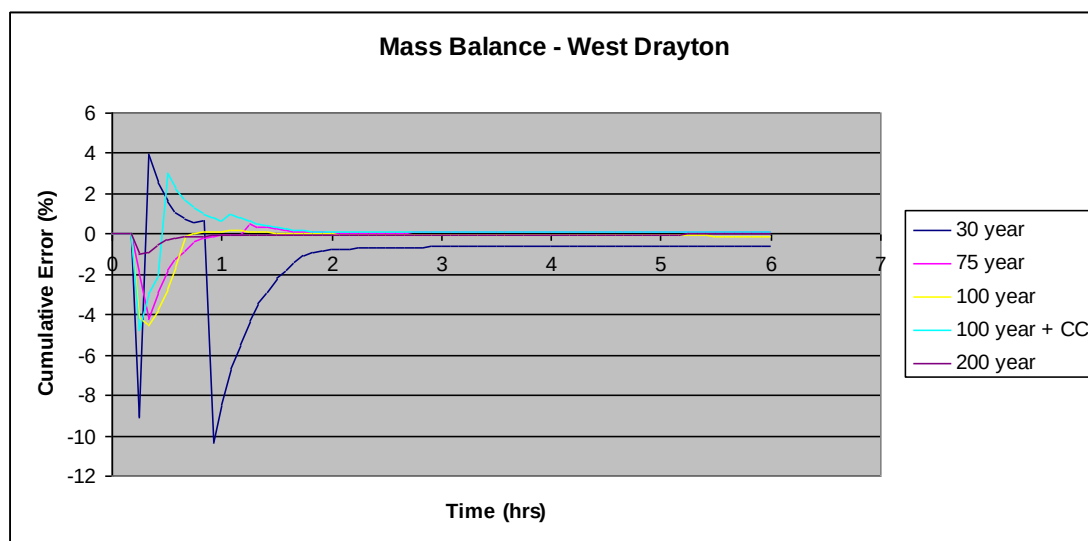


Figure 6: Mass Balance of West Drayton Model

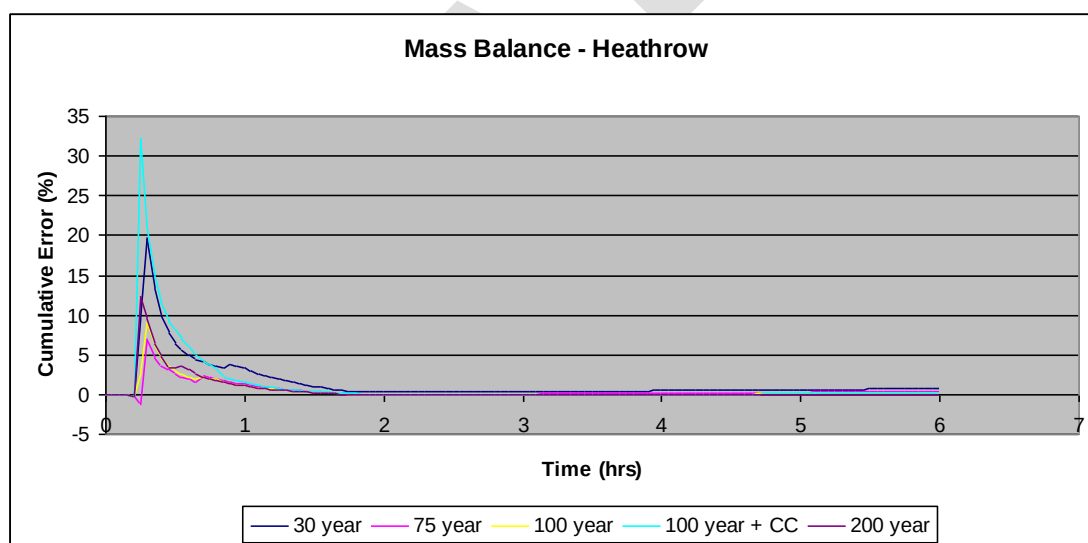


Figure 7: Mass Balance of Heathrow Model

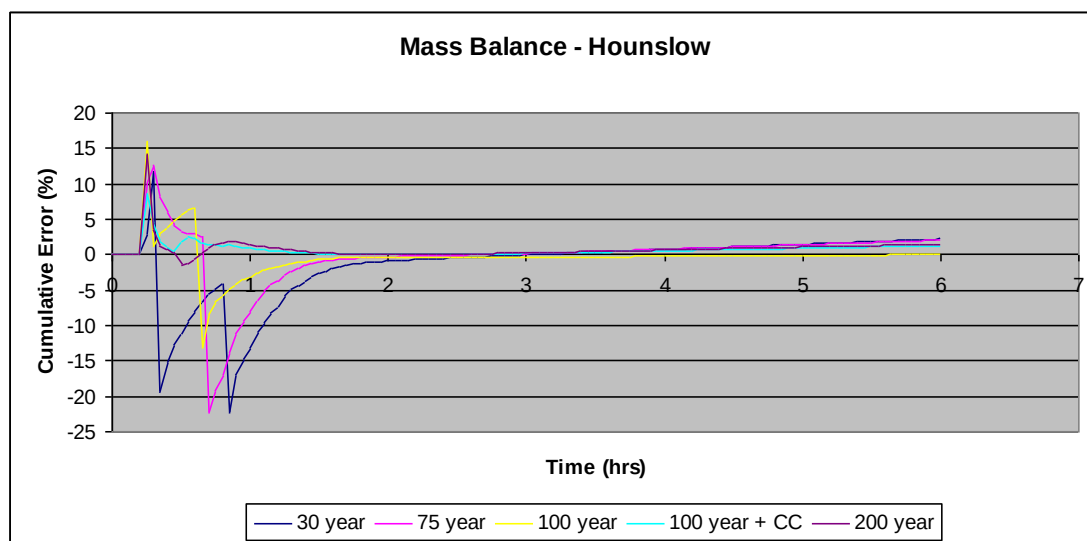


Figure 8: Mass Balance of Hounslow Model

A number of warnings occur in all models, with between 30 and 50 messages outputted during the simulation for each of the five flood events. The warnings relate to areas of poor convergence, or in other words, where TUFLOW has had trouble finding a solution. The warnings were found to be spatially varied and non-persistent in time, which is a relatively common occurrence in these types of models. As the warnings were not found to repeatedly occur, these have a negligible impact on the overall model results and the model is considered fit for purpose.

Conclusions and Recommendations

The hydraulic models constructed for Phase 2 of the Drain London project represent a strategic approach to identify areas at risk of surface water flooding. It represents a significant refinement on the previously available information on surface water flooding in Hillingdon. The models and their mapped results should only be used after a thorough review of this technical appendix and the Drain London Data and Modelling V1.0 (December 2010). Recommendations for future improvements to the models include (but are not limited) to the following:

- Explicitly model the existing drainage network in key areas of risk, as opposed to a London wide assumption on drainage capacity;
- Inclusion of survey data for critical structures;
- Inclusion of river flows and channel capacity (where applicable);
- Reduction in model grid size in key areas of risk;
- Testing of different storm durations;
- Inclusion of defacto defences outside of the scope of the Drain London project (e.g. assets identified through the Asset Register process); and
- The use of better quality or more up to date topographic information particularly in areas of recent development

Appendix C2 – Groundwater

Introduction

As part of the Drain London project Drain London Tier 1 consultants commissioned Jacobs/JBA to produce a dataset referred to as the Increased Potential Elevated Groundwater (iPEG) maps. The assessment was carried out at a Greater London scale. The iPEG mapping assists in identifying areas which have an increased potential to experience groundwater flooding. The iPEG map shows those areas within the borough where there is an increased potential for groundwater to rise to within 2m of the ground surface. When groundwater rises to this level water may be able to enter below ground structures such as basements and communications networks and continue rising to cause surface water flooding. The iPEG map includes an assessment of the potential groundwater to rise in both consolidated aquifers and from superficial permeable deposits (unconsolidated aquifers). The map also includes those areas close to rivers which are underlain by permeable superficial deposits where groundwater may rise to elevated levels driven by high water levels in the river.

Methodology

Large areas within the Drain London area are underlain by permeable substrate and thereby have the potential to store groundwater. Under some circumstances groundwater levels can rise and cause flooding problems in subsurface structures or at the ground surface. The mapping technique described below aims to identify only those areas in which there is the greatest potential for this to happen.

Four data sources have been utilised to produce the increased Potential for Elevated Groundwater map. These data sources are the:

- British Geological Survey (BGS) Groundwater Flood Susceptibility Map;
- Jacobs Groundwater Emergence Maps (GEMs);
- Jeremy Benn Associates (JBA) Groundwater Flood Map; and
- Environment Agency/Jacobs Thames Estuary 2100 (TE2100) groundwater hazard maps.

To produce the iPEG map for consolidated aquifers, an area was defined as having increased potential for elevated groundwater levels if at least two of the three mapping techniques listed above produced a corresponding area. For the permeable superficial deposits, only Band 1 Very High of the BGS and the TE2100 data were used as this was judged to best represent the hazard.

A description of each of the four data sets and how it was used in the production of the iPEG map is summarised in Table 1 below. The iPEG map should be viewed with careful consideration of the strengths and disadvantages of each of the four data sets.

Table 1 Summary of Data Used in the Production of the iPEG Map

	BGS Groundwater Flood Susceptibility Map	Jacobs Groundwater Emergence Map	JBA Groundwater Flood Map	Jacobs TE2100 Groundwater Maps
Mechanisms considered hydrogeological coverage /	Clearwater flooding through all consolidated aquifers and groundwater flooding through Permeable Superficial Deposits (PSD)	All major consolidated aquifers	Unconfined Chalk and Permeable Superficial Deposits	Groundwater emergence in Permeable Superficial Deposits in hydrological continuity with river levels.
Methodology	- Identify from geology where groundwater flooding could not occur - For all other areas, produce a groundwater level surface from National Groundwater Level data, modified to best represent groundwater flooding - Compare the groundwater level surface with the DTM and determine susceptibility to groundwater flooding based on depth to groundwater	Three scenarios: - Where flooding was reported and groundwater contours were available, groundwater emergence zones were defined such that they encompassed incidents of observed flooding. - Where no flooding was reported or no data supplied, but groundwater contours were available, then groundwater emergence zones were based on generalised aquifer properties and observation borehole levels. - Where no groundwater contour information was available, river network classified by BFIHOST was used to identify susceptible areas	For the Chalk maps: - Develop water level – frequency relationships at available boreholes - Extrapolate this relationship to ungauged locations - Compare water level surface with DTM for mapped events	- Identify from geology areas of permeable superficial deposits - Identify mean water level in the Thames Estuary (and tidal watercourses) which will drive the groundwater head - Determine likely distance from the estuary (and tidal watercourses) over which groundwater levels could be influenced - Identify areas where the groundwater level could rise to the level in the estuary and be within 2m of the ground surface

	BGS Groundwater Flood Susceptibility Map	Jacobs Groundwater Emergence Map	JBA Groundwater Flood Map	Jacobs TE2100 Groundwater Maps
Data used in the production of the maps	BGS 1:50 000 geological mapping, with classifications of permeability, NextMap 5m DTM, National Groundwater Level data on a 50m grid.	50m resolution IHDTM; groundwater contour data from EA and BGS for all major aquifer units from various dates; borehole level data; recorded observations of groundwater flooding from 2000/1.	Borehole records from the EA; 5m DTM from Infoterra and 1:625 000 scale geological mapping	BGS 1:50 000 geological mapping, LiDAR data at 2m resolution and information on mean water levels and defence crest heights.
Strengths	• Considers consolidated and superficial aquifers • Based on National Groundwater Level data • Calibrated on winter 2000/1 observations of flooding • Provides number of classes of susceptibility to indicate sensitivity • Could select only highest susceptibility bands	• Calibrated on winter 2000/1 observations of flooding	• Provides explicit representation of 1 in 100 chance outline • Provision of up to three event probabilities could enable sensitivity testing • Calibrated on winter 2000/1 observations of flooding	• Considers an important mechanism not considered by other methods • Important mechanism in east London.
Disadvantages	• Outlines are not explicitly linked to event probabilities • Maps may indicate overly-large areas as susceptible to groundwater flooding	• Does not consider PSD • Outlines are not explicitly linked to event probabilities • Regional scale	• PSD map based on 1:50k background.	• Determination of distance from estuary over which groundwater levels could be influenced could be improved • Could consider an upward slope on groundwater levels away from the estuary

How to Use and Interpret the Map

The increased Potential for Elevated Groundwater map shows those areas within the borough where there is an increased potential for groundwater to rise sufficiently to interact with the ground surface or be within 2 m of the ground surface.

Groundwater may become elevated by a number of means:

- Above average rainfall for a number of months in Chalk outcrop areas;
- Shorter period of above average rainfall in permeable superficial deposits;
- Permeable superficial deposits in hydraulic continuity with high water levels in the river;
- Interruption of groundwater flow paths; and
- Cessation of groundwater abstraction causing groundwater rebound.

With the exception of groundwater rebound which is not covered, the iPEG map will identify those areas most prone to the mechanisms described above. The map shows those areas considered to have the greatest potential for elevated groundwater. Additional areas within the London Boroughs have permeable geology and therefore could also produce elevated groundwater levels. However, to produce a realistic map, only where there is the highest degree of confidence in the assessment are the areas delineated. This ensures resources are focused on the most susceptible areas. In all areas underlain by permeable substrate, groundwater should still be considered in planning developments.

Within the areas delineated, the local rise of groundwater will be heavily controlled by local geological features and artificial influences (e.g. structures or conduits) which cannot currently be represented. This localised nature of groundwater flooding compared with, say, fluvial flooding suggests that interpretation of the map should similarly be different. The map shows the area within which groundwater has the potential to emerge but it is unlikely to emerge uniformly or in sufficient volume to fill the topography to the implied level. Instead, groundwater emerging at the surface may simply runoff to pond in lower areas.

For this reason within iPEG areas, locations shown to be at risk of surface water flooding are also likely to be most at risk of runoff/ponding caused by groundwater flooding. Therefore the iPEG map should not be used as a “flood outline” within which properties at risk can be counted. Rather it is provided, in conjunction with the surface water mapping, to identify those areas where groundwater may emerge and if so what would be the major flow pathways that water would take.

- Railway Embankments - discrete surface water flooding locations along the up-stream side of the raised network rail embankment (running roughly west to east through the South of the Borough);
- Topographical Low Points – areas which are at topographical low points throughout the Borough which result in small, discrete areas of deep surface water ponding; and
- Sewer Flood Risk – areas where extensive and deep surface water flooding is likely to be the influence of sewer flooding mechanisms alongside pluvial and groundwater sources.

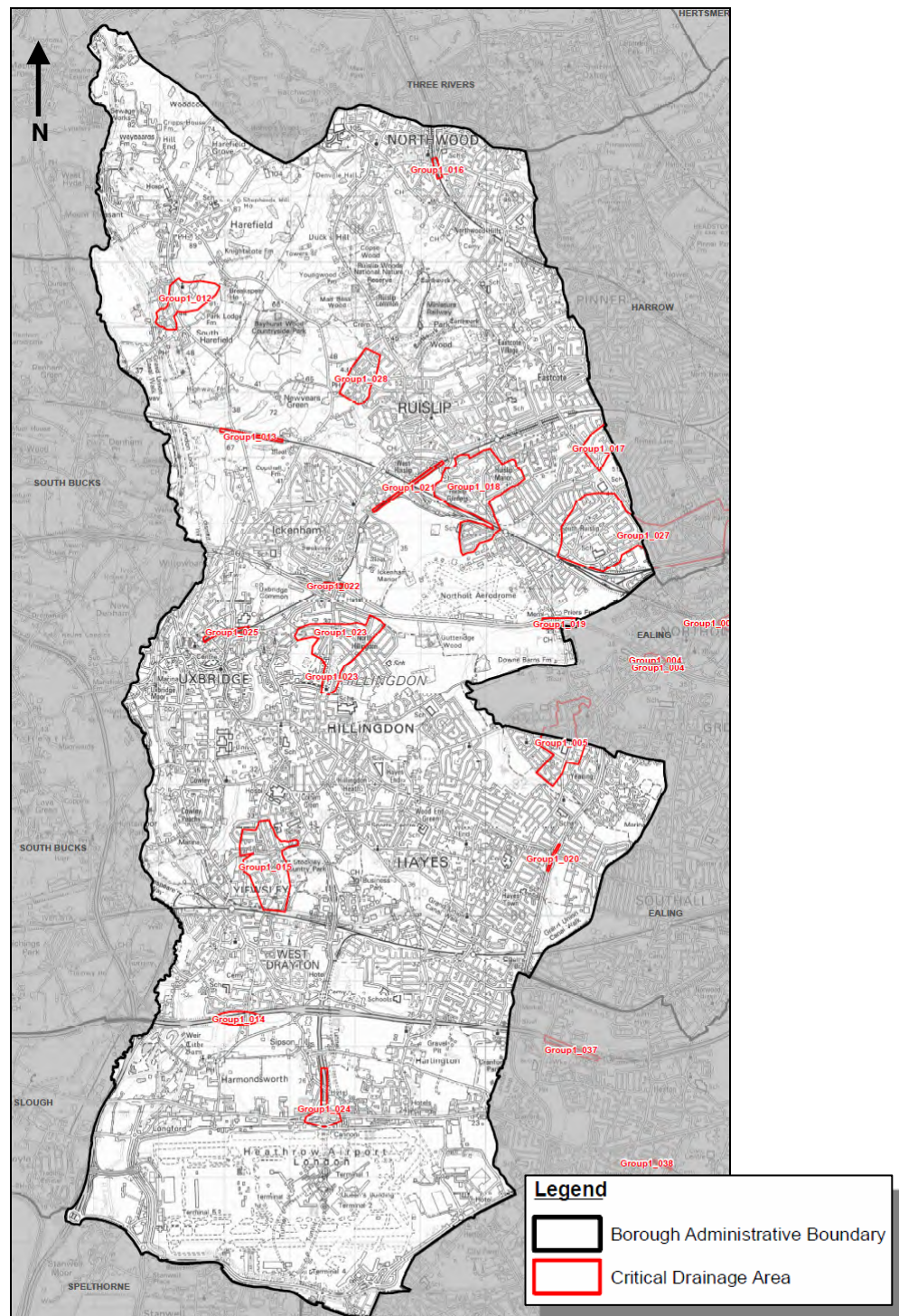


Figure i. Critical Drainage Areas within the London Borough of Hillingdon.

Appendix C – Proposed Drainage Strategy.

[illegible]

DO NOT SCALE FROM THIS DRAWING

1. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
2. THIS DRAWING IS FOR INFORMATION ONLY.
3. DRAWINGS ARE TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS, ENGINEERS AND CONSULTANTS DRAWINGS AND SPECIFICATIONS.
4. THIS DRAWING IS BASED ON:
 - RSP ARCHITECTURAL DRAWINGS
2607-SK-S1-00-DR-E-1000S AND 2607-SK-S1-DR-E-1000E DATED AUGUST 2024.
 - TOPOGRAPHIC AND UTILITIES SURVEY DRAWINGS
11596/T018-11 DATED DECEMBER 2009.
5. THE PROPOSED SURFACE WATER DRAINAGE ON THE SITE IS SUBJECT TO APPROVAL BY THE ROYAL BOROUGH OF HILLINGDON.

	SITE BOUNDARY
	PROPOSED SURFACE WATER SEWER
	EXISTING SURFACE WATER SEWER
	EXISTING FOUL WATER SEWER
	PROPOSED BELOW GROUND SURFACE WATER ATTENUATION
	SURFACE WATER GULLY
	PETROL INTERCEPTOR
	PROPOSED SURFACE WATER CHAMBER
	EXISTING SURFACE WATER MANHOLE
	EXISTING FOUL WATER MANHOLE

EXISTING DRAINAGE TO BE ABANDONED TO MAKE WAY OF THE PROPOSED BUILDING IN THIS AREA. PROPOSED BUILDING LOCATION TO BE CONFIRMED AT LATER DESIGN STAGES.

NOTE
PROPRIETARY SYSTEMS, BUNDING, OR SPILL KITS ARE PROPOSED TO ENHANCE SURFACE WATER QUALITY PRIOR TO SURFACE WATER DISCHARGE INTO THE SOAK-AWAY. THE MOST SUITABLE OPTION TO BE FINALIZED AS DESIGN DEVELOPMENT PROGRESSES IN SUBSEQUENT DESIGN STAGES.

PROPOSED INFILTRATION AREA
BENEATH AND AROUND
PROPOSED SWITCH ROOM SLAB
CL: VARIES, REFER TO PROPOSED
LEVELS DESIGN BY OTHERS.
AREA: 28.08m²
DEPTH: 0.2m BELOW THE SLAB
AND 0.3m AROUND THE SLAB.
VOLUME: 1.851m³
VOID RATIO: 0.3

PROPOSED
LOCATION.

SURROUNDING AREAS MAY REQUIRE LEVELS DESIGN TO ENSURE CONSISTENT FALLS FOR POSITIVE DRAINAGE TO THE INFILTRATION AREAS WITHIN THE RED LINE BOUNDARY. LEVELS DESIGN TO BE DONE BY OTHERS

PROPOSED INFILTRATION AREA
BENEATH AND AROUND
PROPOSED GENERATOR SLAB
CL: VARIES, REFER TO PROPOSED
LEVELS DESIGN BY OTHERS.
AREA: 39.022m²
DEPTH: 0.2m BELOW THE SLAB
AND 0.3m AROUND THE SLAB.
VOLUME: 2.530m³
VOID RATIO: 0.3

PLEASE REFER TO STRUCTURAL ENGINEER'S
DRAWINGS AS WELL AS GEO-TECHNICAL
ENGINEER'S RECOMMENDATIONS FOR
FURTHER DETAILS ON INFILTRATION
GRAVEL/FOUNDATION ARRANGEMENT.

PROPOSED INFILTRATION AREA
BENEATH AND AROUND
PROPOSED GENERATOR SLAB
CL:VARIES, REFER TO PROPOSED
LEVELS DESIGN BY OTHERS.
AREA: 39.022m²
DEPTH:0.2m BELOW THE SLAB
AND 0.3m AROUND THE SLAB.
VOLUME:2.53m³
VOID RATIO:0.3

PROPOSED INFILTRATION
AREAS FOR SURFACE
WATER DRAINAGE TO BE
FULLY COORDINATED
WITH PROPOSED AND
EXISTING SERVICES IN
THIS AREA.
COORDINATION TO BE
DONE BY OTHERS.

PROPOSED INFILTRATION AREA
BENEATH AND AROUND
PROPOSED BULK FUEL TANK
SLAB
CL-VARIES, REFER TO PROPOSED
LEVELS DESIGN BY OTHERS.
AREA: 20.08m²
DEPTH: 0.2m BELOW THE SLAB
AND 0.3m AROUND THE SLAB.
VOLUME: 1.350m³
VOLUME RATIO: 0.3

1. DESIGN SUBJECT TO COUNCIL APPROVAL OF BY THE ROYAL BOROUGH OF HILLINGDON.
2. ALL COVER LEVELS SUBJECT TO CHANGE FOLLOWING FURTHER CONSULTATIONS WITH LANDSCAPE ARCHITECTS AS LEVELS MAY REQUIRE MODIFICATION TO ENSURE POSITIVE DRAINAGE TOWARDS THE INFILTRATION AREAS. LEVELS DESIGN TO BE DONE BY OTHERS.
3. ALL EXISTING SURFACE WATER DRAINAGE ON THE SITE IS ASSUMED TO BE DISCHARGING INTO THE SOAK-AWAY BETWEEN THE HEART SCIENCE BUILDING AND THE TEMPORARY WARDS. PROPOSED LOCATIONS OF CONCRETE BASES FOR MOUNTING THE PROPOSED GENERATORS TO BE FULLY COORDINATED WITH THE PROPOSED SURFACE WATER NETWORK. COORDINATION TO BE DONE BY OTHERS.
4. PROPOSED SURFACE WATER INFILTRATION IN CLOSE PROXIMITY TO EXISTING DRY UTILITIES. CONSULTATIONS WITH THE RELEVANT AUTHORITIES WILL BE REQUIRED PRIOR TO CONSTRUCTION WORKS ONSITE. CONSULTATIONS TO BE DONE BY OTHERS.
5. THE INTERACTION OF THE EXISTING HOSPITAL SEWER NETWORK WITH ANY PUBLIC THAMES WATER SEWERS IS CURRENTLY UNKNOWN.

1. STORAGE VOLUME HAS BEEN PROVIDED WITHIN THE INFILTRATION AREAS TO ACCOMMODATE ALL RAINFALL EVENTS UP TO AND INCLUDING THE 1 IN 100-YR PLUS 40% CLIMATE CHANGE ALLOWANCE RAINFALL EVENT.
2. SURFACE WATER DRAINAGE FLOWS TO BE MANAGED NEAR SOURCE THROUGH INFILTRATION INTO THE GROUND IN LINE WITH THE REQUIREMENTS OF THE LONDON PLAN DRAINAGE POLICY.

PROJECT
HAREFIELD HOSPITAL

TITLE
PROPOSED BELOW GROUND DRAINAGE
STRATEGY (INFILTRATION INTO THE GROUND)
SHEET 1 OF 1

DISCIPLINE CIVIL			SCALE 1:200
DRAWN CM	DESIGNED CM	CHECKED EV	APPROVED EV
DRAWING No 4279-MHT-CV-XX-DR-0500			ISSUE 103

Appendix D – SuDS Proforma.

The London Sustainable Drainage Proforma

Introduction

This proforma is intended to accompany a drainage strategy prepared for a planning application where required by national or local planning policy. It should be used to summarise the key outputs from the strategy to allow assessing officers at the Lead Local Flood Authority (LLFA) to quickly assess compliance with sustainable drainage (SuDS) planning

The proforma is divided into 4 sections, which are intended to be used as follows:

1. Site and project information - Provide summary details of the development, site and drainage
2. Proposed discharge arrangement – Summarise site ground conditions to determine potential for infiltration. Select a surface water discharge method (or mix of methods) following the hierarchical approach set out in the London Plan.
3. Drainage strategy – Prioritise SuDS measures that manage runoff as close to source as possible and contribute to the four main pillars of SuDS; amenity, biodiversity, water quality and water quantity.
4. Supporting information – Provide cross references to the page or section of the drainage strategy report where the detailed information to support each element can be found. This may be more than one reference for each

Policy

Drainage strategies for developments in the London Borough of Hillingdon need to comply with the following policies on SuDS:

1. [London Borough of Hillingdon Local Plan policies EM 6 and DMEI 10](#)
2. [London Plan policy 5.13](#) and draft [New London Plan policy SI13](#)
3. [The National Planning Policy Framework \(NPPF\)](#)

Technical Guidance

- Post-development surface water discharge rate should be limited to greenfield runoff rates. Proposals for higher discharge rates should be agreed with the LLFA ahead of submission of the Planning Application. Clear evidence should be provided with the Planning Application to show why greenfield rates cannot be achieved.
- Greenfield runoff rate is the runoff rate from a site in its natural state, prior to any development. This should be calculated using one of the runoff estimation methods set out in Table 24.1 of CIRIA C753 The SuDS Manual.
- Attenuation storage volumes required to reduce post-development discharge rates to greenfield rates should be calculated using one of the runoff estimation methods set out in Table 24.1 of CIRIA C753 The SuDS Manual.
- 'CC' refers to climate change allowance from the current Environment Agency guidance.
- An operation and maintenance strategy for proposed SuDS measures should be submitted with the Planning Application and include the details set out in section 32.2 of CIRIA C753 The SuDS Manual. The manual should be site-specific and not directly reproduce parts of The SuDS Manual.
- Other useful sources of guidance are:
 - [o London Borough of Hillingdon sustainable drainage requirements](#)
 - [o The London Plan Sustainable Design and Construction SPG](#)
 - [o DEFRA non-statutory technical standards for sustainable drainage](#)
 - [o Environment Agency climate change guidance](#)
 - [o CIRIA C753 The SuDS Manual](#)

1. Project & Site Details	Project / Site Name (including sub-catchment / stage / phase where appropriate)	Harefield Hospital	
	Address & post code	HAREFIELD HOSPITAL, HILL END ROAD HAREFIELD UB9 6JH	
	OS Grid ref. (Easting, Northing)	E	505167
		N	190856
	LPA reference (if applicable)		
	Brief description of proposed work	The proposed development includes the replacement of existing generators with 2 x new generators, 1 x bulk fuel tanks, and 1x generator control panels for substation 2 and 3 to meet HTM 06-01 requirements.	
	Total site Area	310	m ²
	Total existing impervious area	0	m ²
	Total proposed impervious area	144.594	m ²
	Is the site in a surface water flood risk catchment (ref. local Surface Water Management Plan)?	NO	
	Existing drainage connection type and location	N/A	
	Designer Name	Carrington Msipa	
Designer Position	Senior Project Engineer		
Designer Company	Meinhardt (UK)Ltd		

2. Proposed Discharge Arrangements	2a. Infiltration Feasibility		
	Superficial geology classification	Sudbury Formation	
	Bedrock geology classification	Lambeth Formation	
	Site infiltration rate	0.000094	m/s
	Depth to groundwater level	TBC	m below ground level
	Is infiltration feasible?	Yes	
	2b. Drainage Hierarchy		
		Feasible (Y/N)	Proposed (Y/N)
	1 store rainwater for later use	N	N
	2 use infiltration techniques, such as porous surfaces in non-clay areas	Y	Y
	3 attenuate rainwater in ponds or open water features for gradual release	N	N
	4 attenuate rainwater by storing in tanks or sealed water features for gradual release	N	N
	5 discharge rainwater direct to a watercourse	N	N
	6 discharge rainwater to a surface water sewer/drain	N	N
	7 discharge rainwater to the combined sewer.	N	N
2c. Proposed Discharge Details			
Proposed discharge location	Within the immediate space around the proposed generators		
Has the owner/regulator of the discharge location been consulted?	Proposed discharge on private land owned by the trust and drainage strategy follows the existing drainage strategy on the hospital site.		

3. Drainage Strategy	3a. Discharge Rates & Required Storage				
		Greenfield (GF) runoff rate (l/s)	Existing discharge rate (l/s)	Required storage for GF rate (m ³)	Proposed discharge rate (l/s)
	Qbar	0.2			
	1 in 1	0.2	N/A	N/A	0.2
	1 in 30	0.6	1.4	19	0.2
	1 in 100	0.7	1.8	25	0.2
	1 in 100 + CC			25	0.2
	Climate change allowance used		40%		
	3b. Principal Method of Flow Control		Discharge via infiltration locally		
	3c. Proposed SuDS Measures				
			Catchment area (m ²)	Plan area (m ³)	Storage vol. (m ³)
	Rainwater harvesting		0		0
	Infiltration systems		268.240		25.0
	Green roofs		0	0	0
	Blue roofs		0	0	0
	Filter strips		0	0	0
	Filter drains		0	0	0
Bioretention / tree pits		0	0	0	
Pervious pavements		0	0	0	
Swales		0	0	0	
Basins/ponds		0	0	0	
Attenuation tanks		0		0	
Total		268.240	0	25.0	

4. Supporting Information	4a. Discharge & Drainage Strategy	Page/section of drainage report
	Infiltration feasibility (2a) – geotechnical factual and interpretive reports, including infiltration results	Appendix G
	Drainage hierarchy (2b)	Page 20
	Proposed discharge details (2c) – utility plans, correspondence / approval from owner/regulator of discharge location	Proposed discharge on private land owned by the trust and drainage strategy follows the existing drainage strategy on the hospital site.
	Discharge rates & storage (3a) – detailed hydrologic and hydraulic calculations	Appendix E
	Proposed SuDS measures & specifications (3b)	Appendix C
	4b. Other Supporting Details	Page/section of drainage report
	Detailed Development Layout	
	Detailed drainage design drawings, including exceedance flow routes	Appendix C and Appendix F
	Detailed landscaping plans	
	Maintenance strategy	Page 26 - 27
	Demonstration of how the proposed SuDS measures improve:	
	a) water quality of the runoff?	Page 23 - 24
	b) biodiversity?	Page 18
	c) amenity?	Page 18

Appendix E – Modelling Results.

Design Settings

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

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Easting (m)	Northing (m)	Depth (m)
5	0.006	5.00	88.680	1200	505249.867	190733.693	0.307
4	0.008	5.00	88.720	1200	505253.560	190740.946	0.252
3	0.011	5.00	88.680	1200	505247.811	190741.601	0.252
2	0.006	5.00	88.760	1200	505254.059	190750.108	0.358

Simulation Settings

Rainfall Methodology	FEH-22	Starting Level (m)	
Rainfall Events	Singular	Check Discharge Rate(s)	✓
Summer CV	1.000	1 year (l/s)	0.1
Winter CV	1.000	2 year (l/s)	0.1
Analysis Speed	Detailed	30 year (l/s)	0.3
Skip Steady State	✓	100 year (l/s)	0.5
Drain Down Time (mins)	240	Check Discharge Volume	✓
Additional Storage (m³/ha)	20.0	100 year +40% 360 minute (m³)	15

Storm Durations

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

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
1	0	0	0
2	0	0	0
30	0	0	0
100	0	0	0
100	40	0	0

Pre-development Discharge Rate

Site Makeup	Greenfield	Growth Factor 30 year	2.40
Greenfield Method	IH124	Growth Factor 100 year	3.19
Positively Drained Area (ha)	0.031	Betterment (%)	0
SAAR (mm)	669	QBar	0.1
Soil Index	4	Q 1 year (l/s)	0.1
SPR	0.47	Q 2 year (l/s)	0.1
Region	6	Q 30 year (l/s)	0.3
Growth Factor 1 year	0.85	Q 100 year (l/s)	0.5
Growth Factor 2 year	0.88		

Pre-development Discharge Volume

Site Makeup	Greenfield	Return Period (years)	100
Greenfield Method	FSR/FEH	Climate Change (%)	40
Positively Drained Area (ha)	0.031	Storm Duration (mins)	360
Soil Index	4	Betterment (%)	0
SPR	0.47	PR	0.489
CWI	100.593	Runoff Volume (m ³)	15

Node 2 Soakaway Storage Structure

Base Inf Coefficient (m/hr)	0.33840	Invert Level (m)	88.402	Depth (m)	
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)	5	Inf Depth (m)	
Safety Factor	2.0	Pit Width (m)	2.400	Number Required	1
Porosity	0.30	Pit Length (m)	11.830		

Node 4 Soakaway Storage Structure

Base Inf Coefficient (m/hr)	0.33840	Invert Level (m)	88.468	Depth (m)	
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)		Inf Depth (m)	
Safety Factor	2.0	Pit Width (m)	3.620	Number Required	1
Porosity	0.30	Pit Length (m)	10.671		

Node 3 Soakaway Storage Structure

Base Inf Coefficient (m/hr)	0.33840	Invert Level (m)	88.428	Depth (m)	
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)		Inf Depth (m)	
Safety Factor	2.0	Pit Width (m)	3.620	Number Required	1
Porosity	0.30	Pit Length (m)	10.671		

Node 5 Soakaway Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Invert Level (m)	88.373	Depth (m)	
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)		Inf Depth (m)	
Safety Factor	2.0	Pit Width (m)	2.400	Number Required	1
Porosity	0.30	Pit Length (m)	8.667		

Results for 1 year Critical Storm Duration. Lowest mass balance: 96.44%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	5	1	88.373	0.000	0.6	0.0000	0.0000	OK
15 minute summer	4	13	88.482	0.014	0.9	0.1728	0.0000	OK
15 minute summer	3	13	88.446	0.018	1.1	0.2304	0.0000	OK
30 minute summer	2	20	88.415	0.013	0.5	0.1150	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	Outflow (l/s)
15 minute summer	4	Infiltration	0.5
15 minute summer	3	Infiltration	0.7
30 minute summer	2	Infiltration	0.3

Results for 2 year Critical Storm Duration. Lowest mass balance: 97.80%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	5	1	88.373	0.000	1.0	0.0000	0.0000	OK
15 minute summer	4	13	88.491	0.023	1.4	0.2761	0.0000	OK
15 minute summer	3	13	88.458	0.030	1.9	0.3798	0.0000	OK
15 minute summer	2	13	88.423	0.021	0.9	0.1825	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	Outflow (l/s)
15 minute summer	4	Infiltration	0.8
15 minute summer	3	Infiltration	1.1
15 minute summer	2	Infiltration	0.6

Results for 30 year Critical Storm Duration. Lowest mass balance: 98.40%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m ³)	Flood (m ³)	Status
15 minute summer	5	1	88.373	0.000	2.7	0.0000	0.0000	OK
30 minute summer	4	21	88.540	0.072	3.6	0.8759	0.0000	OK
30 minute summer	3	22	88.535	0.107	4.8	1.3295	0.0000	OK
15 minute summer	2	13	88.465	0.063	2.7	0.5590	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	Outflow (l/s)
30 minute summer	4	Infiltration	1.8
30 minute summer	3	Infiltration	1.8
15 minute summer	2	Infiltration	1.3

Results for 100 year Critical Storm Duration. Lowest mass balance: 98.24%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	5	1	88.373	0.000	3.5	0.0000	0.0000	OK
30 minute summer	4	22	88.574	0.106	4.7	1.2929	0.0000	OK
30 minute summer	3	23	88.585	0.157	6.2	1.9502	0.0000	OK
30 minute summer	2	22	88.495	0.093	3.1	0.8175	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	Outflow (l/s)
30 minute summer	4	Infiltration	1.8
30 minute summer	3	Infiltration	1.8
30 minute summer	2	Infiltration	1.3

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 98.48%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	5	1	88.373	0.000	5.0	0.0000	0.0000	OK
30 minute summer	4	23	88.640	0.172	6.5	2.1099	0.0000	OK
30 minute summer	3	25	88.680	0.252	8.6	3.1370	0.0000	OK
30 minute summer	2	23	88.557	0.155	4.4	1.3646	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	Outflow (l/s)
30 minute summer	4	Infiltration	1.8
30 minute summer	3	Infiltration	1.8
30 minute summer	2	Infiltration	1.3

Appendix F – Flood Exceedance.

[illegible]

1. DO NOT SCALE FROM THIS DRAWING
2. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
3. THIS DRAWING IS FOR CONSTRUCTION PURPOSES.
4. DRAWINGS ARE TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS, ENGINEERS AND CONSULTANTS DRAWINGS AND SPECIFICATIONS.
5. THIS DRAWING IS BASED ON:
 - RSP ARCHITECTURAL DRAWINGS
 - 2607-SK-S1-00-DR-E-10005 AND 2607-SK-S1-DR-E-10006 DATED AUGUST 2024.
 - TOPOGRAPHIC AND UTILITIES SURVEY DRAWINGS 11596/01B-11 DATED DECEMBER 2009.
6. THE PROPOSED SURFACE WATER DRAINAGE ON THE SITE IS SUBJECT TO APPROVAL BY THE ROYAL BOROUGH OF HILLINGDON.
7. DESIGN BASED ON INFILTRATION TEST RESULTS RECEIVED 14.10.24.

 SITE BOUNDARY
 FLOOD EXCEEDANCE ROUTE
 ExSW → ExSW → EXISTING SURFACE WATER SEWER
 ExFW → ExFW → EXISTING FOUL WATER SEWER
 EXISTING SURFACE WATER MANHOLE
 EXISTING FOUL WATER MANHOLE

EXISTING GULLY FOR DISCHARGE OF FLOOD EXCEEDANCE FLOWS FROM GENERATOR SITE. PROPRIETARY MITIGATION MEASURES REQUIRED TO PROTECT DOWNSTREAM EXISTING HOSPITAL DRAINAGE FROM CONTAMINATION. CHOICE OF PROPRIETARY MITIGATION MEASURES TO BE FINALISED AT LATER DESIGN STAGES AS DESIGN DEVELOPMENT PROGRESSES.

1. DESIGN SUBJECT TO COUNCIL APPROVAL OF BY THE ROYAL BOROUGH OF HILLINGDON. DESIGN SUBJECT TO CHANGE FOLLOWING DISCUSSIONS WITH THE LOCAL PLANNING AUTHORITY REGARDING TREE REMOVAL IN THE AREA EARMARKED FOR INFILTRATION IMPLEMENTATION.
2. ALL COVER LEVELS SUBJECT TO CHANGE FOLLOWING FURTHER CONSULTATIONS WITH LANDSCAPE ARCHITECTS AS LEVELS MAY REQUIRE RAISING TO PROVIDE ADEQUATE COVER ABOVE PROPOSED SURFACE WATER PIPES. LEVELS DESIGN TO BE DONE BY OTHERS.
3. ALL EXISTING SURFACE WATER DRAINAGE ON THE SITE IS ASSUMED TO BE DISCHARGING INTO THE SOAK-AWAY BETWEEN THE HEART SCIENCE BUILDING AND THE TEMPORARY WARDS. FURTHER SURVEYS REQUIRED TO CONFIRM IF THERE ARE NO ADDITIONAL SEWERS BEYOND THE SOAK-AWAY LOCATION DISCHARGING ELSEWHERE.
4. AREA EARMARKED FOR INFILTRATION WILL REQUIRE THE REMOVAL OF EXISTING TREES SUBJECT TO LOCAL PLANNING AUTHORITY APPROVAL.
5. PROPOSED LOCATIONS OF CONCRETE BASES FOR MOUNTING THE PROPOSED GENERATORS TO BE FULLY COORDINATED WITH THE PROPOSED SURFACE WATER NETWORK. COORDINATION TO BE DONE BY OTHERS.
6. PROPOSED SURFACE WATER INFILTRATION GRATES IN CLOSE PROXIMITY TO EXISTING DUTY UTILITIES. CONSULTATIONS WITH THE RELEVANT AUTHORITIES WILL BE REQUIRED PRIOR TO CONSTRUCTION WORKS ONSITE. CONSULTATIONS TO BE DONE BY OTHERS.
7. THE INTERACTION OF THE EXISTING HOSPITAL SEWER NETWORK WITH ANY PUBLIC THAMES WATER SEWERS IS CURRENTLY UNKNOWN AND FURTHER SURVEYS WILL BE REQUIRED TO CONFIRM ANY INTERACTION POST PLANNING.
8. FURTHER CCTV SURVEYS REQUIRED TO CONFIRM THE DISCHARGE LOCATIONS OF THE EXISTING SURFACE WATER DRAINAGE NETWORK AS WELL AS THE CONDITION OF THE EXISTING SURFACE WATER NETWORK.

1. THE PROPOSED SURFACE WATER DRAINAGE IS DESIGNED TO ACCOMMODATE ALL STORMS UP TO AND INCLUDING THE 1 IN 100-YEAR PLUS 40% CLIMATE CHANGE RAINFALL EVENT.
2. A TOTAL INFILTRATION AREA OF 42m² HAS BEEN PROVIDED TO ACCOMMODATE ALL RAINFALL EVENTS UP TO AND INCLUDING THE 1 IN 100-YR PLUS 40% CLIMATE CHANGE ALLOWANCE RAINFALL EVENT.
3. THE DESIGN INFILTRATION RATE HAS BEEN TAKEN AS 0.000904 m/s IN ACCORDANCE WITH THE INFILTRATION TEST RESULTS RECEIVED 14.10.24.

MEINHARDT
10 Aldersgate Street, London EC1A 4JU
Telephone: +44 (0)20 7831 7969
www.meinhardt.co.uk

Appendix G – Infiltration Testing.

SOAKAWAY TEST RESULTS

BRE DIGEST 365 - SOIL INFILTRATION RATE

Project: Harefield Hospital - Stage 3 Surveys
Client: CAM & Co. Consulting Limited

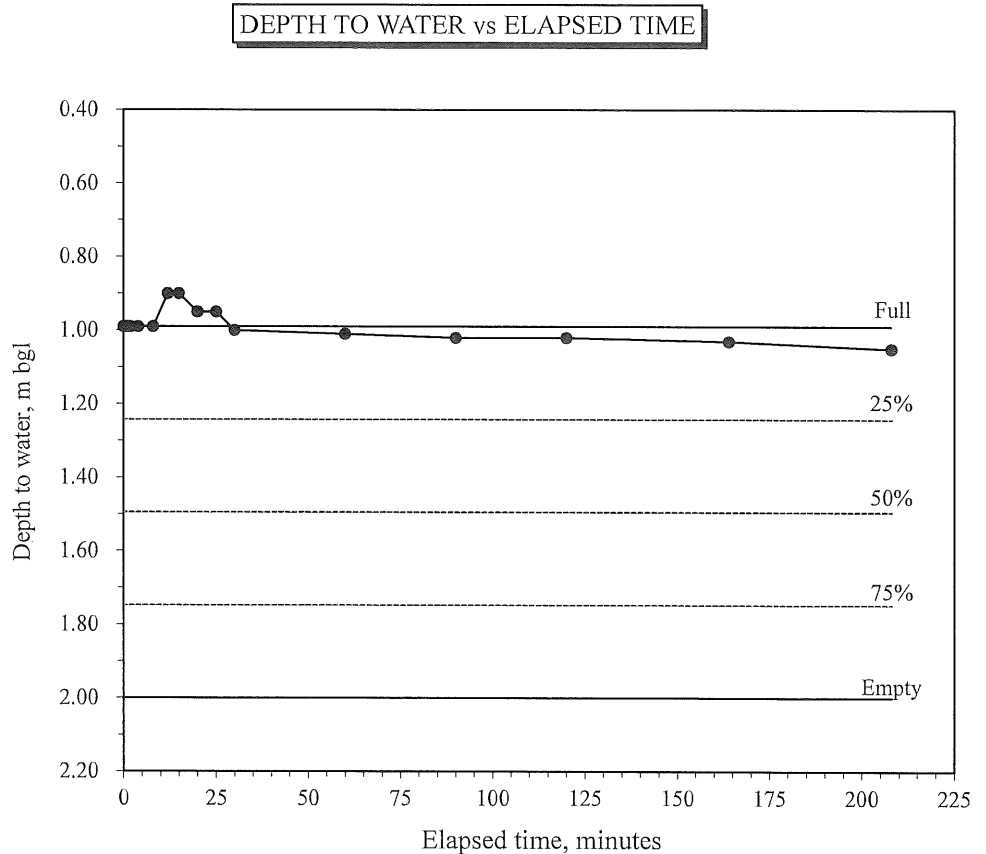
Project No: 118858
Sheet No: 1\1

Trial Pit: TP 4
Depth: 2.00
Length: 1.90
Width: 0.50

Description of Stratum under test: Clayey SAND and GRAVEL

Depth to water prior to test: Dry
(below ground level)

Elapsed Time min	Depth to Water m
0	0.99
1	0.99
2	0.99
4	0.99
8	0.99
12	0.90
15	0.90
20	0.95
25	0.95
30	1.00
60	1.01
90	1.02
120	1.02
164	1.03
208	1.05



$$f = (V75 - V25) / A50(T75 - T25)$$

$$V75 - V25 = 0.480$$

$$A50 = 3.374$$

$$T75 - T25 = 1513 \text{ extraploated}$$

Extrapolated Soil Infiltration Rate $f = \underline{9.40E-05} \text{ m/s}$

Slight sidewall spalling 12 minutes after start of test resulted in higher water level until 30 minutes

Appendix H – Geotechnical Report.



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GROUND INVESTIGATION REPORT

HAREFIELD HOSPITAL

STAGE 3 SURVEYS

Report Reference No. 118858

On behalf of:-

CAM & Co. Consulting Limited
213 Botley Road
Chesham
HP5 1XT

January 2025

CAM & CO. CONSULTING LIMITED

CONSULTING ENGINEERS

GROUND INVESTIGATION

HAREFIELD HOSPITAL

STAGE 3 SURVEYS

Report Reference No. 118858

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INTRODUCTION

CAM & Co. Consulting Limited, the client, are involved in the design of new generators, associated drainage, switchrooms and fuel tanks at Harefield Hospital, Rickmansworth Road, Harefield, Middlesex, for the Royal Brompton & Harefield Hospitals, which are part of the Guy's and St Thomas' NHS Foundation Trust.

The proposed generators and tanks are anticipated to impose relatively light loads of up to 40kN/m².

Lucion Ground Engineering Limited was instructed by the client to carry out a ground investigation to determine the nature and geotechnical properties of the underlying soils in relation to foundation design and construction. In addition, a contamination assessment was to be included within the scope of this investigation.

LOCATION, TOPOGRAPHY AND GEOLOGY OF THE SITE

Harefield Hospital is located on the western side of Rickmansworth Road and west of the junction between Rickmansworth Road and Hill End Road, some 320m north of the centre of Harefield, Middlesex. The hospital is centred at National Grid Reference TQ 051 908.

The hospital complex is approximately 300m wide and 300m long, and is butterfly shaped with curved eastern and western wings, flanking a central core of buildings (see site location plan). The limits of the hospital are defined by; Rickmansworth Road to the east: Park Lane to the south: Hill End Road to the north: and fields to the west.

The proposed generators and fuel tanks are to be located within the southern and northern parts of the hospital, to the east of the Heart Science Centre and to the north-east of the hospital's Boiler House. The respective National Grid References for these locations are TQ 0524 9075 and TQ 0510 9100.

At the time of the investigation, the proposed southern generator/drainage site was near the south-eastern corner of the hospital grounds, immediately to the north of The Harefield G.P. Practice and east of the Heart Science Centre. The area was bounded by a peripheral belt of trees to the east and was surfaced with grass that was crossed south to north by a gravel-surfaced track. An existing emergency diesel generator and associated fuel tank was present at the southern end of the plot. The tree belt included a number of mature Ash, Sycamore, Lime, Hawthorn and Maple trees. This part of the hospital site stands on ground that very gently slopes down towards the north-west from 88.7mOD to 88.4mOD.

The proposed generator/fuel tank site within the northern part of the hospital grounds is located to the north-east of the Boiler House where the ground is covered with hardstanding including asphalt-covered roadways and concrete bases/plinths. This part of the hospital site is largely devoid of vegetation and stands at about 83.8mOD.

Harefield Hospital is located on relatively high ground on the eastern flank of the River Colne valley. The River Colne locally flows southwards 1km to the west of the site.

The 2005 geological map for the area at 1:50,000 scale, Sheet 255, shows the eastern part of the hospital site to be covered by the Gerrards Cross Gravel Member, and underlain by the solid geology of the London Clay Formation that thins towards the west. The London Clay is absent beneath the western part of the hospital site where the Gerrards Cross Gravel Member is underlain by the Upnor and Reading Beds of the underlying Lambeth Group.

A previous ground investigation for extensions within the central parts of Harefield Hospital, by Ground Engineering Limited (C13433, 2015), encountered a 0.40m to 1.30m thick surface layer of made ground, mantling the Gerrards Cross Gravel that was underlain by the initially reworked solid geology of the London Clay at 4.00m in BH 1 (east) and 3.30m in BH 2 (west). The London Clay was present to the base of the two 20.00m deep boreholes. The underlying Lambeth Group, anticipated immediately below the Gerrards Cross Gravel in the western part of the hospital site, was not reached by this investigation.

Preliminary Unexploded Ordnance (UXO) Threat Assessment

The client requested that a Preliminary UXO Threat Assessment be undertaken for the hospital site in order to comply with CIRIA's UXO Risk Management Framework document "Unexploded Ordnance (UXO) – A Guide for the Construction Industry" (C681). Such a report was obtained from 6Alpha Associates Limited, and their report is presented as Appendix 1 at the rear of this report.

The study found that no World War II bombing, bomb damage or military activity was recorded for the study area and consequently no further action was required in this regard for the ground investigation or future construction.

SITE WORK

Two cable percussive boreholes (BH 1 & BH 2), three foundation inspection pits (TP 1, TP 2 & TP 3), and a soakaway test pit (TP 4) were undertaken between 30th September and 7th October 2024 at the positions detailed on the exploratory hole location plan at the rear of this report.

The investigation was undertaken following the protocols detailed in British Standards (BS) 'Code of Practice for Site Investigations' (BS5930:2015+A1:2020), 'Methods of test for soils for engineering purposes' (BS1377:1990), and 'Investigation of Potentially Contaminated Sites' (BS10175:2001+A2:2017).

The site work was undertaken under the supervision of a Geo-environmental Engineer from Lucion Ground Engineering Limited. The works were carried out making due reference to generic and site specific risk assessments, and method statements.

Services information was provided prior to the start of the investigation and was referenced in relation to the exploratory hole positions prior to boring and a scan was undertaken using a cable avoidance tool (CAT).

The exploratory hole records give the descriptions and depths of the various strata encountered, results of the in-situ tests, details of all samples taken and the groundwater conditions observed during excavation/boring and on completion. The elevation of the ground at the borehole and pit positions has been interpolated from the topographical survey plans and added to the hole records.

Cable Percussive Borehole

The cable percussive boreholes were undertaken by a standard cable percussive boring rig on 2nd and 3rd October 2024. Prior to boring at each position, a starter pit was dug to 1.20m below ground level using hand tools, in order to ensure the absence of buried services.

The boreholes were then advanced using weighted shell and claycutter tools, initially working within 150mm diameter casing. Water was added to enable boring through sand and gravel. It was necessary to employ chiselling techniques to advance BH 2 through a concretionary limestone nodule at 7.40m depth. The boreholes were completed at the scheduled depth of 15.00m below ground level.

Standard penetration tests (SPT) were undertaken in order to give an indication of the in-situ relative density strength of the soils encountered. The test was made by driving a 50mm diameter solid cone (C) into the soil at the base of the borehole by means of an automatic trip hammer weighing 63.50kg falling freely through 760mm. The penetration resistance was usually determined as the number of blows (N) required to drive the tool the final 300mm of a total penetration of 450mm into the soil ahead of the borehole. Where the full penetration was not achieved the actual penetration and number of blows was recorded. The SPT results have been added to the borehole records; and tabulated to the rear of the hole records.

Representative small (D) and bulk (B) disturbed samples of soil were taken from the boring tools at regular intervals throughout the depth of the boreholes. Water samples (W) were recovered once sufficient water had accumulated for collection.

On completion of boring, the holes were backfilled with arisings and the surface layers reinstated.

Foundation Inspection Pits

Three foundation pits were undertaken at positions TP 1, TP 2 and TP 3 and excavated to 1.05m, 3.25m and 0.95m below ground level, respectively, on 30th September 2024, using a small hydraulic breaker and long handled hand digging tools. The exposed strata and foundations were logged and the soils sampled by the supervising Geo-environmental Engineer.

Disturbed and environmental samples of soil were taken at regular intervals throughout the foundation inspection pits and placed in polycarbonate pots (D samples) and amber glass jars (ES samples), and large plastic bags (B samples).

An indication of the California Bearing Ratio (CBR) of the sub-grade soils to about 1.00m depth was determined using a dynamic cone penetrometer (DCP) at positions TP 1 and TP 2. The test consisted of dropping an 8kg weight through 575mm and driving a 20mm diameter cone point into the ground. The penetration achieved was plotted against a cumulative blow count and the gradient used to calculate CBR values. The test results are presented together following the pit records.

Following completion of the excavations, the spoil was returned to the pits and placed in layers, which were recompacted and the surface layers reinstated.

Sketch sections, plans and photographs of the exposed footings are presented on the pages following the record for each excavation.

Soakaway Trial Pit

At position TP 4 on 7th October 2024, a 3T mini-excavator was used to dig a 2.00m deep trial pit.

A soakaway test in line with BRE Digest 365 was then undertaken to allow the assessment of soil infiltration rates in the completed pit. The general method of the test was to excavate the trial pit with vertical sides trimmed square to the base. The dimensions were accurately measured and recorded, and the pit was then filled with clean water from a bowser. The rate of water dissipation of water into the ground was measured by recording the head of water at regular time intervals. The very slow rate of infiltration recorded meant that only one filling was undertaken during the single day allowed for these tests.

The results are presented to the rear of the TP 4 trial pit record.

LABORATORY TESTING

The samples were inspected in the laboratory and assessments of the soil characteristics have been taken into account during preparation of the exploratory hole records. The soil sample descriptions are in accordance with BS5930:2015+A1:2020.

Chemical analysis of selected soil samples recovered from the exploratory holes was undertaken, by an independent laboratory, primarily for characterisation purposes. The chemical testing programme was devised by Ground Engineering Limited for a broad suite of potential contaminants, outlined by the Environment Agency (EA) and National House Building Council (NHBC) document R&D 66; 2008 'Guidance for the Safe Development of Housing on Land Affected by Contamination'.

The geotechnical tests were conducted to BS1377:1990 & 2016 and other industry standards, and the results are presented at the rear of this report, whilst the results of the chemical tests are presented in Appendix 2.

Geotechnical Testing

The index properties of selected soil samples were determined as a guide to soil classification and behaviour. The liquid limit was determined by the cone penetrometer method.

The particle size distributions of selected samples were obtained by sieve analysis. Results of these tests are given as particle size distribution curves at the end of this report.

Test specimens were prepared at full diameter from selected undisturbed samples. Immediate undrained triaxial compression tests were made at a single cell pressure equivalent to the overburden pressure for that sample's depth. The moisture content and bulk density of the specimens were also determined.

California Bearing Ratio (CBR) tests were performed on a selected near surface sample. The test consisted of jacking into the remoulded soil a cylindrical plunger with a cross sectional area of 1935mm². A force of 50N was applied initially to seat the plunger on the soil

surface and then the plunger was made to penetrate the soil at a uniform rate of 1mm/min. Readings of force were taken at intervals of penetration of 0.25mm to a penetration not exceeding 7.50mm. The CBR value is the ratio of the force required to achieve 2.50mm or 5.00mm penetration to standard forces expressed as a percentage.

Selected samples of soil and water were analysed to determine the concentration of soluble sulphates. The pH values were also determined using an electrometric method.

Chemical Testing

Six soil samples recovered from the exploratory holes were tested for total concentrations of arsenic, cadmium, chromium, lead, mercury, selenium, nickel and benzo[a]pyrene, together with speciated polycyclic aromatic hydrocarbons (PAH), boron, copper and zinc, phenols, total and free cyanide, hexavalent chromium, sulphate, sulphide and pH. The organic content of these samples was also determined, and they were screened for asbestos containing material (ACM). The six samples were also tested for speciated petroleum hydrocarbons (TPH).

Six samples of made ground from the boreholes and TP 4 were tested for a Waste Acceptance Criteria (WAC) CEN Leachate Suite at 10l/kg.

GROUND CONDITIONS

The ground conditions found by this investigation were broadly as expected with a generally thin cover of made ground, mantling the expected Gerrards Cross Gravel Member, and at depth, the solid geology of the London Clay Formation. The latter was met at about 5.50m below ground level in the boreholes and found to at least 15.00m depth. Groundwater was met within the sand and gravel at about 3.00m below ground level but rose to about 2.00m depth once the borehole casing had been removed.

Made Ground

The exploratory holes in the boiler house area penetrated a thin surface layer of asphalt, which was laid on reinforced concrete (TP 1) or extremely weak concrete (TP 2) to 0.23m and 0.20m below ground level, respectively. This covered a sub-base of dark brown, and locally black, sand and gravel of flint and concrete that was proved to 0.45m and 0.50m depth.

In TP 2 below 0.50m depth, the excavation encountered a firm, brown, light brown and grey mottled, slightly gravelly, silty clay with a gravel fraction of flint, concrete and ash, which became soft, brown grey and dark brown mottled below 2.80m depth. This clay fill was found to at least 3.25m below ground level, adjacent a presumed basement structure.

The part of the hospital to the east of the Heart Science Centre investigated by the two boreholes and trial pits TP 3 and TP 4 encountered a surface layer of topsoil or coarse-grained material that surfaced the track between the hospital and the peripheral tree belt. The coarse-grained fill was proved to between 0.45m and 0.80m depth. The coarse-grained fill was a brown and dark brown or grey, slightly clayey, silty, very gravelly sand or clayey sand and gravel with a gravel fraction of brick, ash, flint, chalk, ironstone, limestone, quartz and quartzite.

Gerrards Cross Gravel Member

The Gerrards Cross Gravel Member was met beneath the made ground at 0.45m to 0.80m depth and at 0.75m depth in TP 4 was initially a firm, orange brown and grey mottled, slightly sandy, gravelly, silty clay with a gravel fraction of angular to sub-rounded flint and chalk. This 0.30m thick clay layer was proved to 1.05m depth.

Below 0.80m depth in the two boreholes; at 0.45m depth in TP 1 and TP 3, and from 1.05m below ground level in TP 4, the Gerrards Cross Gravel Member comprised a very dense, orange brown, initially clayey, silty sand and gravel with a gravel fraction of angular to rounded flint, quartz and quartzite, and occasional chalk. Trial pits TP 1, TP 3 and TP 4 were completed within this stratum at 1.05m and 2.00m depth, respectively, whilst in the two boreholes the sand and gravel became medium dense from 4.00m below ground level.

The base of the Gerrards Cross Gravel Member was proved in the two boreholes at 5.60m (BH 1) and 5.40m (BH 2) below ground level, about 83mOD.

London Clay Formation

The underlying London Clay Formation was initially weathered to a firm or stiff, brown, orange brown and grey mottled, silty clay, which was 0.40m and 0.30m thick and proved to 6.00m and 5.70m below ground level in boreholes BH 1 and BH 2, respectively.

This weathered horizon was followed by a stiff, closely fissured, grey brown, silty clay, which contained occasional silt partings and in BH 2 a cobble size nodule of medium strong, grey, concretionary limestone at 7.40m depth.

The London Clay was very stiff below 10.00m depth and was found to at least 15.00m below ground level where the two boreholes were completed.

Groundwater

A water seepage was noted at the base of the relatively shallow TP 1 within the Gerrards Cross Gravel Member sand/gravel at 1.05m depth.

The three other foundation inspection pits were dry during excavation and on completion.

Water was observed at 3.00m and 3.40m depth within the Gerrards Cross Gravel Member in the boreholes and rose slightly to 2.90m and 3.10m before drilling resumed. Water was added to enable drilling through the sand/gravel below 3.00m and 4.00m depth. Water was sealed out by the borehole casing at 6.00m below ground level in both holes, and the 15.00m deep boreholes were dry or damp on completion.

Removal of the casing yielded water levels standing at 2.00m and 2.10m below ground level in BH 1 and BH 2, respectively.

Observations

Live roots were observed within the surface layers of the exploratory holes within the site to the east of the Heart Science Centre and were recorded to depths between 0.10m and 0.45m below ground level.

The sides of the hand dug and machine dug trial pits were recorded as stable during the short time that they were open.

Evidence of Contamination

Based on visual inspection, the made ground contained pieces of brick, concrete, ash and asphalt.

There was no visual or olfactory evidence of any hydrocarbon contamination within the soils encountered.

There was no visual evidence of asbestos containing materials within the soils encountered during the site work.

Existing Foundations

Trial pit TP 1 was dug against a 0.10m high concrete slab that was laid upon a 0.07m thick layer of bitumen and concrete, below which was a 0.16m thick reinforced concrete layer that projected 0.23m from the slab edge and was based on a sand/gravel sub-base at 0.23m below ground level.

Trial pit TP 2 was excavated against a 0.12m high concrete slab that was 0.42m thick. The excavation also uncovered another concrete below ground structure, separate from the slab and presumably a basement, that was found to at least 3.25m below ground level.

Trial pit TP 3 uncovered a 0.68m deep, 0.18m thick concrete strip footing, with a 0.15m projection, to the curved corridor structure that bounds the south-eastern part of the hospital complex. The footing was based on the Gerrards Cross Gravel Member sand/gravel.

COMMENTS ON THE GROUND CONDITIONS IN RELATION TO FOUNDATION DESIGN AND CONSTRUCTION

The investigation proved a generally thin cover of made ground to between 0.45m and 0.80m depth. The underlying and expected cover of the very dense becoming medium dense Gerrards Cross Gravel Member was proved to about 5.50m below ground level, whilst the underlying solid geology of the stiff becoming very stiff London Clay Formation was found to at least 15.00m below ground level.

Foundations for the proposed generators, tanks and ancillary structures will need to penetrate the made ground and be based within the initially very dense Gerrards Cross Gravel Member at between 0.45m and 1.05m depth. This stratum lies within the reach of shallow foundations and has excellent bearing properties to support the envisaged relatively light loads of these structures. The use of piled foundations should not therefore be necessary.

Foundation Depths

The exploratory holes encountered natural ground at 0.45m to 0.80m below ground level, although an increased thickness of made ground was present associated with existing buried structures, most notably within the Boiler House Area. Large scale processes of natural sedimentation allow a certain degree of confidence to be placed in the absence of important variation of the engineering properties of natural soils across sites. By contrast, made ground, whose history is not completely known, must, despite any amount of investigation, inevitably present the possibility of conditions existing which could not be accepted when considering the material as a bearing stratum.

The near surface initially clayey sand and gravel or gravelly clay (TP 4) had modified plasticity indices between 9% and 14%, and so are of a low volume change potential. A minimum footing depth of 0.75m below finished ground level could be adopted, where the naturally deposited sand/gravel is present at such a depth, in order to be below the depth affected

by seasonal moisture content and resultant volume change. The silty sand/gravel below 1.00m to 2.00m depth may be regarded as non-shrinkable but potentially frost susceptible.

The presence of numerous trees to the east of the Heart Science Centre should be taken into account where footings are within the initially clayey sand and gravel. Foundation excavations should be inspected for live roots, and where present, taken down into the underlying non-shrinkable sand/gravel.

In summary, foundations will need to be taken through the made ground and into the clayey Gerrards Cross Gravel Member at a minimum depth of 0.75m; deepened to penetrate the made ground; or extended into the non-shrinkable sand and gravel.

If foundations are contemplated close to the existing hospital buildings, then new foundations should be taken down to the same depth as the existing foundations, so as not to undermine them, and into naturally deposited Gerrards Cross Gravel Member.

Bearing Pressure and Settlement

The in-situ standard penetration tests in BH 1 and BH 2 indicate that the initially very dense Gerrards Cross Gravel Member has a net allowable bearing pressure of 400kN/m² for foundations up to 1.20m wide and up to 25mm of immediate settlement. Wider foundations, such as rafts to support generators or fuel tanks, could be designed using a similar or slightly lower bearing pressure but this would still be greatly in excess of the 40kN/m² required to support the proposed lightly loaded structures.

Such foundations would impose negligible loads on the medium dense sand and gravel below 4.00m depth, and the London Clay about 5.50m below ground level.

Excavations/Groundwater

The base of foundation excavations should be inspected on completion to ensure that the condition of the soil complies with that assumed in design. Should pockets of inferior material be present, they should be removed and replaced with well graded hardcore or lean mix

concrete. The excavated surface should be protected from deterioration and a blinding layer of concrete used where foundations are not completed without delay.

Water was recorded in the boreholes at between 2.00m and 3.00m depth and so groundwater should not be an issue in respect of the envisaged foundation excavations. However, the excavations may encounter ‘perched’ surface water within the soils at shallow depth, such as was the case in TP 1, which would need to be dealt with by pumping from screened sumps.

Safety precautions should not be neglected especially where personnel are to enter excavations, when the sides of excavations should either be closely supported, or where space permits, ‘battered back’ in order to maintain excavation stability. All excavations should be undertaken in accordance with CIRIA Report 97 ‘*Trenching Practice*’.

Piled Foundations

The use of piled foundations for the proposed loads and structures should not be necessary.

Floor Slabs

Ground bearing floor slabs could be adopted where shallow foundations are adopted. However, where the floor slabs are based on deepened foundations, they may need to be suspended in order to avoid differential movement between the foundations and the floors.

Where constructed within the range of influence of trees, whether they have been removed or are retained, floor slabs may need to be suspended on foundations and separated by a sub-floor gap, as detailed in the previously cited NHBC Standards for low volume change potential clays. This measure is usually adopted for buildings or dwellings with limited tolerances for building movement and may be considered excessive for the proposed structures.

Pavement Design

The DCP tests undertaken within trial pits TP 1 and TP 2 found variable and locally initially high values of CBR within the roadway sub-base, though below 0.50m/0.60m depth the results fell to 0.70% (TP 1) and 2.1% (TP 2) in the sand/gravel and clay fill, respectively.

The results of the laboratory CBR tests within the coarse-grained fill in BHs 1 and 2 ranged between 5.4% and 15%.

The likely pavement formation level within the Boiler House Area would generally be within clayey sand/gravel or gravelly clay, as would the sub-grade to the east of the Heart Science Centre. IAN 73/06 (2009) Design Guidance for Road Pavements recommends design CBR values of 20% for a thin pavement on a poorly graded sand and average construction conditions, whilst a gravelly clay sub-grade with an average plasticity index of 30% could be designed using a conservative CBR value of 3.0%.

A conservative CBR design value for pavement construction on the existing near surface layers of 3.0% would be considered appropriate on the condition that careful inspection and proof rolling takes place prior to construction, and any topsoil, soft or loosely compacted material is removed and replaced with a well graded hardcore or lean mix concrete.

Drainage

The BRE 365 soakaway test undertaken within the 2.00m deep machine excavated TP 4 yielded an extrapolated soil infiltration rate of 7.99×10^{-5} m/s within the initially clayey sand and gravel of the Gerrards Cross Gravel Member.

This result rates these near surface soils as having a 'poor' drainage potential, according to Figure 6 of BS8004:1986, but could be used in the design of soakaway drainage.

It is important that such drainage is kept as far away as possible from buildings and hardstanding in order to avoid the potential for the removal of 'fines' and consequential subsidence.

On this site it is understood that there is limited space for the disposal of water, and that soakaways may well be within influencing distance (<5m) of the proposed generators, tanks and ancillary structures. The initially clayey nature of the sand/gravel beneath this site would be considered likely to inhibit the removal of 'fines' (principally silt and fine sand) and so the close proximity of these structures to soakaways may be considered acceptable if the proposed generators and tanks are designed so that they can tolerate small-scale differential movement/subsidence.

Alternatively, foundations for those structures within 5m of soakaways could be deepened so that they are based at or just above the groundwater level within the sand/gravel, which was met at 3.00m depth in the boreholes but was later recorded at about 2.00m below ground level following casing removal. The scope for the removal of 'fines' from the thin layer of remnant sand and gravel beneath the foundations and above the groundwater level would consequently be greatly diminished and should be within tolerable limits.

The advice of specialist drainage contractors with local experience in these soils should be sought in this regard.

Buried Concrete

Sulphate analysis of the soil and water samples tested in the geotechnical laboratory gave results in Design Sulphate Class DS-1 of the BRE Special Digest 1, Table C2 (2005) presented in Appendix 3. The pH results were between 6.0 and 7.6 and so acidic to alkaline.

Using the soil/water sulphate and pH results, an Aggressive Chemical Environment for Concrete (ACEC) Class of AC-2z would be considered appropriate for buried concrete at shallow depth beneath this site as detailed in the above cited BRE document.

COMMENTS ON THE CHEMICAL TEST RESULTS

The results of the laboratory chemical testing on near surface soil samples have primarily been compared to soil screening values (SSVs) produced by Land Quality Management Limited (LQM) and the Chartered Institute for Environmental Health (CIEH) presented in their document ‘The LQM/CIEH S4ULs for Human Health Risk Assessment: 2015 (Publication Number S4UL3608)’. The LQM/CIEH S4ULs are intended for use in assessing the potential risks posed to human health by contaminants in soil and are transparently-derived and cautious ‘trigger values’ above which further assessment of the risks or remedial action may be needed. The S4ULs (Suitable for Use Levels) have been derived, in accordance with UK legislation and Environment Agency policy, using a modified version of the Environment Agency CLEA 1.06 software.

Reference has also been given to AGAC soil screening values produced by Society of Brownfield Risk Assessment (SoBRA) as documented in their July 2020 publication ‘Development of Acute Generic Assessment Criteria for Assessing Risks to Human Health from Contaminants in Soil’. With the absence of a S4UL for cyanide, the SoBRA AGAC has been used as the soil screening criteria within this report because the acute dose toxicity thresholds for free cyanide are very close to the chronic dose toxicity thresholds.

In 2014 the Department for Environment Food and Rural Affairs (DEFRA) published, in their document SP1010, Category 4 Screening Levels (C4SL) for several contaminants including lead. The C4SL represent screening levels below which the land could be considered suitable for a specified use and definitely not contaminated land in respect of those determinands. With the absence of S4UL for lead the C4SL has been used as the soil screening criteria within this report.

For each contaminant the adopted soil screening criteria have been calculated for the following land use:

- Commercial and industrial usage

The intended purpose of the SSVs are as “intervention values” in the regulatory framework for assessment of human health risks in relation to land use. These values are not binding standards but are intended to inform judgements about the need for action to ensure that a new use of land does not pose any unacceptable risks to the health of the intended users.

Tables 1 and 2 compare the test results for the made ground with the SSVs in relation to the specified uses. The number of test results, which exceed these values, are also provided.

Table 1: Comparison of Chemical Test Results for Made Ground with SSV

Determinand	Number of Samples	Min Value (mg/kg)	Max Value (mg/kg)	Number of Samples Exceeding SSV for:	Soil Screening Criteria SSV (1% SOM)	
					Assessment Method	Commercial/ Industrial mg/kg
Organic Content	6	1.2%	5.0%	-	-	-
Arsenic	6	7.2	22	0	S4UL	640
Cadmium	6	0.17	0.71	0	S4UL	190
Chromium	6	14	46	0	S4UL	8600
Boron	6	<0.40	1.6	0	S4UL	240,000
Lead	6	18	190	0	C4SL	2330
Mercury	6	<0.05	0.34	0	S4UL	320
Selenium	6	0.55	2.3	0	S4UL	12,000
Nickel	6	8.6	59	0	S4UL	980
Phenols	6	<0.10	<0.10	0	S4UL	440
Copper	6	14	110	0	S4UL	68,000
Zinc	6	51	260	0	S4UL	730,000
Free Cyanide	6	<0.50	0.80	0	AGAC	24
Notes S4UL and C4SL for metals were derived using 6% SOM. These values are not sensitive to SOM and would also be applicable for 1% SOM and 2.5% SOM LQM/CIEH S4ULs ‘Copyright Land Quality Management Limited reproduced with permission; Publication Number S4UL3608. All rights reserved’ AGAC soil screening values produced by Society of Brownfield Risk Assessment (SoBRA) as documented in their July 2020 publication ‘Development of Acute Generic Assessment Criteria for Assessing Risks to Human Health from Contaminants in Soil’.						

Table 2: Comparison of Chemical Test Results for Made Ground with SSV

Determinand	Number of Samples	Min Value (mg/kg)	Max Value (mg/kg)	Number of Samples Exceeding SSV for:	Soil Screening Criteria SSV (1% SOM)	
				Commercial/Industrial	Assessment Method	Commercial/Industrial mg/kg
Acenaphthene	6	<0.10	0.36	0	S4UL	84,000
Acenaphthylene	6	<0.10	0.44	0	S4UL	83,000
Anthracene	6	<0.10	0.80	0	S4UL	520,000
Benzo[a]anthracene	6	<0.10	1.1	0	S4UL	170
Benzo[a]pyrene	6	<0.10	1.2	0	S4UL	35
Benzo[b]fluoranthene	6	<0.10	1.8	0	S4UL	44
Benzo[g,h,i]perylene	6	<0.10	3.1	0	S4UL	390
Benzo[k]fluoranthene	6	<0.10	3.1	0	S4UL	1200
Chrysene	6	<0.10	1.8	0	S4UL	350
Dibenz[a,h]anthracene	6	<0.10	0.35	0	S4UL	3.5
Fluoranthene	6	0.33	2.9	0	S4UL	23,000
Fluorene	6	<0.10	0.44	0	S4UL	63,000
Indeno[1,2,3-cd]pyrene	6	<0.10	0.80	0	S4UL	500
Naphthalene	6	<0.10	0.81	0	S4UL	190
Phenanthrene	6	0.24	1.9	0	S4UL	22,000
Pyrene	6	0.24	3.0	0	S4UL	54,000
Notes LQM/CIEH S4ULs 'Copyright Land Quality Management Limited reproduced with permission; Publication Number S4UL3608. All rights reserved'						

Discussion of Results

The results of the laboratory analyses (Tables 1 & 2) indicate that the made ground beneath the site did not contain elevated concentrations of the contaminants tested for when compared with the thresholds for commercial/industrial end uses.

Visual and olfactory evidence of hydrocarbon impacted soils was not detected within the soils encountered during the site work. The TPH results recorded within samples tested from the boreholes, TP 4 and within the WAC tests were between less than the very low

10mg/kg analysing laboratory detection limit and 190mg/kg. These results are not considered significant.

No asbestos containing materials were identified within the six samples screened within the analysing laboratory.

Based on the results obtained from the chemical analyses, the soils beneath the site would be considered suitable for re-use exposed at the surface in a commercial/industrial setting, representative of the existing and proposed end use.

COMMENTS ON CONTAMINATION IN RELATION TO PROPOSED DEVELOPMENT

Details of the proposed transformers, tanks and ancillary structures were not known at the time of report preparation. This investigation may not have revealed the full extent of contamination on the site and appropriate professional advice should be sought if subsequent site works reveal materials that appear to be contaminated.

Contaminated Soil

The exploratory holes generally found up to 0.80m of made ground beneath the site. No concentrations of contaminants were found that exceeded the SSVs for a commercial/industrial setting, representative of the existing and proposed end use.

There is a low likelihood that the made ground soils would affect groundworkers, and a low likelihood that future end users of the site would be impacted where the made ground is exposed at the surface within landscaped areas in the proposed redevelopment, if envisaged.

Existing Drainage

Redundant foul or surface water drain runs should be removed from beneath the site and precautions should be in place to ensure that any remaining effluent is directly disposed off-site. The integrity of existing drainage should be checked, and where they are to be retained, any damaged sections should be replaced prior to redevelopment. The latter measures should remove any future risk to human health and to the water environment.

Human Health - Construction Workers

The results of the chemical testing indicate that the likelihood of ground contamination affecting groundworkers would be low.

No special precautions would be required during the development of the site by workers who may come into contact with the soil during groundworks, providing that standard precautions are adopted which should generally include the procedures given by the Health and Safety Executive industry guidance.

For the protection of workers during groundworks the following is recommended:

- a) Limit repeated or prolonged skin contact with soils by wearing gloves with sleeves rolled down.
- b) Washing facilities should be made available to groundworkers, as to minimise the potential for inadvertent ingestion of soil.
- c) If any soils are revealed which are different to those encountered by this ground investigation, the advice of a specialist should be sought in view of classifying the material and ascertaining its risk to groundworkers.
- d) Dust suppression measures such as 'dampening down', could also be adopted to prevent the spread of soil contaminants.

Human Health - Users of Completed Development

The results of the chemical testing indicate that the likelihood of ground contamination affecting the site users when exposed at the surface would be low. No formal scheme of remediation is therefore required.

Off-Site Disposal of Soil Arisings

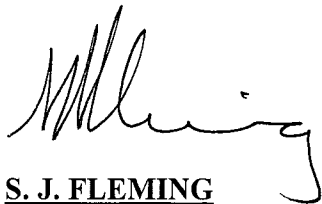
The results of chemical analysis are provided in Appendix 2 and can be used for the basic characterisation of the soil destined for landfill. The Environment Agency publication Hazardous Waste, Technical Guidance WM3 outlines the methodology for classifying wastes

and should be referenced for guidance. The test results (total metals, hydrocarbons and cyanide) should be compared to the relevant thresholds to determine whether they fall into the primary categories of non-hazardous waste or hazardous waste and will help indicate the likely European Waste Catalogue (EWC) code, which is determined by the waste type. The results of the Waste Acceptance Criteria (WAC) leachate testing should be used to check whether if categorised as non-hazardous waste it could be disposed of at an inert waste landfill; or if categorised as hazardous waste whether it could qualify as stable non-reactive hazardous waste for disposal in non-hazardous landfill.

Excavated material and excess spoil should always be classified prior to removal from site as required by 'Duty of Care' (Environmental Protection Act, 1990) legislation. This means that material must be given a proper description and waste classification prior to removal. Basic characterisation is the responsibility of the waste producer and compliance checking and on-site verification are generally the responsibility of the landfill operator. The landfill operator will need to liaise with the waste producer as the approach relies on the information from basic characterisation.

It is expected that clean arisings from foundation excavations into the natural soils across this site would also fall into the inert category under the European Waste Catalogue description 'Soil and Stones', EWC code 17 05 04 with restrictions excluding topsoil and peat.

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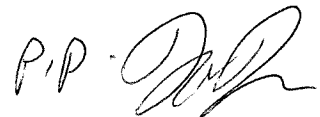


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Ground Engineering Director



S. WEATHERLEY

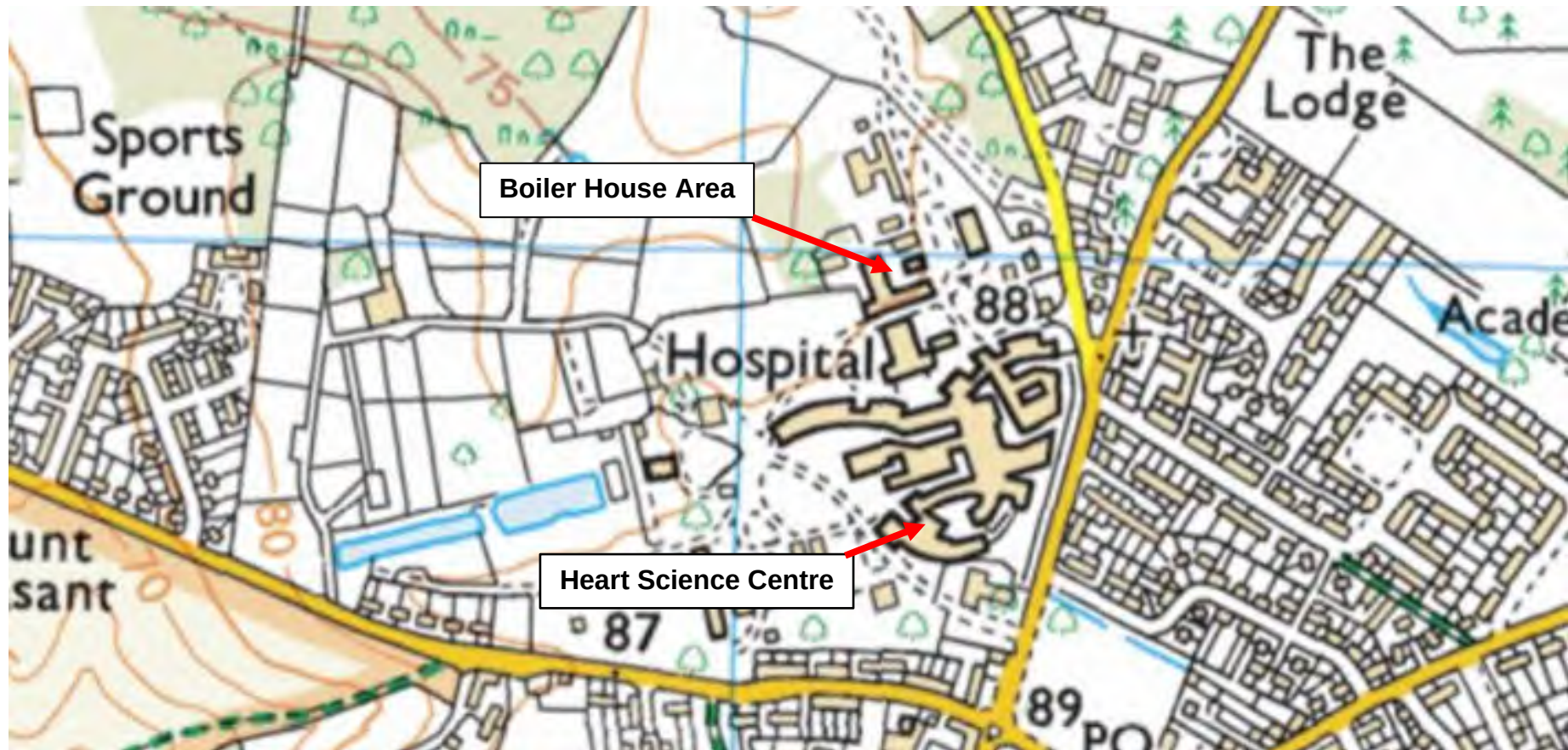
B.Eng.(Hons.),

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Associate

Site Location Plan

Based on Ordnance Survey mapping © Crown Copyright AL100005523 (Not to scale)



Project : Harefield Hospital – Stage 3 Surveys

Client : CAM & Co. Consulting Limited

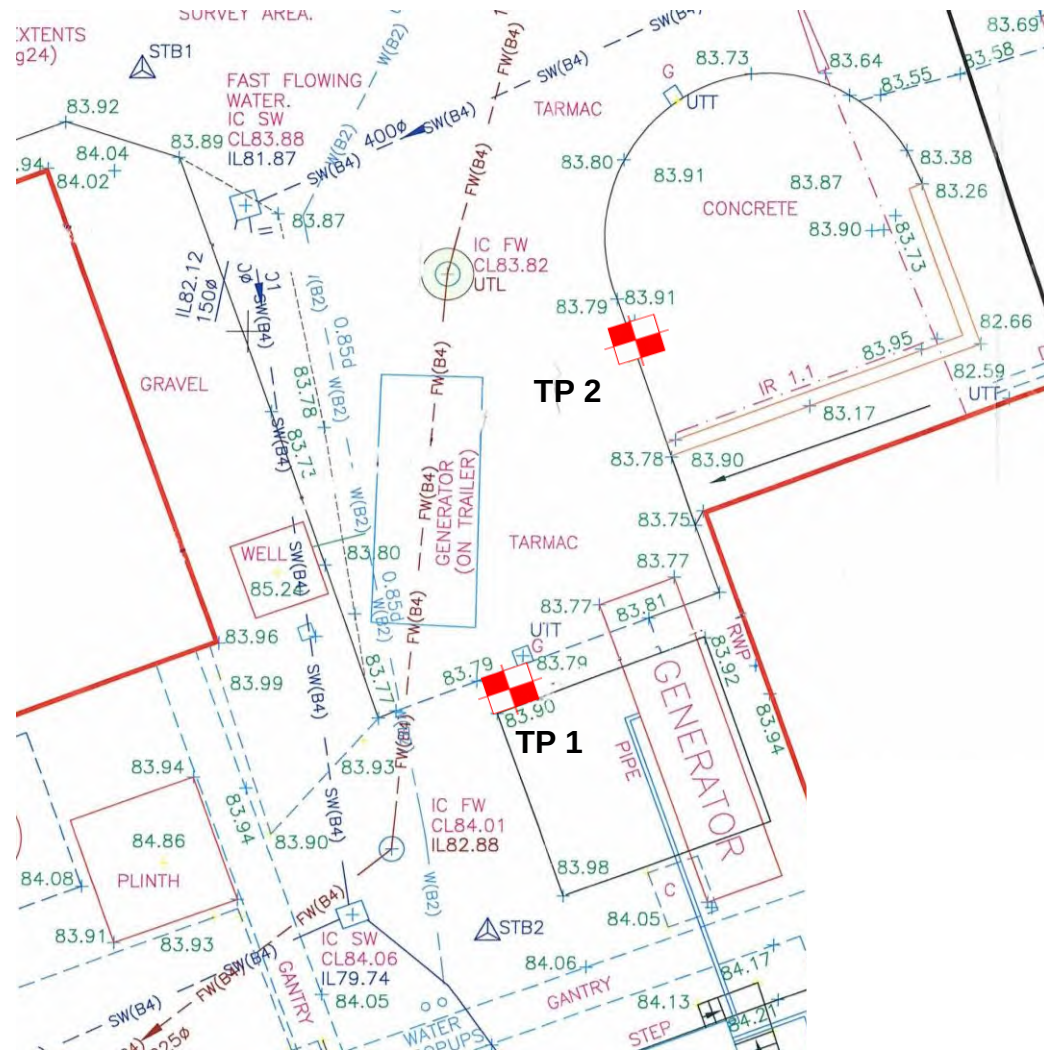


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Project No.
118858

Exploratory Hole Location Plan – Boiler House Area

Reproduced from a topographical plan provided by the client (Not to scale)



Key –

Trial Pit



Project : Harefield Hospital – Stage 3 Surveys

Client : CAM & Co. Consulting Limited