Luton Rising
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February 2023

London Luton Airport Expansion

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5.02 Appendix 20.3 Hydrogeological Characterisation Report

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5.02 ENVIRONMENTAL STATEMENT APPENDIX 20.3 HYDROGEOLOGICAL CHARACTERISATION REPORT

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1 INTRODUCTION

1.1 Report Context

1.1.1 This report is part of the suite of documents prepared to support the application for development consent for the proposed expansion of London Luton Airport ('the airport'). Specifically, this Hydrogeological Characterisation Report (HCR) is a technical appendix supporting **Chapter 20** Water Resources and Flood Risk of the Environmental Statement (ES) **[TR02000/APP/5.01]**.

1.1.2 This report provides a comprehensive description of the existing hydrogeological characteristics underlying the airport. These baseline conditions have been taken into consideration through the design and assessment process.

1.1.3 The report has been prepared based on the hydrological and hydrogeological data available at this time and should be considered, and if required revisited, during detailed design and prior to commencement of construction activities.

1.2 Proposed Development

1.2.1 An overview of the Proposed Development and the site and surroundings in which it is proposed is provided in **Chapter 2** Site and Surroundings of the ES **[TR02000/APP/5.01]**. A detailed description of the Proposed Development is provided in **Chapter 4** The Proposed Development of the ES **[TR02000/APP/5.01]**. A summary of those elements of the Proposed Development relevant to this assessment is provided below:

a. extension and remodelling of the existing passenger terminal (Terminal 1) to increase the capacity;

b. new passenger terminal building and boarding piers (Terminal 2);

c. earthworks to create an extension to the current airfield platform, the vast majority of material for these earthworks would be generated on site;

d. airside facilities including new taxiways and aprons, together with relocated engine run-up bay and fire training facility;

e. landside facilities, including buildings which support the operational, logistics, energy and servicing needs of the airport;

f. enhancement of the existing surface access network, including a new dual carriageway road (Airport Access Road (AAR)) accessed via a new junction on the existing New Airport Way (A1081) to the new passenger terminal along with the provision of forecourt and car parking facilities;

g. extension of the Luton Direct Air to Rail Transit (Luton DART) with a station serving the new passenger terminal;

h. landscape and ecological improvements, including the replacement of existing open space; and

i. further infrastructure enhancements and initiatives to support the target of achieving zero emission ground operations by 2040[1], with interventions to support carbon neutrality being delivered sooner, including facilities for greater public transport usage, improved thermal efficiency, electric vehicle charging, on-site energy generation and storage, new aircraft fuel pipeline connection and storage facilities and sustainable surface and foul water management installations.

[1] This is a Government target, for which the precise definition will be subject to further consultation following the *Jet Zero Strategy*, and which will require further mitigations beyond those secured under the Development Consent Order.

2 TOPOGRAPHY AND HYDROLOGY

2.1 Topography

2.1.1 The airport is located immediately north east of the River Lee (also spelt Lea but Lee applied to all documents related to the Proposed Development) on an elevated escarpment area that forms part of a scarp slope of the Chilterns Hills.

2.1.2 The topography of the Proposed Development, encompassing the whole of the proposed airport expansion, varies between 98 to 164 metres Above Ordnance Datum (mAOD). The highest ground is located in the north west and the land gradually lowers to the south east where the topography includes a dry valley network. The Proposed Development includes two branches of the dry valley network which join approximately 250m south east of the Proposed Development. The topography of the Proposed Development is illustrated in **Figure 1** in **Appendix A** to this report.

2.2 Hydrology

2.2.1 No surface watercourses run through the Proposed Development. The nearest large watercourses are the River Lee situated 450m to the south west of the Main Application Site (as defined in **Chapter 2** of the ES **[TR02000/APP/5.01]**) and the River Mimram situated 3.5km east of the Proposed Development. These are both likely to be in continuity with the Chalk aquifer, and act as major sinks for the groundwater in the area.

2.2.2 More detail on the local river network is available in the Flood Risk Assessment (FRA) provided as **Appendix 20.1** and the Water Framework Directive (WFD) Compliance Assessment provided as **Appendix 20.2** of the ES **[TR020001/APP/5.02]**.

2.3 Rainfall

2.3.1 Monthly rainfall records have been obtained from the nearby Runley Wood Pumping Station over the period from January 1989 to July 2022 (Ref. 1). These are shown in **Figure 2** in **Appendix A** to this report. Rainfall varies significantly from month-to-month and year-to-year but is generally observed to be highest during winter months and lower during summer months. Monthly rainfall values from this Station range from 1.2mm (June 2018) to 176.4mm (May 2007).

3 GEOLOGY

3.1 Data sources

3.1.1 The following resources have been utilised in the conceptual understanding of the geology around the airport:

a. the British Geological Survey (BGS) report "The physical properties of major aquifers in England and Wales" (Ref. 2);
b. BGS Geology of Britain webviewer (Ref. 3); and
c. on-site ground investigation as documented in a Contamination Quantitative Risk Assessment (Ref. 4).

3.2 Published geology

Superficial

3.2.1 Superficial deposits that occur at the Proposed Development include:

a. Made Ground;
b. Head deposits; and
c. Clay-with-Flints.

3.2.2 Both the Made Ground and Clay-with-Flints underlie the majority of the Proposed Development whereas the Head deposits are found in a thin band within the dry valley bottoms.

3.2.3 Alluvium and Glaciofluvial Deposits are found to the west of the Proposed Development along the course of the River Lee.

3.2.4 The geological map of the Proposed Development is shown in **Figure 3** in **Appendix A** to this report which shows where superficial deposits are present.

Bedrock

3.2.5 The bedrock beneath the Proposed Development consists of Cretaceous Chalk (undifferentiated Lewes Nodular and Seaford Chalk formations). These are classified as being part of the "White Chalk Subgroup".

3.2.6 These are composed of firm and hard chalk strata with common nodular and tabular flints and hardgrounds.

3.2.7 These in turn are underlain by the older Holywell Nodular and New Pit Chalk formations, also part of the “White Chalk Subgroup”, which outcrop within the dry valleys. These are generally similar in composition to the overlying Chalk formations but are generally flintless.

3.2.8 The Chalk is unusual compared to many other limestones, due to its almost entirely biogenic origin. In general, the Chalk is extremely fine grained (<10 µm), soft and micritic. Coccoliths and other microfossils such as foraminifera and calcispheres make up a notable quantity of the matrix.

3.2.9 An update of the ground model of the Proposed Development, originally presented by Arup, 2017 (Ref. 4) is shown in **Figure 4** in **Appendix A** to this report.

Geological structure

3.2.10 The BGS webviewer (Ref. 3) does not show there to be any faults that run through or close to the Proposed Development. However, there is expected to be fracturing and jointing within the Chalk bedrock, and secondary dissolution features which provide the primary flow mechanism through the saturated Chalk.

3.3 Previous ground investigations and studies

3.3.1 Several ground investigations have been undertaken across the area of the proposed expansion works. The following ground investigation reports have been used to provide data across the Proposed Development:

a. AECOM (2019) Luton Airport Landfill, Main Ground Investigation - Factual Report (AECOM 2019 report) (Ref. 5);
b. AECOM (2018) Luton Hangar 24 Ground Investigation, Factual Ground Investigation Report (AECOM 2018 report) (Ref. 6);
c. Structural Soils Limited (2017) Landfill Factual Report on Ground Investigation (Structural Soils 2017 report A) (Ref. 7);
d. Structural Soils Limited (2017) Century Park Factual Report on Ground Investigation (Structural Soils 2017 report B) (Ref. 8);
e. Structural Soils Limited (2017) Century Park Access Road, Factual Report on Ground Investigation (Structural Soils 2017 report C) (Ref. 9);
f. Concept Site Investigations (2015) Luton Airport Terminal Extension, Site Investigation Report (Ref. 10);
g. Soil Engineering (2012) Report on a Ground Investigation for Luton Airport FBO (Ref. 11);
h. RSK Environmental Limited (2012) Ocean Sky Jet Building, Luton Airport, Geoenvironmental and Geotechnical Ground Investigation (Ref. 12);
i. Delta Simmons Environmental Consultants Limited (2012) Preliminary Site Investigation Report for Proposed Taxiway Foxtrot (Ref. 13);
j. Wardell Armstrong (2008) Stirling Place (Former Kimpton distribution centre) Ground Investigation Report (Ref. 14);
k. RSA Geotechnics Limited (2004) LLA Hangar and Taxiway Extension at Luton Airport, Preliminary Factual Report (Ref. 15); and
l. Fugro Engineering Services (2003) East Corridor Improvements for Luton Borough Council (Ref. 16).

3.3.2 Borehole installation was undertaken as part of the most recent ground investigation from 2017 onwards. Due to access constraints this investigation was completed in two stages. The Wigmore Park area of the Proposed Development was completed in June to July 2018 (Ref. 6). Investigation of the Airport Long Stay Car Park was undertaken in December 2018. More recently, further work was carried out by AECOM (2019) (Ref. 5) in the vicinity of the landfill.

3.3.3 The exploratory holes undertaken as part of the ground investigations have provided a network of monitoring boreholes to assess the groundwater levels and quality. These locations were monitored for groundwater levels and chemical quality throughout 2018 and early 2019. A single additional monitoring round was undertaken in March 2020 of a number of priority boreholes that were still accessible. This ground investigation is detailed further in this report in **Sections 5.5** to **5.7**.

3.4 Site investigation

3.4.1 The site investigation of the Proposed Development generally confirms the published geology. A summary of the geological conditions encountered from the 2017 report (Ref. 4) are provided below.

3.4.2 The Clay-with-Flints Formation is comprised of stiff, reddish-brown, slightly sandy gravelly clay with a medium cobble content. The gravel is angular to rounded and comprises flint gravel and occasional chalk. The Clay-with-Flints deposit varies in thickness beneath the Proposed Development. It is mainly present on the plateau and valley sides and absent from the base of the valley. It is typically 3.7m thick but has been recorded up to 15m thick. This reflects the irregular dissolution contact between the Clay-with-Flints and the Chalk group.

3.4.3 The condition of the Chalk encountered beneath the Proposed Development is variable. In the upper levels of the Chalk the material has been found to be heavily weathered and was generally recovered as structureless sandy to very silty gravel or sandy gravelly silt. The Chalk material recovered was occasionally recorded as having yellowish brown staining on what are considered to be natural fracture surfaces. Soft grey marl bands were also recovered from within the Chalk.

3.4.4 Ground investigation at the Proposed Development since the 2017 report (Ref. 4) was published, including the AECOM 2019 (Ref. 5) and 2018 (Ref. 6) reports, also confirms the presence of marl bands within the Chalk. In addition, this recent ground investigation has shown the presence of fractures within the Chalk, sometimes infilled with clay material. These fractures provide the primary mechanism through which groundwater flows in the saturated Chalk.

4 REGIONAL HYDROGEOLOGY

4.1 Data sources

4.1.1 The following resources have been utilised in the conceptual understanding of the groundwater flow around the airport in the Upper Lee Chalk groundwater catchment and surrounding area:

a. the BGS hydrogeological map of the area "Sheet 14: Hydrogeological Map of the area between Cambridge and Maidenhead (1:100,000)" published in 1984 (Ref. 17);
b. the BGS report "The physical properties of major aquifers in England and Wales" (Ref. 2);
c. data provided by the Environment Agency (EA) for groundwater monitoring installations within a 3km radius from the Main Application Site;
d. the EA Hertfordshire Groundwater Model, developed and constructed by Mott MacDonald in 2019, which incorporates the previous numerical groundwater model for the area (the EA Vale of St Albans numerical groundwater model). This is the tool used by the EA for predicting flows and levels in the Hertfordshire region. The Main Application Site is situated in the northern part of the model; and,
e. on-site groundwater monitoring from boreholes constructed as part of the project.

4.2 Regional water resource status

4.2.1 There are two groundwater bodies located in the vicinity of the Proposed Development:

a. an extensive Chalk bedrock aquifer that underlies the Main Application Site; and
b. a smaller superficial aquifer associated with head deposits in the upper reaches of the River Mimram catchment.

4.2.2 The Chalk is designated by the EA as a Principal Aquifer, which are defined as "*layers of rock or drift deposits that have high intergranular and/or fracture permeability - meaning they usually provide a high level of water storage and are likely to support water supply and/or river base flow on a strategic scale*".

4.2.3 EA Groundwater Vulnerability mapping (Ref. 20) indicates that the Chalk aquifer has High and Intermediate vulnerability across the Proposed Development. The Groundwater Vulnerability Maps show the vulnerability of groundwater to a pollutant discharged at ground level. Areas of high groundwater vulnerability are areas with a high susceptibility where a pollutant could be easily transmitted to groundwater. Nationally or regionally significant infrastructure schemes are required by the EA to protect groundwater from contamination.

4.2.4 The Chalk aquifer is a designated WFD groundwater body, ‘the Upper Lee Chalk’. For groundwater bodies, there are two separate classifications, quantitative status and chemical status that in combination provide an overall water body status.

4.2.5 The quantitative status for the Upper Lee Chalk is designated by the EA as Poor, and this is related to over-abstraction of groundwater from this groundwater body. The chemical status for this groundwater body is also designated as Poor. This is related to issues associated with elevated levels of nitrate, pesticides, solvents and other contaminants that occur on a wider catchment scale. On the basis of both the chemical and quantitative status, in 2019 the groundwater body was designated as having a Poor overall status.

4.2.6 The Proposed Development lies within the Upper Lee catchment, part of the Thames River basin district. The EA published their licensing strategy for the Upper Lee (based on evidence gathered during its Catchment Abstraction Management Strategy (CAMS) process) in February 2019. Key points relating to this unit are:

a. There are two downstream assessment points. AP12 relates to the Upper River Lee to Howe Green and AP11 relates to the Upper Mimram. The water resource availability of both catchments is classed as "*No water available for licencing*". The availability of water is heavily restricted due to the requirement to safeguard flows for the Lower Lee which contains a sizable surface water public water supply abstraction.

b. The EA consider all groundwater abstractions in the catchment will directly impact surface flows which are measured at the surface water assessment points. As a result, the EA advises that "*No new consumptive abstractions for groundwater will be granted. An exemption may apply to small scale consumptive licences that result in an overall net benefit to the water environment. These proposals may be considered, subject to a local impact assessment*."

4.2.7 The Proposed Development lies within a Source Protection Zone III (Total Catchment); indicating an area around a source of supply within which all the groundwater ends up at the abstraction point.

4.3 Hydrogeology of the Chalk

4.3.1 The Chalk Group forms the main water bearing strata in the region and most important aquifer unit within the Thames Basin. It supplies drinking water for public consumption and supports river flows within Chalk bournes (intermittent streams flowing from a spring, characteristic of the Chilterns and North Downs).

4.3.2 The Chalk is a soft white carbonate rock traversed by flint and marl layers. The Chalk is composed of minute calcareous shells which impart a high porosity to the matrix. Although this Chalk matrix has a high average porosity of approximately 35%, the permeability of the matrix is low due to the high resistance to flow and drainage through small pore throats. Therefore, most of the Chalk's effective storage is derived from secondary porosity within fractures and fissures.

4.3.3 Most of the flow in the Chalk is likely to occur in a few dilated fractures, through dissolution enhanced features typically occurring within the top 30m of the Chalk. These features are commonly observed in the Chalk, due to the composition and the solubility of the rock material. There is evidence of the presence of solution features in the local area, and although none have been directly observed, it is considered likely that there are solution features beneath the Proposed Development.

4.3.4 Flow within the Chalk is influenced by the presence of these solution features which can lead to interlinkages between groundwater catchments. A strong topographical control on transmissivity is also evident with high transmissivity values occurring within main river valleys, decreasing towards the interfluves.

4.4 Regional aquifer properties

4.4.1 The ‘Thames and Chilterns Middle Chalk’ (Holywell Nodular Chalk and New Pit Chalk) has been observed to have porosity values ranging from 9.5% to 52.6%, with a mean of 31.4% (Ref. 2). The transmissivities of the Chalk within the Thames Basin are usually within the range of 1,500 to 3,000 m^2/d, with storage coefficients (the volume of water which an aquifer releases or takes into storage per unit surface area of aquifer per unit change in head) generally in excess of 2% (Ref. 21).

4.5 Regional groundwater levels, recharge and flow

4.5.1 The hydrogeological map of the area (Ref. 17) and monitoring of regional groundwater levels in the area indicates that the regional flow within the Chalk of the northern Thames Basin is predominately towards the southeast along the dip direction of the Chalk. The main area of groundwater recharge is the Chiltern Hills along the northern boundary where the high topographical escarpments form a major groundwater divide.

4.5.2 The Hertfordshire numerical groundwater model (Ref. 19) also shows the dominant direction of regional flow across the entire model extent to be to the south east, following the dip of the Chalk from the recharge point of the Chiltern Hills to the confined aquifer under London. Along the northern extent of the model, groundwater divides are common.

4.5.3 The regional flow system of the aquifer is likely to be locally modified by abstraction and discharge to groundwater. As mentioned in the 2017 report (Ref. 4) the groundwater flow direction in the Lee catchment is influenced by local abstractions and flows in a westerly direction (four abstractions at approximately NGR 510870 221220 and four at approximately NGR 510700 220650). Similarly, the groundwater flow in the Mimram catchment is affected by the potable abstraction near Kings Walden (located at approximately NGR 514960, 223135) which creates a local easterly flow direction (Ref. 21). In addition, the presence of infiltration basins may cause increases or 'doming' in groundwater levels. This is observed at the existing airport where on-site infiltration basins are causing local increases in groundwater level. This is discussed further in **Section 5.6** of this report.

4.6 Groundwater-surface water connection

4.6.1 Chalk streams receive the majority of their water from groundwater, where in most Chalk streams it constitutes at least 90% baseflow. Chalk streams often display 'bourne' behaviour and stream-aquifer interactions show both spatial and temporal variations based on local interactions. During winter months when groundwater recharge and levels are high, river levels are also high and laterally extensive with springs and surface flows occurring where the water table emerges at the ground surface. In contrast during summer months as water table levels decline, the Chalk streams are vulnerable to drying out. Therefore, Chalk streams show both gaining and losing behaviour based on the seasonal migration of the water table (Ref. 22).

4.6.2 The main rivers near the Proposed Development are the River Lee to the west and River Mimram to the east.

4.6.3 The River Lee (from Luton to Luton Hoo Lakes) is designated as a heavily modified waterbody. During the 2019 Cycle 2 WFD classification, the River Lee was classified as achieving a Bad WFD status with the target to achieve Good by 2027.

4.6.4 The River Lee (from Luton Hoo Lakes to Hertford) is not designated artificial or heavily modified. During the 2019 Cycle 2 WFD classification, the River Lee was classified as achieving a Moderate WFD status with the target to achieve Good by 2027.

4.6.5 The River Mimram (Codicote Bottom to Lee) is not designated artificial or heavily modified. During the 2019 Cycle 2 WFD classification, the River Mimram was classified as achieving a Moderate WFD status.

4.7 Groundwater flooding

4.7.1 Groundwater flooding occurs when water levels in the ground rise above the ground surface. In the Chalk aquifer the main causes of groundwater flooding are from rising groundwater levels due to prolonged intense rainfall, reduced groundwater abstraction, or increases due to artificial obstructions.

4.7.2 The Luton Borough Council (LBC) Local Flood Risk Management Strategy (Ref. 23) presents flood risk into three categories:

a. Limited potential for groundwater flooding to occur (green).
b. Potential for groundwater flooding of property situated below ground level to occur (amber).
c. Potential for groundwater flooding to occur at surface (red).

4.7.3 In the Luton area, there are currently two main areas identified with the potential for groundwater flooding to occur at surface. These areas are the River Lee to the west of the existing airport, and the dry valley to the south east of the Proposed Development.

5 LOCAL HYDROGEOLOGY

5.1 Published information

Hydrogeological map

5.1.1 The hydrogeological map of the area (Ref. 17) shows there to be a groundwater divide under the existing airport just to the west of the long stay car park area. The highest groundwater levels shown are approximately 110mAOD.

Hertfordshire numerical model

5.1.2 Groundwater contours from the Hertfordshire groundwater model have been plotted for the following five time periods; March 1990 (typical high groundwater levels), October 1993 (average levels), December 1997 (minimum levels), November 1999 (typical low levels) and April 2001 (record maximum levels). Maps of the various head levels are shown in relation to the site in **Appendix B** to this report.

5.1.3 The contour outputs from the Hertfordshire model also show the flow beneath the Proposed Development to be influenced by the presence of two groundwater divides, one situated beneath the housing estate to the north of the airport and the other directly to the south of the airport. These groundwater divides are present under all groundwater conditions, and their axis positions do not vary significantly under different groundwater situations. The groundwater divide axis runs almost exactly along the eastern boundary of the existing airport. This suggests that the existing airport infrastructure is located primarily within, and will drain to, the River Lee catchment, whereas the Proposed Development is located within the River Mimram catchment.

5.1.4 The Hertfordshire groundwater model (Ref. 19) shows that at minimum groundwater levels (December 1997) the groundwater levels range from 100mAOD (~20mBGL) around the dry valleys along the eastern extent of the Proposed Development to approximately 105mAOD (~50mBGL) in the centre of the groundwater divide.

5.1.5 At maximum groundwater levels (April 2001) the groundwater levels range from 110mAOD (~10mBGL) within the dry valleys to approximately 125mAOD (~35mBGL) in the centre of the groundwater divide.

5.1.6 The EA model suggests that there is a larger fluctuation in groundwater levels (between minimum and maximum groundwater conditions) in the centre of the groundwater divide at the interfluves than within the bordering valleys. Therefore, a steeper hydraulic gradient exists across the site during times of high groundwater levels. This suggests that the likely seasonal range in groundwater level is approximately 15m within the centres of the groundwater divide. However, away from the groundwater divide within the dry valleys, the seasonal variation in groundwater level is less and typically a maximum of around 5m.

Environment Agency observation boreholes

5.1.7 Groundwater level data obtained from EA observation boreholes within 3km of the Proposed Development have been used to develop the understanding of the groundwater levels in the region. The EA observation boreholes show a long-term record of groundwater levels. The location of these in relation to the Proposed Development are shown on **Figure 5** in **Appendix A** to this report. The groundwater level data have been plotted as a hydrograph in **Figure 6** in **Appendix A** to this report and as box and whisker plots in **Appendix C** to this report.

5.1.8 The outputs from the EA observation boreholes fit with the conceptual understanding demonstrated in the regional Hertfordshire model of the Chalk groundwater system around the Proposed Development.

5.1.9 Putteridge Bury observation borehole, located upgradient of the site, shows a maximum groundwater level of 121mAOD, which corresponds to the April 2001 maximum groundwater levels predicted by the Hertfordshire model. Luton observation borehole, located to the south of the Proposed Development, shows similar groundwater levels to Putteridge Bury. Both these observation boreholes lie along the axis of the groundwater divide, close to the centre of the divide.

5.1.10 Groundwater levels in the river valleys at the monitoring locations Lilley Bottom and Mimram 1 (Mimram 1 demonstrates levels in the river) are noticeably lower than those located in the interfluves.

5.1.11 Statistical analysis, plotted as box and whisker graphs show there to be large differences between the minimum groundwater levels and the 10th percentiles, as well as maximums and 90th percentiles. In some cases, the difference between the two is greater than 5m. This demonstrates the low effective storage of the Chalk aquifer and how extreme years can result in large variation in groundwater levels.

5.1.12 In addition, the box and whisker plots show that Lilley Bottom and Mimram 1 (the monitoring points located within the dry valleys) show less variation in groundwater levels across the year as well as from year-to-year in comparison to Putteridge Bury and Luton observation boreholes located within the interfluves.

5.2 Groundwater recharge

5.2.1 Because there are no surface water features in the vicinity of the Proposed Development the recharge to the underlying Chalk aquifer is almost entirely from infiltration of precipitation, either naturally through soils in the undeveloped areas, or more focused through several on-site engineered infiltration basins. There may also be a minor contribution from mains or sewer leakage.

5.2.2 The anticipated low permeability of the Clay-with-Flints deposit, as well as hardstanding areas associated with the existing airport, may limit rainfall infiltration and therefore recharge to the underlying Chalk aquifer. Recharge is likely to occur to the Chalk aquifer where there is an absence of these low permeability deposits, such as where the Chalk is exposed in the dry valleys.

5.2.3 Observed groundwater levels show a seasonal response to rainfall.

5.3 Groundwater-surface water connection

5.3.1 Local groundwater flow in the Chalk is generally from topographically higher areas which form sub catchment divides to lower lying areas where discharge occurs. Springs and surface flows occur where the groundwater table intersects the land surface. The eastern extent of the Proposed Development occupies a 'dry valley'. Groundwater is closer to the ground surface here, due to the incised topography.

5.3.2 The nearest watercourses are the River Lee situated 450m from the south-western corner of the site and the River Mimram situated 3.5km east of the site. These are both perennial streams and are likely to be in continuity with groundwater, acting as major sinks for the groundwater in the area. The nearest confirmed springs to the site are the source of the River Mimram, 3.5km to the east. The springs at the source of the River Mimram occur at an elevation of approximately 95mAOD.

5.3.3 There are several soakaways present at the existing airport, including the main soakaway for the airport (Central soakaway) which is located to the south of the former landfill and north of the existing runway, these are shown in **Figure 7,** provided in **Appendix A** of this report.

5.3.4 The figurative Conceptual Site Model in **Figure 8** in **Appendix A** to this report shows the surface water and groundwater interactions at the Proposed Development.

5.4 Groundwater flooding

5.4.1 The EA identify an area of groundwater flooding associated with the dry valleys approximately 500m south east of the Proposed Development in the vicinity of Kimpton (Ref. 24). Investigations into the Kimpton groundwater flooding (Ref. 25) provides information on the extent of the area south east of the Proposed Development that flooded during the winter of 2000 to 2001.

5.4.2 The 2000-2001 flooding led to a re-emergence of the historically dry River Kym and subsequently caused flooding of Kimpton village downgradient. EA monitoring borehole records in the area confirm that groundwater levels within these dry valleys were at peak levels.

5.4.3 The groundwater flooding event of February 2001 is the only recorded historical event within the dry valleys downgradient of the Proposed Development. Therefore, groundwater flooding in the vicinity of the Proposed Development is expected to be associated with extreme groundwater levels only (refer to **Section 5.8**).

5.4.4 The Surface Water Drainage Asset Management report produced by Mott MacDonald (2008) (Ref. 26) reports that surface flooding occasionally occurs within and adjacent to the Central soakaway. Based on this report, the surface flooding is due to drainage exceeding infiltration capacity rather than groundwater flooding.

5.5 Ground investigation

5.5.1 Seventeen boreholes were installed for the purposes of groundwater monitoring as part of the ground investigation works undertaken in 2017 (Ref. 7 to 9). These are listed below in **Table 5.1** and shown in **Figure 5** in **Appendix A** to this report**.** The 2017 ground investigation was focused on the landfill (boreholes labelled LF within Area A as defined within **Chapter 17** Soils and Geology of the ES **[TR020001/APP/5.01]**) and beneath the Green Horizons Park (formally Century Park) development (boreholes labelled CP within Area B as defined within **Chapter 17** Soils and Geology of the ES **[TR020001/APP/5.01]**). The areas are illustrated in **Figure 7** in **Appendix A** to this report.

Table 5.1: Groundwater monitoring borehole installation details (all details taken from Arup, 2017 (Ref. 4))

Borehole	Final depth (mBGL)	Top of slotted casing (mBGL)	Base of slotted casing (mBGL)	Geology of response zone	Location
LF-BH01	58.5	27.5	57.5	Chalk	A
LF-BH02	58.5	30.0	58.0	Chalk	A
LF-BH03	55.5	25.0	55.0	Chalk	A
LF-BH04	60.0	25.0	59.0	Chalk	A
LF-BH05	58.5	25.0	58.0	Chalk	South of A
LF-BH08	57.0	25.0	57.0	Chalk	A
LF-BH10	57.0	25.0	56.0	Chalk	A
LF-BH13	35.1	20.0	35.0	Chalk	A
CP-BH11	39.0	29.0	39.0	Chalk	B
CP-BH12	41.1	15.1	41.1	Chalk	B
CP-BH24	42.5	20.0	42.5	Chalk	B
CP-BH27	53.7	30.7	53.7	Chalk	B
CP-BH29	44.3	20.0	44.3	Chalk	B
CP-BH32	41.0	20.3	41.0	Chalk	B
CP-BH50	26.0	9.5	23.5	Chalk	B
CP-BH51	32.0	15.0	32.0	Chalk	B
CP-BH55	52.7	23.9	52.7	Chalk	B

Note: Area A comprises the historic Eaton Green landfill and is situated in a public open space known as Wigmore Valley Park (WVP) to the north of the existing airport.
Area B comprises the north of Wigmore Valley Park and agricultural land adjacent to the eastern edge of the landfill.

5.5.2 An additional seven boreholes, shown in **Table 5.2**, were installed at the Proposed Development over the period from June to December 2018. These were drilled to provide extra information on groundwater levels at the Proposed Development.

Table 5.2: Groundwater monitoring borehole installation details (Ref. 4).

Borehole	Final depth (mBGL)	Top of slotted casing (mBGL)	Base of slotted casing (mBGL)	Geology of response zone	Location
GW201	52.00	20.00	52.00	Chalk	North of A
GW202	54.00	22.45	54.00	Chalk	West of A
GW203	63.00	28.50	63.00	Chalk	A
GW204	64.00	29.00	64.00	Chalk	A
GW205	63.00	28.00	63.00	Chalk	A
GW206	64.50	27.00	64.50	Chalk	A
GW207A	62.00	22.00	62.00	Chalk	A

5.6 On-site aquifer properties

5.6.1 On-site packer testing has been carried out in the boreholes drilled in 2018 to inform on the permeability of the Chalk aquifer at the site. The results of these are shown in **Table 5.3**.

Table 5.3: Results of on-site packer tests

Borehole name	Test date	Permeability test number	Depth of top of test interval (mAOD/Geology)	Hydraulic conductivity (m/s)
GW201	03/07/2018 to 04/07/2018	1	109.65 / Chalk	4.36×10^{-7}
		2	104.65 / Chalk	1.70×10^{-5}*
		3	99.65 / Chalk	2.74×10^{-7}
GW202	10/07/2019 to 12/07/2019	1	111.80 / Chalk	4.34×10^{-7}
		2	105.80 / Chalk	1.38×10^{-6}
		3	99.80 / Chalk	3.81×10^{-7}
		4	95.80 / Chalk	6.55×10^{-8}

Borehole name	Test date	Permeability test number	Depth of top of test interval (mAOD/Geology)	Hydraulic conductivity (m/s)
GW203	25/06/2018 to 21/09/2018	1	133.75 / Chalk	No groundwater encountered
		2	121.95 / Chalk	3.20×10^{-5}*
		3	111.45 / Chalk	2.70×10^{-5}*
		4	100.95 / Chalk	6.61×10^{-7}
		5	91.45 / Chalk	3.80×10^{-8}
GW204	03/07/2018 to 05/07/2018	1	113.6 / Chalk	1.68×10^{-6}
		2	106.6 / Chalk	1.43×10^{-7}
		3	99.6 / Chalk	4.69×10^{-8}
		4	93.6 / Chalk	1.99×10^{-7}
GW205	17/07/2018 to 18/07/2018	1	107.7 / Chalk	4.71×10^{-7}
		2	104.2 / Chalk	3.99×10^{-7}
		3	98.2 / Chalk	1.16×10^{-7}
		4	93.2 / Chalk	9.00×10^{-7}
GW206	04/12/2018 to 05/12/2018	1	117.65 / Chalk	1.50×10^{-5}*
		2	109.65 / Chalk	2.80×10^{-8}
		3	101.65 / Chalk	4.16×10^{-7}
GW207	05/12/2018 to 07/12/2018	1	120.15 / Chalk	1.50×10^{-5}*
		2	109.65 / Chalk	1.90×10^{-5}*
		3	101.65 / Chalk	2.14×10^{-7}
GW207A	10/12/2018	1	95.85 / Chalk	2.12×10^{-7}
*These tests were reported by the contractor as "*unsuccessful due being unable to pressurize system due to rapid water take*". The available information has been used to estimate a permeability.				

5.6.2 Permeability results from the packer tests are variable, with some tests displaying behaviour characteristic of a high conductivity system. These high values are likely to be a result of interception of low frequency but high permeability fissures.

5.6.3 Packer tests only sample a small volume of aquifer and therefore the results of these tests will be representative of the horizon they sample. In the Chalk this will depend on whether high permeability fissures are or are not encountered, with lower permeabilities representative of the Chalk matrix. In addition, all of the packer tests show a degree of dilatancy or wash-out response. These dilatancy responses typically occur when at the highest pressure, the rock fissures are temporarily hydro-jacked which allows for more water take and where and when increased pressure is applied, the water removes the material. When the pressure is reduced the fissures close again. This wash-out occurs when there is either natural or drilling induced infilling of fissures or mud cake development on the borehole.

5.6.4 The packer test data has been separated into 20m intervals from below ground level to observe the hydraulic properties of the Chalk with depth, the averages for each interval are shown in **Table 5.4**.

Table 5.4: Hydraulic conductivity with depth from top of Chalk

Depth from top of Chalk (m)	Mean hydraulic conductivity (m/s)	Geometric mean hydraulic conductivity (m/s)
0 – 20	2.37×10^{-5}	1.30×10^{-6}
20 – 30	8.04×10^{-6}	3.65×10^{-6}
30 – 40	6.00×10^{-6}	7.00×10^{-7}
40 – 52	3.36×10^{-7}	2.38×10^{-7}

5.6.5 The results from this analysis fit the conceptual model of the permeability of the Chalk from regional information. The hydraulic conductivity is shown to decrease with depth, with the higher conductivities associated with the secondary fissures formed from dissolution around the water table fluctuations within the top 20m of the Chalk.

5.6.6 Results show that the on-site data is generally lower than the average regional permeability obtained from published sources (Ref. 2). This may be due to the test method and factors such as dilatancy and wash-out responses or may be a true reflection of the system. The packer test data is considered valid, with appropriate caveats, and has been used to define a value of permeability for use in in the on-site conceptual model for design and risk assessment purposes. It is recommended that additional ground investigation, that includes infiltration and aquifer testing, is carried out as part of the final design process to validate the conductivity values for final engineering design.

5.7 On-site groundwater level monitoring

5.7.1 Groundwater level measurements have been taken from the on-site boreholes presented in **Table 5.1** and **Table 5.2** and shown in **Figure 5** in **Appendix A** to this report. Groundwater level measurements from November 2016 to March 2020 are plotted and shown in **Figure 9 and Figure 10** in **Appendix A** to this report. The groundwater levels beneath the landfill **(Figure 9** in **Appendix A** to this report**)** have been compared with those in the Chalk external to the landfill **(Figure 10** in **Appendix A** to this report**)**.

5.7.2 The groundwater levels beneath the former landfill range between 105mAOD and 125mAOD and are generally at significant depth below ground surface (approximately 30mBGL to 45mBGL). In LF-BH05 located to the south west of the landfill as shown on **Figure 9** in **Appendix A** to this report**,** the highest groundwater level recorded was 124.46mAOD (28.55mBGL) in June 2018. The groundwater levels recorded at this borehole are consistently higher than the levels recorded beneath the remainder of the landfill. It is possible that the groundwater levels in this borehole are being influenced by the nearby Central soakaway for the existing airport (which is located to the south of the former landfill and north of the existing runway and shown in **Figure 7** in **Appendix A** to this report) and is likely to be artificially increasing the levels of groundwater at this monitoring point (Ref. 4).

5.7.3 There are several soakaways present on-site of the existing airport, including the main Central soakaway mentioned above. These are expected to cause local increases in groundwater levels, however the doming is thought to be localised and not to be directly influencing the location of the main groundwater divide in the area.

5.7.4 The groundwater levels recorded under the landfill from January 2018 to December 2018 show a maximum seasonal variation of 10.94m, this was observed in borehole LF-BH04 between January and June 2018. This is due to a high groundwater level reading taken in June 2018 that is dissimilar to all other readings at this location and is considered to be an anomalous reading. However, this should be confirmed with further groundwater monitoring. The next highest seasonal variation observed is 7.6m within LF-BH05.

5.7.5 Beneath the landfill, from year-to-year, groundwater levels can also vary. In borehole LF-BH05, May 2017 levels were substantially lower (9.85m lower) than those observed the following year in June 2018.

5.7.6 Larger seasonal and year-to-year variations in groundwater levels were observed beneath the landfill area, than within the dry valley (part of the Green Horizons Park development ground investigation). Within the dry valley, most of the boreholes display a seasonal variation, between January and December 2018, of less than 5m. The largest seasonal groundwater variation recorded was 5.22m in CP-BH24. Though, due to the lower topographical elevation within the dry valley, groundwater levels are closer to surface (15mBGL to 35mBGL). This variation of fluctuation related to topography is common in the Chalk aquifer, as described in **Section 4.3** above.

5.7.7 The 2018 and 2019 groundwater monitoring suggests a comparable seasonal variability in groundwater levels around the site to those included in the 2017 report (Ref. 4). This round of monitoring supports the Hertfordshire groundwater model outputs, suggesting that the likely seasonal range in groundwater levels is approximately 5m to 10m in the vicinity of the landfill, and up to a maximum 5m variation within the dry valleys.

5.7.8 The minimum and maximum groundwater levels recorded in the on-site boreholes were in May 2017 and June 2018 respectively. (Higher groundwater levels were observed in the March 2020 monitoring visit, however this visit was limited to only a select few of the boreholes). The observed levels within these on-site groundwater monitoring, as well as EA observation boreholes within 3km of the site, have been used to construct the groundwater contour plots across the Proposed Development for these two months. These contour plots are shown in **Figure 11** and **Figure 12** in **Appendix A** to this report. These plots show the range in groundwater levels during the period of on-site monitoring.

5.7.9 Like the regional models, these levels are also suggestive of a groundwater divide underneath the Proposed Development. The divide is observed to be to the east of the existing airport with the centre of this divide located to the north of the current airport runway. As observed on the groundwater contour plots the majority of the flow across the Proposed Development is to the east. As per the Hertfordshire model, these contour plots also show there to be steeper hydraulic gradients across the site during times of high groundwater levels.

5.7.10 The exact positioning of the groundwater divide at the site is shown to be uncertain, with different sources of information predicting different positioning in relation to the airport. However, in all cases the airport extension is located within the River Mimram catchment, with the groundwater divide occurring along the eastern extent of the existing airport.

5.7.11 Although these on-site monitoring rounds can give a good insight into the groundwater levels at the Proposed Development, it is noted that the on-site groundwater monitoring is of limited duration and can only provide a short-term, non-continuous dataset of the groundwater levels and is unlikely to record extreme minimum and maximum groundwater events. Therefore, although June 2018 displays the maximum groundwater captured in the full on-site monitoring, it is not the maximum groundwater conditions that should be used for design and risk assessment purposes (as supported by the limited monitoring undertaken in March 2020). For this, a wider review of monitoring data has been completed.

5.8 Assessment of maximum on-site groundwater conditions

Assessment of April 2001 maximum levels

5.8.1 Consideration was given to constructing a bespoke numerical groundwater model for the Proposed Development, to inform both engineering design and Environmental Impact Assessment. However, after consultation with the EA regional resources team, it was agreed that the EA's existing Hertfordshire model can provide a reliable and robust description of groundwater levels in the vicinity of the Proposed Development. A new numerical model was therefore not required. Utilising the Hertfordshire groundwater model, a number of simulations were undertaken to provide spatial groundwater heads at different time periods for use in this assessment. These outputs are shown in **Appendix B** of this report.

5.8.2 Analysis of the model outputs show that when compared to groundwater levels from the EA monitoring boreholes the Hertfordshire model provides a good match to observed levels near the Proposed Development.

5.8.3 The EA boreholes and Hertfordshire model provide a good temporal resolution of the maximum groundwater levels within the Upper Lee catchment. However, the EA boreholes and Hertfordshire model provide a relatively low spatial resolution around the Main Application Site. This is due to the lack of observation boreholes within the Order Limits and therefore, in the case of the Hertfordshire model, a lack of model calibration data around the immediate area of interest. To further evaluate the groundwater levels presented in this Hertfordshire model with those acquired on site, the following assessment was carried out.

5.8.4 As shown by the Hertfordshire model, the maximum groundwater levels observed in the Luton area in the historical record were in April 2001. Due to the on-site monitoring commencing in 2016, groundwater level data for April 2001 were not available, and therefore no observed levels within the Proposed Development are available for this maximum event. The maximum on-site groundwater levels in June 2018 correspond to a low rainfall year.

5.8.5 Comparisons of groundwater levels within nearby EA observation boreholes between the months of April 2001 and June 2018 demonstrate that the maximum on-site levels recorded in June 2018 are significantly lower than those experienced in April 2001. Therefore, it can be concluded that the June 2018 levels do not represent the true maximum groundwater levels experienced at the site.

5.8.6 To construct a higher spatial resolution groundwater level dataset around the airport during the maximum April 2001 event, an uplift factor was applied to the high-spatial resolution on-site June 2018 data. This was acquired by comparing the observed groundwater levels in the EA monitoring boreholes within 3km of the site between June 2018 and April 2001. The differences between the two months in each EA borehole was recorded. **Appendix D** of this report further details the methodology behind how these uplift factors were generated.

5.8.7 Groundwater levels tend to fluctuate more at the groundwater divide (within the interfluves) than within the valley bottoms. Putteridge Bury, located along the central axis of the groundwater divide shows a difference of 11m between the April 2001 and June 2018 levels. In contrast the monitoring of the Mimram river in the valley bottom (Mimram 1) shows a difference of only 1.3m. The relationship between groundwater level elevation differences between April 2001 and June 2018 and the distance from the groundwater divide exhibits linear behaviour and is shown on **Figure 13** in **Appendix A** to this report. Therefore, the uplift factor for each June 2018 groundwater level was assigned based on the distance of the monitoring borehole from the axis of the divide.

5.8.8 The "uplifted" data was then used to synthesise a contour plot of predicted maximum groundwater levels from site for the extreme winter groundwater high of April 2001, as shown in **Figure 14** in **Appendix A** to this report.

Assessment of 1 in 5, 1 in 10 and 1 in 25-year maximum groundwater events

5.8.9 April 2001 is considered a peak maximum event from the Hertfordshire model, and has been used in the design of the excavations. To interpret "more likely" high groundwater events, contour plots for a 1 in 5-year, a 1 in 10-year and a 1 in 25-year were also created for comparison for the following months:

a. March 2014, is calculated as representative of a 1 in 25-year maximum event, shown in **Figure 15** in **Appendix A** to this report;
b. March 2003, is calculated as representative of a 1 in 10-year maximum event shown in **Figure 16** in **Appendix A** to this report; and
c. March 2013, is calculated as representative of a 1 in 5-year maximum event, shown in **Figure 17** in **Appendix A** to this report.

5.8.10 Statistical analysis of the groundwater elevation frequency was undertaken. More details about this and how it has been applied to groundwater elevation frequency is discussed further in **Appendix D** to this report.

Assumptions and validity of method

5.8.11 For the purpose of this assessment the following assumptions have been made:

a. The April 2001 contours show higher groundwater levels in comparison to the April 2001 output from the Hertfordshire model, especially within the interfluves where groundwater levels are up to 10m higher. It is thought that this is due to the current soakaways on-site, which are shown to increase local groundwater levels. Therefore, the uplifted April 2001 contours underneath the site are exceptionally high when compared to the Hertfordshire model output contours for the same month. However, as a consequence, the results from this assessment are expected to be overly conservative, and the assumed groundwater levels are suitable for assessment of impacts.
b. The statistical analysis software applied is usually utilised for fluvial flood analysis and has been adopted as a proxy to determine groundwater level frequency. The software applies statistical distributions to a set of numbers to allow derivation of events of different probabilities and although the software is usually utilised for fluvial flood analysis it is considered appropriate for analysis of groundwater level frequency.
c. An analytical uplift factor has been applied linearly based on distance of monitoring borehole from the axis of the groundwater divide. It is noted that this may be an over-simplistic way of representing the groundwater levels and flow within the catchment but is considered to be a robust approach.

5.8.12 Due to lack of groundwater monitoring data from April 2001 at the Proposed Development, it is not known what the exact groundwater levels were during this maximum event. However, it is anticipated that the new contours generated as part of this assessment do represent a conservative assessment of maximum groundwater levels at the site, for the following reasons:

a. April 2001 is considered an extreme maximum event within the EA Hertfordshire model. This is confirmed by the groundwater hydrographs from EA observation boreholes, shown in **Figure 6** in **Appendix A** to this report, which shows that groundwater levels during the winter of 2000 to

2001 to be up to 3m higher than any other groundwater levels recorded during monitoring history (50 year record).

b. The contours from this assessment are up to 10m higher than the April 2001 maximum contours from the Hertfordshire model. Although this in part may be due to on-site discharges to groundwater, the groundwater levels from this assessment are considered to be exaggerated as a consequence of the methodology used, as such they can be considered to be conservative and are therefore appropriate for use in the design of the below ground elements of the Proposed Development.

5.8.13 Further monitoring throughout the period of construction and operation of the Proposed Development is recommended to confirm the groundwater fluctuations as well as responses during high groundwater levels.

5.9 Updated hydrogeological conceptual model

5.9.1 The figurative hydrogeological conceptual site model is shown in **Figure 8** in **Appendix A** to this report. The Chalk aquifer is the main aquifer underlying the Proposed Development. The flow through this geology is predominantly through fractures and associated dissolution features.

5.9.2 Hydraulic conductivity within in the Chalk shows a variation with depth. Hydraulic conductivity in the Chalk underlying the Main Application Site appears to be slightly lower than the regional average (Ref. 2), with the on-site packer testing showing evidence of fracture infill. In the top 20m of the Chalk, where the groundwater table is located, conductivities at the Main Application Site are shown to be, on average, 2.37×10^{-5}m/s. At 40 to 52m from the top of the Chalk, average conductivities are two orders of magnitude lower at 3.36×10^{-7}m/s. This is likely due to the presence of more permeable zones associated with fractures and increased dissolution features that occur within the typical range of fluctuation in water table levels at the top of the Chalk.

5.9.3 Groundwater divides are common in the Luton area, one is located in the vicinity of the existing airport separating the River Lee catchment from the River Mimram catchment. The axis of the groundwater divide is shown in the Hertfordshire model contour outputs in **Appendix B** to this report, and is located along the eastern edge of the existing airport. In addition to the groundwater divide separating the two river catchments, on-site monitoring data shows there to be local increases in groundwater levels as a result of the on-site soakaways.

5.9.4 From the assessment carried out as part of this report, maximum baseline groundwater levels beneath the Main Application Site are expected to range from 134mAOD in the centre of the groundwater divide to 112mAOD in the dry valleys.

5.9.5 The contours from this assessment are up to 10m higher than the published maximum contours from the Hertfordshire model. Due to the large difference between the two, the groundwater levels from this assessment are considered conservative and therefore are considered appropriate for use in the design of the below ground elements of the Proposed Development.

5.10 Prediction of future groundwater levels as a result of climate change

5.10.1 The uncertainty with the impacts of climate change makes it difficult to predict the likely changes in groundwater levels in the future. Future groundwater levels will be affected by both long-term climatic changes in precipitation, evapotranspiration and effective recharge as well as other responses such as changes to groundwater abstraction as a result of increasing demand and changes to recharge as a result of changes to land-use.

5.10.2 Trends in the recent past have shown that the UK climate is continuing to warm. The UK Climate Impact Programme 2018 (UKCP18) (Ref. 27) provides the most recent climate predictions, which are as follows:

a. average summer temperatures are estimated to increase by 5°C, whilst the average winter temperatures are estimated to increase by 3.4°C (both 50th percentile);
b. the average summer rainfall rate is estimated to decrease by 30%, whereas the average winter rainfall rate is estimated to increase by 31% (both 50th percentile); and
c. an overall increase in extreme weather events.

5.10.3 The UK Groundwater Forum (Ref. 28) states the following concerning the expected changes in groundwater changes. In the long term, it is expected that groundwater recharge may reduce but greater variability in rainfall could mean more frequent and prolonged periods of high or low water levels. The effects of climate change on groundwater in the UK therefore may include:

a. a long-term decline in groundwater storage;
b. increased frequency and severity of groundwater droughts; and
c. increased frequency and severity of groundwater-related floods.

5.10.4 The Future Flows and Groundwater Levels (FFGL) project (Ref. 29) provides an assessment of the impact of climate change on river flows and groundwater levels across England, Scotland and Wales, using climate projections from UKCP09. The closest catchment monitoring point used for the FFGL project is located at Therfield Rectory. It has been chosen as illustrative of the potential changes at the airport for the following reasons:

a. Therfield Rectory borehole is the closest catchment monitoring point to the airport, at approximately 25km north east of the site;
b. the 84m deep well at Therfield Rectory is located on the same bedrock geology and aquifer as that of the airport; Lewes Nodular Chalk Formation and Seaford Chalk Formation (undifferentiated);
c. water levels have been monitored at Therfield Rectory since 1883 and it is one of the few sites in the UK with continuous monitoring prior to 1900; and
d. the BGS have developed a groundwater model to simulate groundwater levels at Therfield Rectory from 1961 to 2006, which obtained a

satisfactory degree of calibration to the observed levels in the borehole. This groundwater model was then used to forecast future groundwater levels under a variety of climate change scenarios.

5.10.5 The results of the FFGL project analysis of the groundwater levels at Therfield Rectory suggest that future levels could range from +3.0m to -1.0m in winter and spring months, or +2.5m to -1.5m in summer months by 2050, depending on the climate scenario used. As the climate scenario most representative of future climate is not known, it cannot be concluded which groundwater level scenario is most likely.

5.10.6 There is the potential for a long-term increase in groundwater levels overall in the Luton area due to a move to reducing abstraction pressures in the catchment. Conversely, climate change may drive land use changes and an increase in abstraction that would result in a reduction of groundwater levels.

5.10.7 At present, the effects of climate change are uncertain. Although short term and seasonal fluctuations may become more variable.

6 HYDROGEOLOGICAL IMPACTS FROM THE PROPOSED DEVELOPMENT

6.1 Proposed earthworks and construction activities

6.1.1 To facilitate the Proposed Development a range of earthworks activities would need to be undertaken, which are likely to include the following:

a. levelling and preparation of a suitable site platform so the expanded airfield would be level with the runway;
b. excavation and relocation of material from an area of the former Eaton Green Landfill; and
c. landscaping.

6.1.2 Currently absolute maximum groundwater levels are expected to be at 134mAOD at the western extent of the excavations and at 116mAOD at the eastern extent of the excavations. All civil works and excavations have been designed to avoid this maximum groundwater level, to make sure that groundwater is not intercepted during construction or operation. Therefore, there would be no changes to groundwater flow or quantity as a result of the proposed earthworks. The only impact to groundwater quantity could be from changes in site drainage, this is detailed further in **Section 6.2**.

6.2 Impact of the proposed infiltration basins on groundwater

Drainage strategy background

6.2.1 As part of the Proposed Development, drainage systems would manage surface water runoff and discharge to ground, via a combination of two infiltration tanks, after treatment as described in the Drainage Design Statement (DDS) which is provided in **Appendix 20.4** of the ES **[TR020001/APP/5.02]**.

6.2.2 The existing drainage at the airport discharges into a combination of soakaways and the Thames Water (TW) sewage network. The new drainage system for the Proposed Development would receive part of the existing drainage from three existing soakaways to the south of the airport that would be decommissioned.

6.2.3 Two new infiltration tanks would be constructed, the locations of these are shown in **Figure 4.2** of the ES **[TR020001/APP/5.03]**. Both of these tanks would be underground, removing the requirement for open water at surface.

6.2.4 The larger of the two infiltration tanks, from hereon named the ‘Southern infiltration tank, would be located downgradient of the existing runway. This tank would be approximately 260m in length by 120m in width. This tank would predominantly be used for discharge of surface water runoff from the whole of the new Proposed Development. The design of the tank includes 75,000m^3 of storage capacity.

6.2.5 The smaller infiltration tank, from hereon named the ‘Northern infiltration tank, is approximately 120m in length by 60m in width. This tank would be used for the discharge of treated sewage and treated surface water run-off where required (such as runoff where de-icing products may be present). The design of the tank includes 15,600m^3 of storage capacity.

6.2.6 A new Water Treatment Plant (WTP) would be provided close to the Northern Infiltration tank to treat both contaminated run-off from the runway activities and all sewage generated at the airport.

6.2.7 The drainage system includes a large underground attenuation tank system of 70,900m^3 volume, to capture an airside first flush event and to provide a degree of redundancy in the system.

6.2.8 The drainage arrangements for the Proposed Development have been designed to accommodate the maximum groundwater levels (April 2001) plus a 40% allowance for climate change as outlined in **Appendix 20.4** of the ES **[TR020001/APP/5.02]**.

6.2.9 The infiltration tanks have been designed to an infiltration rate of 0.085m/hr, which corresponds to the hydraulic conductivity in the top 20m of the Chalk, acquired from on-site permeability testing. Actual infiltration rates would be confirmed following further site investigation including permeability testing, during detailed design.

6.2.10 All underground tanks (storage and infiltration) have been designed with the bottom of the tanks at least 1m above the April 2001 maximum water table level.

6.2.11 Concentrating natural recharge in a small area can cause local water table levels to rise in a process called groundwater mounding. The potential groundwater mounding from each infiltration tank is based on the following physical variables:

a. Hydraulic conductivity - based on packer test data, and an important variable in determining the groundwater mounding beneath an infiltration tank.
b. Specific yield (or effective porosity) – which determines how much water can be stored in the unsaturated zone.
c. Tank shape and depth – the infiltration rates from the tank are directly related to both surface area and impounded water depth.
d. Depth to water table – a thicker unsaturated zone can store more water and would therefore reduce and/or delay the infiltrating water reaching the water table.

6.2.12 The DDS provided as **Appendix 20.4** of the ES **[TR020001/APP/5.02]** indicates that an assessment is therefore required to confirm: 1) whether the infiltration tanks would work at their locations; and 2) if so, to determine the potential groundwater mounding beneath the tanks and the impact on the local water environment.

Mounding assessment methodology

6.2.13 The Hantush (1967) analytical equation for assessing mounding beneath an infiltration tank has been utilised in assessing the magnitude and extent of groundwater mounding beneath the two new infiltration tanks proposed as part of the Proposed Development. The Hantush method is described in the 1967 paper titled "*Growth and Decay of Groundwater-Mounds in Response to Uniform Percolation*". The United States Geological Survey (USGS) have created a spreadsheet (Ref. 30) to numerically integrate the equations presented by Hantush and this spreadsheet has been used in this assessment.

6.2.14 The Hantush equation is used to assess a short-term recharge event. It is anticipated that the peak flows into the infiltration tanks mentioned above would be of short duration and therefore the Hantush equation is suitable for use in this assessment of mounding beneath the infiltration tanks.

6.2.15 The Hantush equation assumes that water is directly discharged into groundwater. In reality, there would be an unsaturated zone between the base of the infiltration tank and the groundwater table. A thicker unsaturated zone would store more water and therefore delay the water reaching the water table and prevent peaky, responsive mounding underneath the tank. Neglecting the unsaturated zone in this model is considered a conservative approach to predicting the groundwater mound beneath the tank.

6.2.16 No losses due to evapotranspiration have been factored into this assessment. In storm events evapotranspiration is anticipated to be relatively small due to rapid runoff; in addition, the infiltration tanks would be underground structures and evapotranspiration from any standing water within the tanks is not expected.

Mounding assessment results

6.2.17 During peak groundwater levels of April 2001 (refer to **Figure 14** in **Appendix A** to this report) the maximum water table beneath the Southern infiltration tank was modelled at approximately 117mAOD (15mBGL) at the western basin extent and approximately 114mAOD (5mBGL) at the eastern tank extent. Note that due to the size of the tank there is a 3m reduction in water table from west to east. At the Northern infiltration tank, the groundwater level was approximately 120mAOD.

6.2.18 The invert level of the Southern infiltration tank is set at 116.8mAOD and the invert level of the Northern infiltration basin is set at 121.795mAOD. On this basis, during peak groundwater levels there would be on average at least 1m of unsaturated thickness below the infiltration tanks. During typical summer groundwater conditions, the unsaturated zone would exceed 10m at the Southern infiltration tank.

6.2.19 Under long-term average rainfall conditions, the discharge to the Southern infiltration tank is predicted to cause localised mounding of 2.7m. At the Northern infiltration tank, the discharge is predicted to cause localised mounding of 5.7m. The inputs and outputs for this assessment, included assessed mounding extents, are shown in further detail in **Appendix E** to this report.

6.2.20 As the invert of the infiltration tanks is designed a minimum of 1.0m above the assessed peak groundwater level (April 2001), groundwater mounding during extreme groundwater level events has the potential to rise to the base of the infiltration invert. In the case of a two-hour storm event with a return period of 1:100 years when groundwater levels are at their peak, then the design storage of the tank would accommodate the full runoff volume allowing infiltration to continue without groundwater flooding occurring.

6.2.21 When de-icing is used on the airport apron, run-off would be monitored and potentially treated (if trigger levels are exceeded) by the WTP. During these times, the 70,900m^3 and 15,600m^3 storage in the infiltration tanks would be used to buffer the peak discharge rates.

6.2.22 No receptors within the mounding zone of influence of the infiltration tanks are considered to be impacted by the localised mounding.

Groundwater Quality

6.2.23 The Proposed Development is located on a Principal Chalk Aquifer and within the Source Protection Zone 3 (Total Catchment) of public water supplies in the area. Any contaminated discharge to ground is a potential risk to consumers, and needs to be considered in detail.

6.2.24 Discharge of treated wastewater and surface water runoff is discussed in detail in the Hydrogeological Risk Assessment: Drainage document provided as **Appendix 20.6** of the ES **[TR020001/APP/5.02]** and the Drainage Design Statement document provided as **Appendix 20.4** of the ES **[TR020001/APP/5.02]**.

Conclusions

6.2.25 The mounding assessments for the Southern and Northern infiltration tanks indicate that under normal conditions the combination of unsaturated zone beneath the tanks, together with storage capacity of the infiltration tanks are sufficient to accommodate the recharge mound.

6.2.26 As the invert of the infiltration tanks is designed a minimum of 1.0m above the assessed peak groundwater level (April 2001), groundwater mounding during peak storm events has the potential to rise to the base of the infiltration invert. In the case of a two-hour storm event with a return period of 1:100 years when groundwater levels are at their peak, then the design storage of the attenuation and infiltration tanks would accommodate the full runoff volume allowing infiltration to continue without overtopping occurring.

6.2.27 Because the height of the mounding is highly sensitive to aquifer properties such as the assumed hydraulic conductivity, it is important that the hydraulic conductivity at the proposed site of the infiltration tanks are properly verified with additional ground investigation as part of the detailed design after development consent is granted and prior to construction.

6.2.28 It is therefore concluded that all the factors required for an effective infiltration tank are present at the site, for all but the most extreme maximum groundwater conditions. In the most extreme condition, the storage in the infiltration tanks would be used to contain storm water before infiltration. At this stage it is only possible to asses this risk by using conservative “worst case” assumptions.

Further site investigation and verification of hydraulic conductivity is required to refine these assumptions and to confirm that the proposed infiltration tank design would work in all conditions.

6.2.29 In summary, all the important criteria that need to be considered in determining the location, size and design for an infiltration tank have been reviewed. They show that an infiltration tank would work effectively at the proposed location, providing the hydraulic conductivity of the Chalk is sufficiently high, and is demonstrated to be at least 2.37x 10^{-5}m/s or higher.

6.2.30 No receptors within the mounding zone of influence of the infiltration tanks are considered to be impacted by the localised mounding.

6.2.31 Assuming the dispersal of the groundwater mound downgradient is gradual and reflective of the calculated permeabilities, the risk of the mound being responsible for elevating groundwater levels in locations such as Kimpton is considered very low. This is based on the time it would take for the water to reach the downstream location, with the chalk attenuating the groundwater flow downstream.

6.2.32 However, the risk of the Main Application Site affecting conditions at Kimpton could increase if there is an accelerated dispersal rate. This could occur if a significant fracture flow pathway becomes active, although there is no indication that this pathway exists at the Main Application Site. However, additional site investigation works are proposed in advance of construction to assess this risk further and allow mitigation to be deployed if required.

6.2.33 Discharge of treated wastewater and surface water runoff is discussed in detail in the Hydrogeological Risk Assessment: Drainage document provided as **Appendix 20.6** of the ES **[TR020001/APP/5.02]** and the Drainage Design Statement document provided as **Appendix 20.4** of the ES **[TR020001/APP/5.02]**.

7 HYDROGEOLOGICAL CHARACTERISATION CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 The EA Hertfordshire groundwater model, which is the current groundwater level resource planning tool available in the vicinity of the Proposed Development, presents a detailed numerical assessment of the maximum April 2001 groundwater levels.

7.1.2 Groundwater divides are common in the Luton area, with one lying along the eastern extent of the existing airport, which separates the River Lee catchment from the River Mimram catchment. The axis of this groundwater divide is represented in the Hertfordshire model and can be observed in figures in **Appendix B** to this report. In addition, local discharges to ground cause localised mounding, as observed by the on-site monitoring data.

7.1.3 To evaluate maximum levels directly under the Proposed Development, an assessment of the high-spatial resolution on-site groundwater monitoring was carried out. These contours have been used as the conceptual maximum groundwater levels.

7.1.4 From this assessment, these maximum groundwater levels are estimated to range from 134mAOD in the centre of the groundwater divide to 112mAOD in the dry valleys.

7.1.5 The groundwater level contours from this assessment are up to 10m higher than the April 2001 contours from the Hertfordshire model. Due to the large difference between the two, the groundwater levels from this assessment are considered conservative and therefore appropriate for use as a maximum design value for the below ground elements of the Proposed Development.

7.1.6 It is unclear how future climate change will impact on groundwater levels though most predictions estimate that intense events may lead to times of high recharge and higher maximum groundwater events. The uncertainty regarding climate change and how it will affect groundwater recharge and subsequent groundwater levels make it difficult to provide a quantitative analysis of groundwater changes.

7.1.7 The maximum groundwater levels assessed have been used as a design level for the below ground elements of the Proposed Development. These levels are to be reviewed as additional groundwater monitoring data becomes available at detailed design. The maximum groundwater levels presented are conservative and it is therefore not anticipated that the excavation works would intercept groundwater. As a result, no impacts to baseline groundwater levels or flow are expected as a consequence of the below ground excavations.

7.1.8 A feasibility assessment shows that infiltration tanks would work effectively at the proposed location, providing the hydraulic conductivity of the Chalk is sufficiently high, and is demonstrated to be at least 2.37×10^{-5}m/s or higher.

7.1.9 The review of historical groundwater flooding has identified that although flooding occurred in 2001 in the dry valleys down gradient of the Proposed Development, no groundwater flooding has occurred in the dry valleys within the Proposed Development. Measures are included in the design of the infiltration tanks to contain treated water in the event that, during extreme storm events, the infiltration capacity of the tank is exceeded.

7.1.10 Assuming the dispersal of the groundwater mound downgradient is gradual and reflective of the calculated permeabilities, the risk of the mound being responsible for elevating groundwater levels in locations such as Kimpton is considered very low. This is based on the drainage design attenuating water during peak storm conditions and the time it would take for the water to reach the downstream location, with the chalk attenuating the groundwater flow downstream.

7.1.11 This estimate uses a permeability of 2.37×10^{-5}m/s, which is the average conductivity within the top 20m of the Chalk acquired from on-site permeability testing, and the depth at which the infiltration tank would be installed. There is some uncertainly about this estimate which appears to be quite low when compared with wider regional assessments. However, to be prudent, these conservative values has been used throughout the proposed drainage design and assessment of environmental impacts.

7.2 Recommendations

7.2.1 By using conservative or “worst case” assumptions it has been demonstrated at this stage of design, the proposals would not have a detrimental impact on the groundwater levels and flows (e.g. drawdown from construction activities or mounding through infiltration). However, in order to refine or optimise the design of the infiltration tanks and in order to validate the current assessment of maximum groundwater levels underneath the Proposed Development, it is recommended that groundwater level monitoring is continued at an increased frequency from all available monitoring installations in the near future. This is to ensure a sufficient baseline is obtained prior to commencement of construction and submission of any relevant environmental permits. The monitoring should continue throughout the construction and operation of the Proposed Development to demonstrate the groundwater impact, particularly during extreme high or low groundwater conditions.

7.2.2 In addition, it is recommended that additional ground investigation and permeability testing is carried out on site as the design progresses, especially in the vicinity of the infiltration tank.

7.2.3 The groundwater quality impacts from the Proposed Development are discussed in further detail in the Hydrogeological Risk Assessment: Drainage document provided as **Appendix 20.6** of the ES **[TR020001/APP/5.02]** and the Drainage Design Statement document provided as **Appendix 20.4** of the ES **[TR020001/APP/5.02]**.

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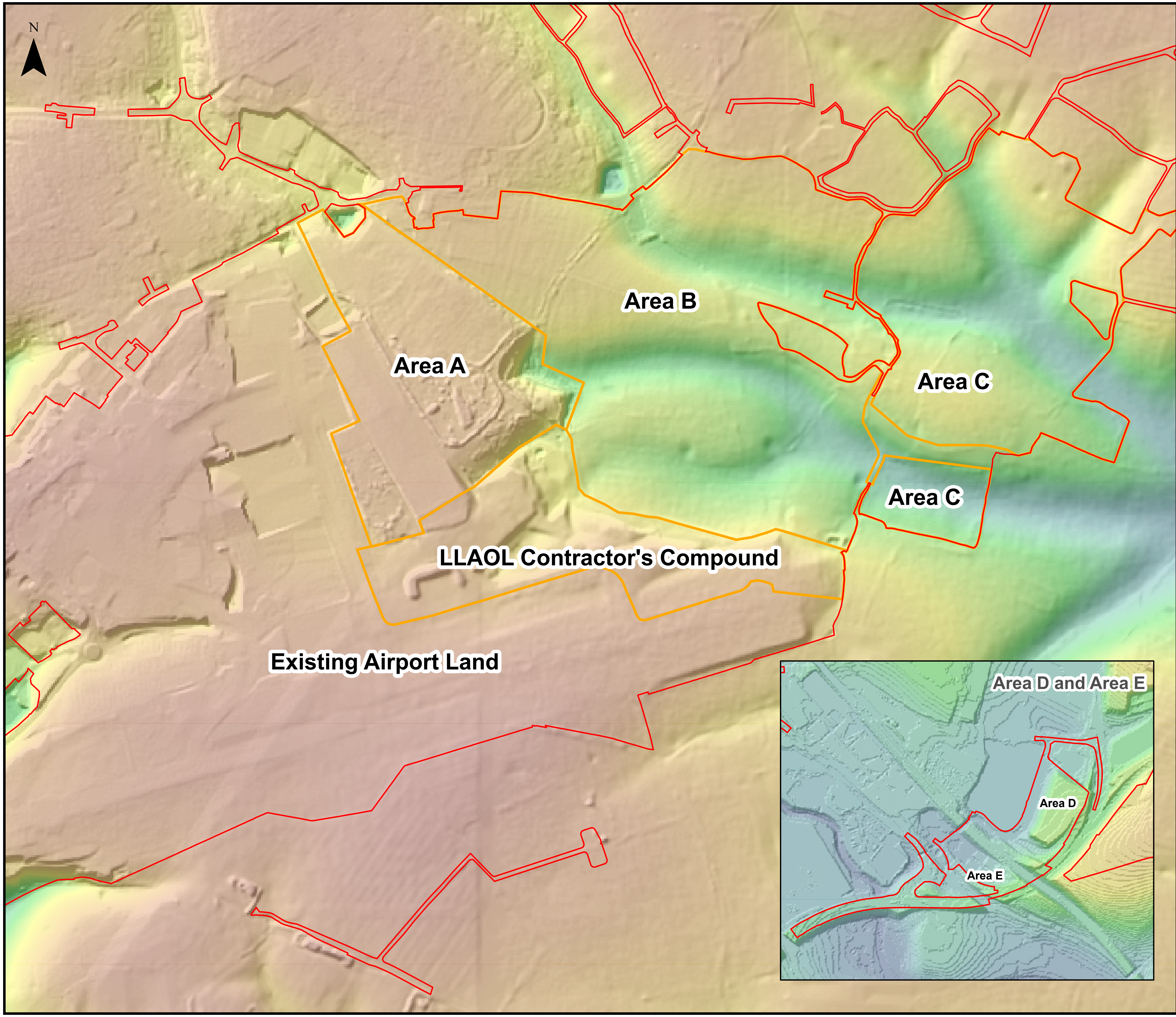
GLOSSARY AND ABBREVIATIONS

Term	Definition
Aquifer	An aquifer is a body of rock and/or sediment that holds groundwater.
Borehole	A borehole is a general term used to describe a deep hole intended to abstract or monitor water
BGS	British Geological Survey
Chalk	Chalk is a soft, white, porous, sedimentary carbonate rock. It is a form of limestone composed of the mineral calcite and originally formed deep under the sea by the compression of microscopic plankton that had settled to the sea floor.
Clay with Flints	Superficial deposits of stiff red, brown or yellow clay containing unworn whole flints as well as angular shattered fragments, also with a variable admixture of rounded flint, quartz, quartzite and other pebbles
DART	Direct-Air-Rail-Transit
DCO	Development Consent Order
DDS	Drainage design statement
EA	Environment Agency
ES	Environmental Statement
Groundwater	Groundwater is any water found beneath the surface that fills pores or cracks in the underlying soil and rocks.
Groundwater mounding	A localised increased in groundwater level.
HCR	Hydrogeological characterisation report
Hertfordshire numerical groundwater model	Numerical representation of the groundwater regime in the Hertfordshire region, developed by Mott Macdonald on the behalf of the EA
Hydraulic conductivity	Hydraulic conductivity is a physical property which measures the ability of the material to transmit fluid through pore spaces and fractures in the presence of an applied hydraulic gradient.
LBC	Luton Borough Council
mAOD	Metres above ordnance survey
mBGL	Metres below ground level
Packer testing	Packer testing is a test for measuring the permeability of ground in sections of boreholes
PEIR	Preliminary environmental information report
Permeability	A measure of the ability of a material (such as rocks) to transmit fluids

Term	Definition
River Lee	Main river located 450m to the west of the Proposed Development. A tributary of the River Thames. Upper reaches are groundwater fed.
River Mimram	Main river located 3.5km to the south-east of the Proposed Development. A tributary of the River Thames. Upper reaches are groundwater fed.
Transmissivity	The degree to which a medium allows something, in particular water, to pass through it.
WFD	Water Framework Directive

APPENDIX A – FIGURES



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Legend

Order Limits

Site Subdivisions

Topography

High : 164 mAOD

Low : 99 mAOD

mAOD: metres Above Ordnance Datum
LLAOL: London Luton Airport Operations Limited

Revision History	Drawn	Checked / Approved	Date	Rev.
First Issue	AB	TG / CS	17/02/23	P01

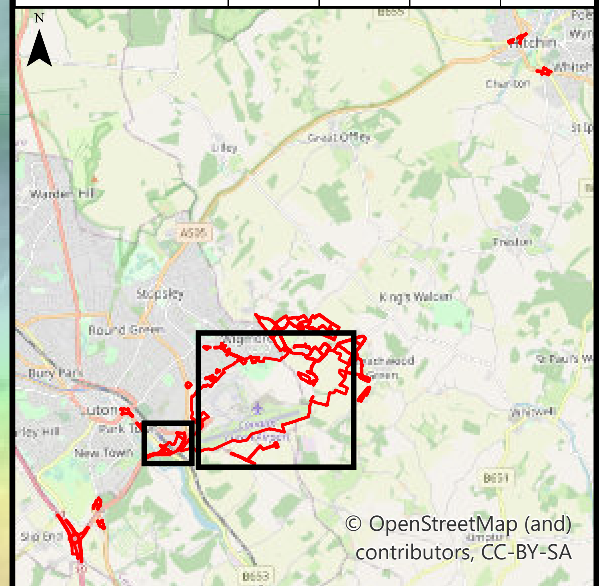

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Figure 1
Topographical map of the Proposed Development

Purpose of issue					Suitability
SUITABLE FOR INFORMATION					S2
Drawn	Checked	Approved	Date	Scale	Size
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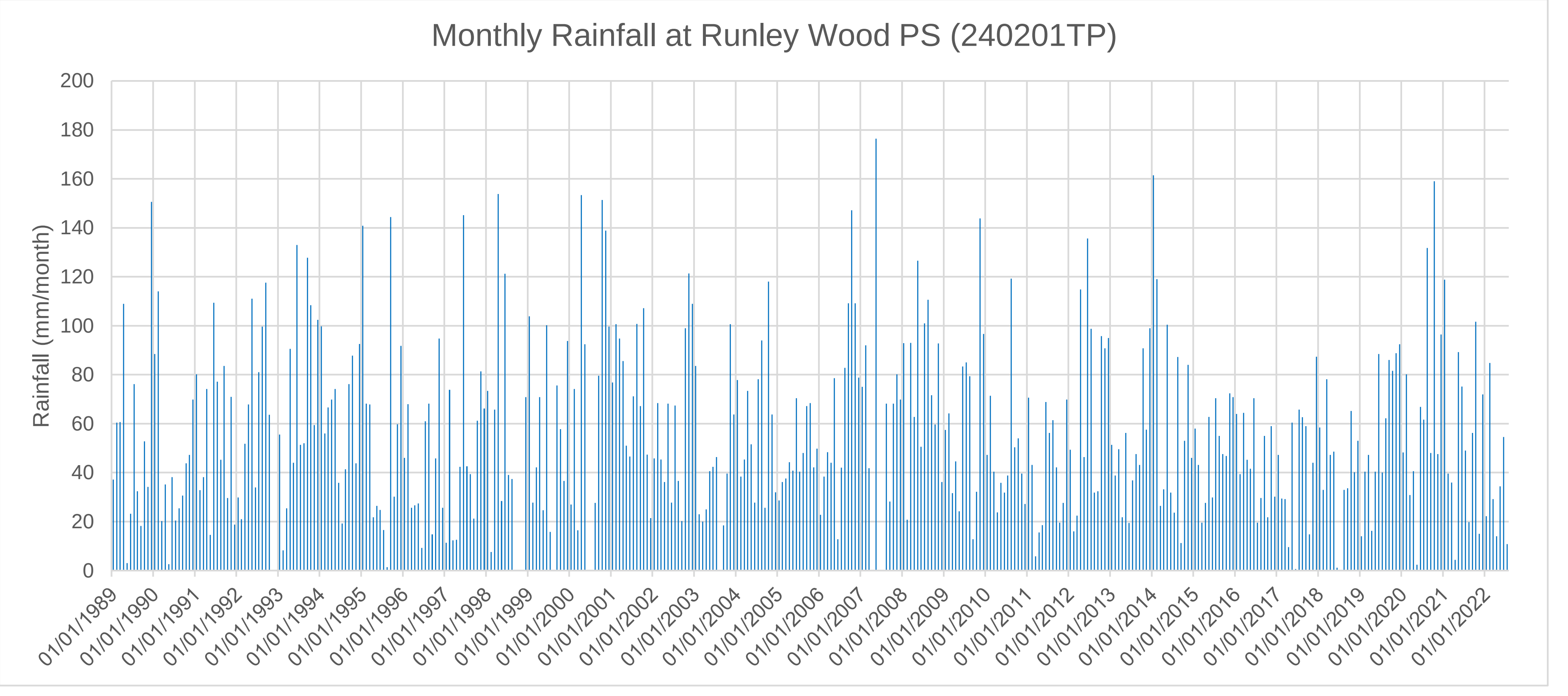
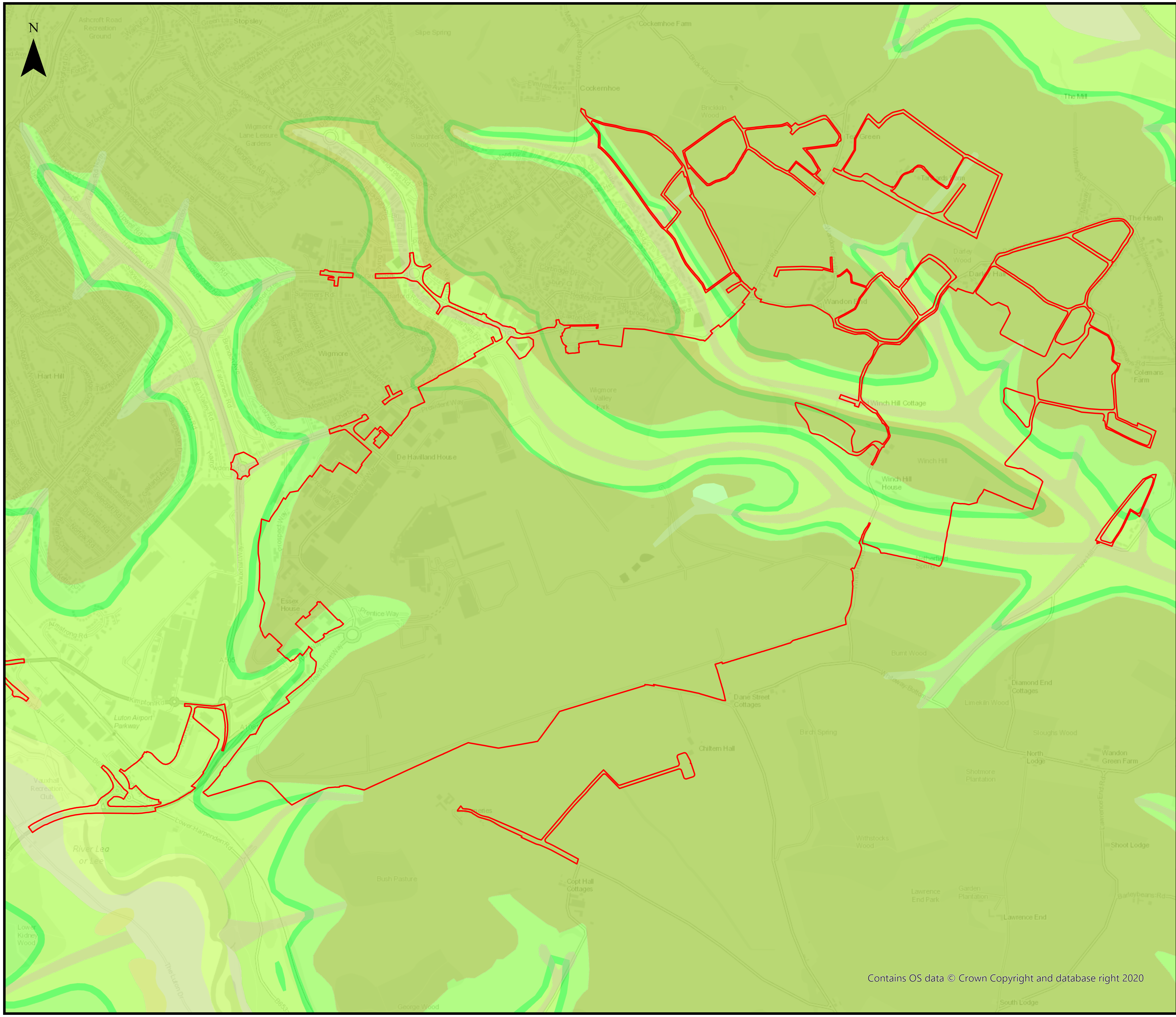
DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0232	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

Figure 2: Rainfall at Runley Wood Pumping Station





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Legend

Order Limits

Bedrock

- Holywell Nodular Chalk Formation and New Pit Chalk Formation (Undifferentiated)
- Zig Zag Chalk Formation
- Lewes Nodular Chalk and Seaford Chalk Formation (Undifferentiated)
- Gault Formation - Mudstone
- West Melbury Marly Chalk Formation
- Lambeth Group - Clay, Silt and Sand

Superficial

- Alluvium
- Clay-with-flints
- Glaciofluvial Deposits
- Head
- Till
- River Terrace Deposits

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Figure 3
Geological map of the Proposed Development

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SUITABLE FOR INFORMATION				S2	
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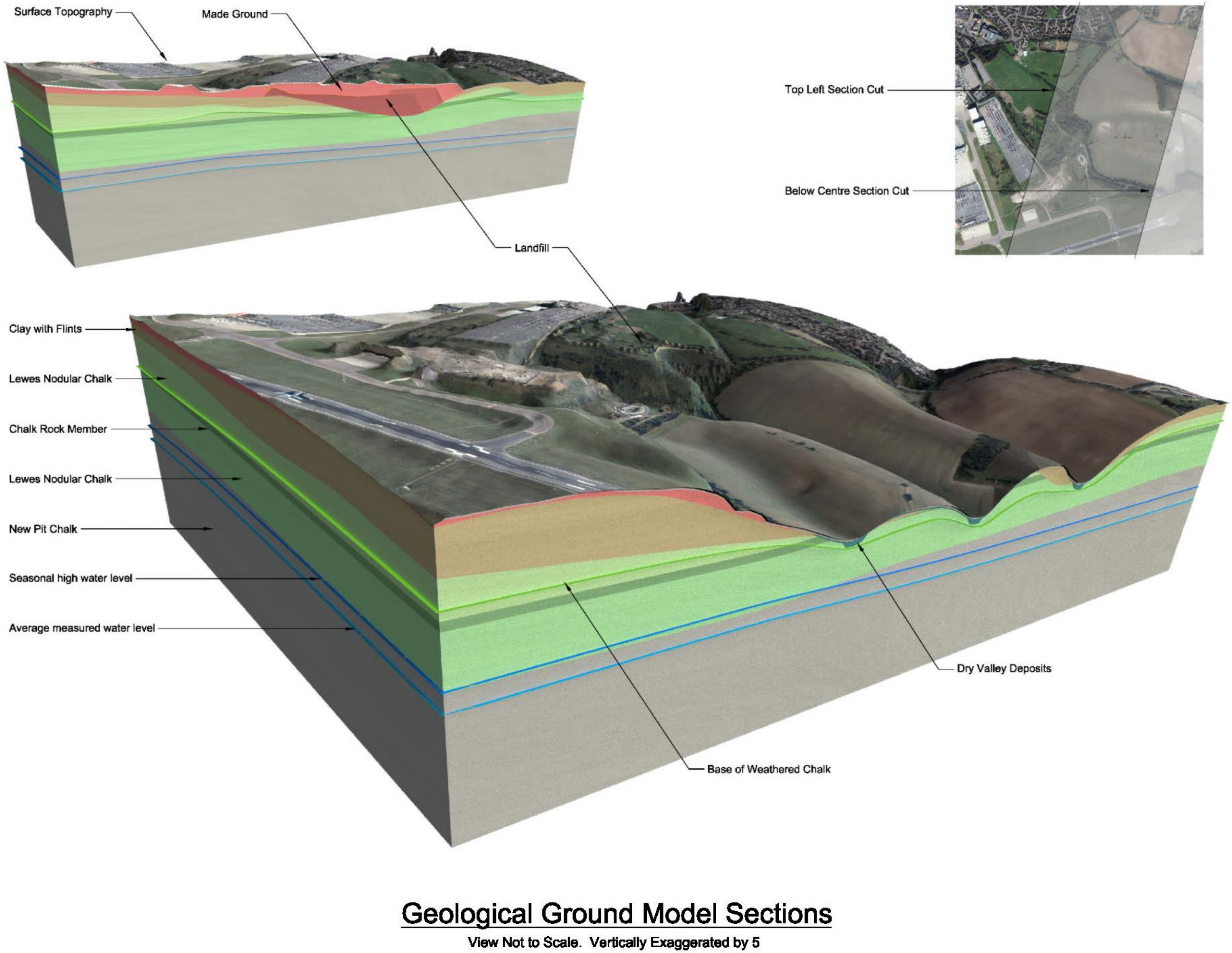
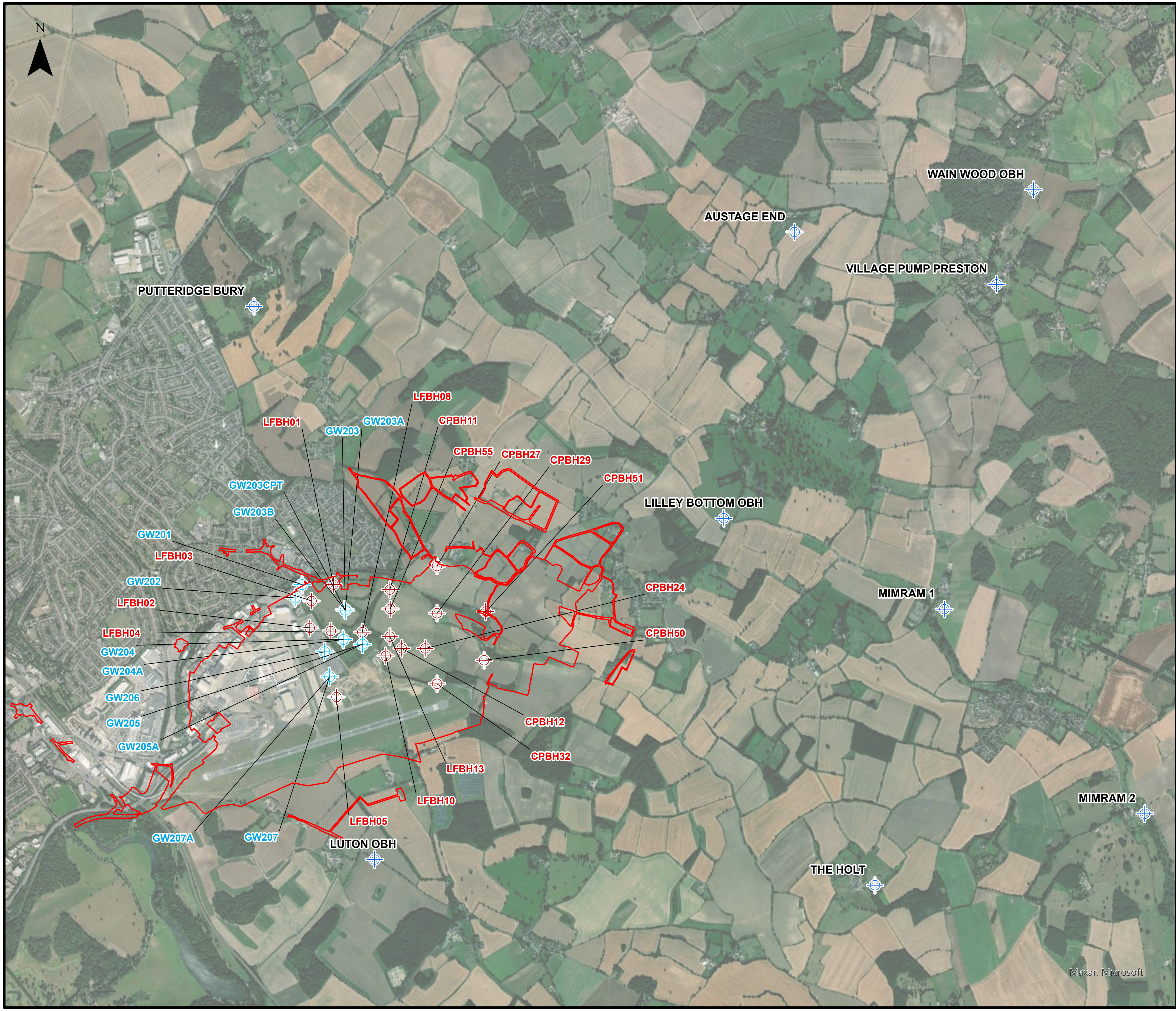
Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0234	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

Figure 4 – Ground Model of Proposed Development





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Legend

- Order Limits
- Groundwater Locations
- Arup Monitoring Locations
- Environment Agency Observation Boreholes (OBH)

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Figure 5
Groundwater Monitoring Locations

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SUITABLE FOR INFORMATION				S2	
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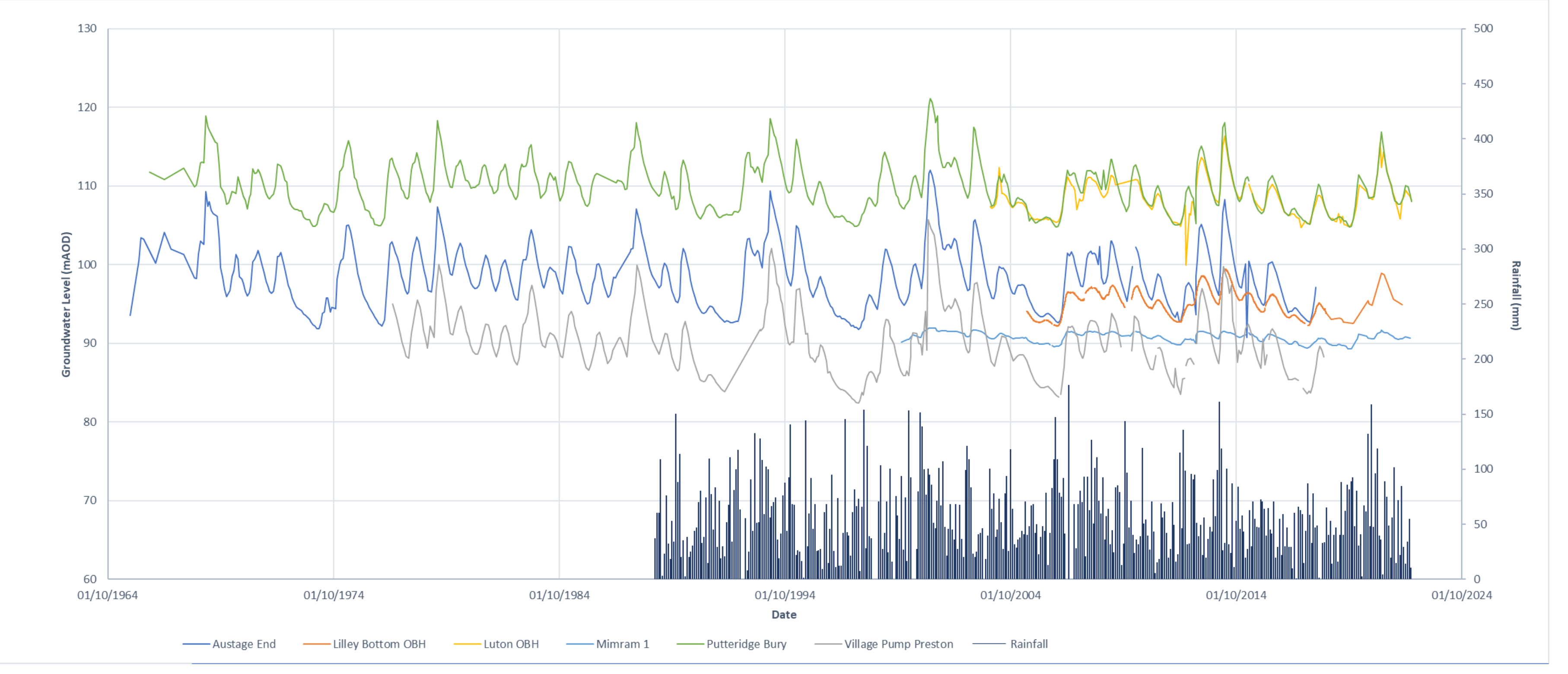
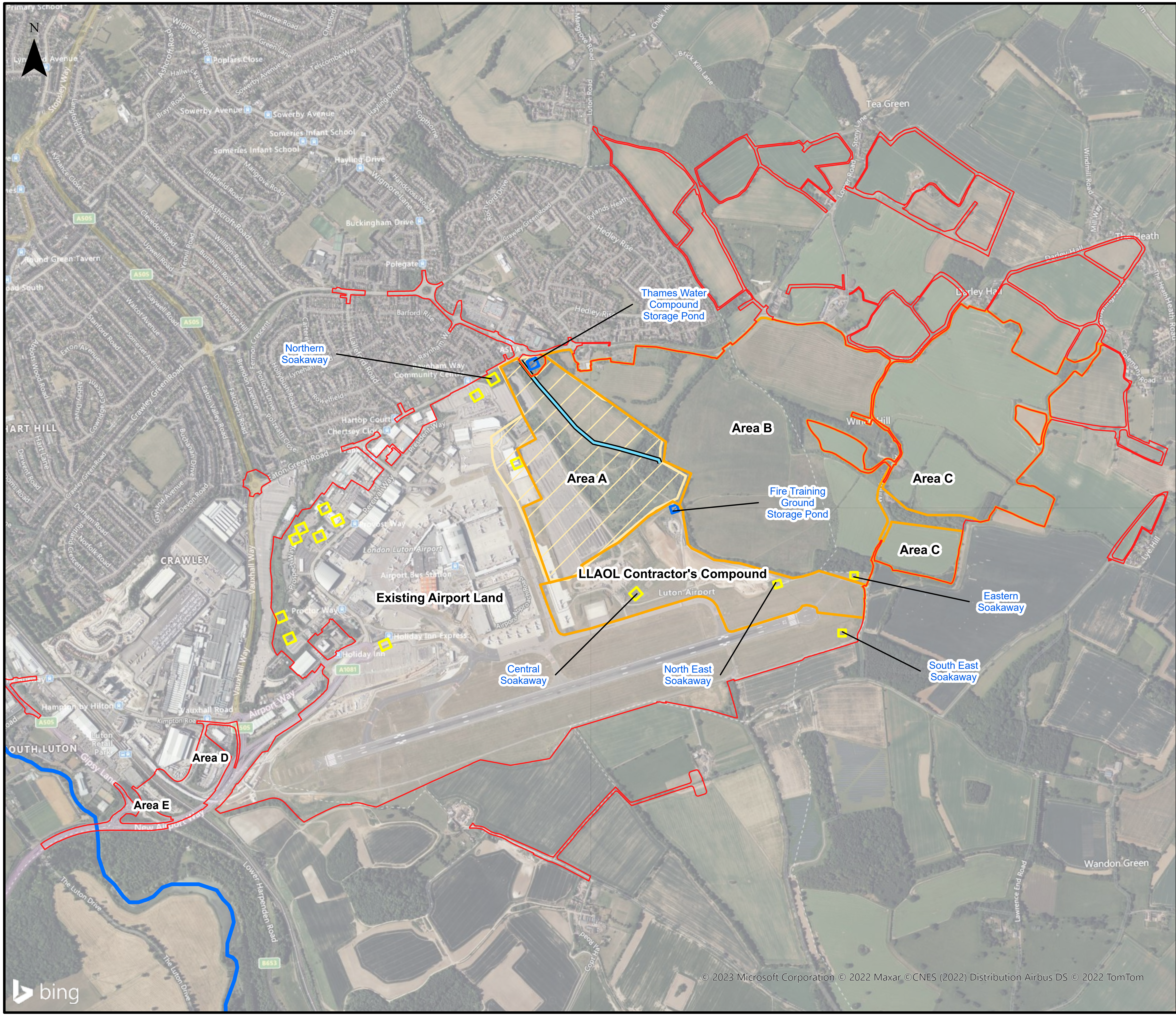
Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0235	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

Figure 6: Groundwater Hydrographs of EA Monitoring Boreholes within 3km of Proposed Development





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- Order Limits
- Site Subdivisions
- Soakaway
- Storage Pond
- Thames Valley Drain
- Interpreted Landfill Extent
- River Lee

LLAOL: London Luton Airport Operations Limited

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Figure 7
Existing Water Infrastructure

Purpose of issue				Suitability	
SUITABLE FOR INFORMATION				S2	
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DCO Application Ref.	APFP Regulation	DCO Document Ref.
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Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0236	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

Figure 8 – Conceptual Site Model of the Proposed Development

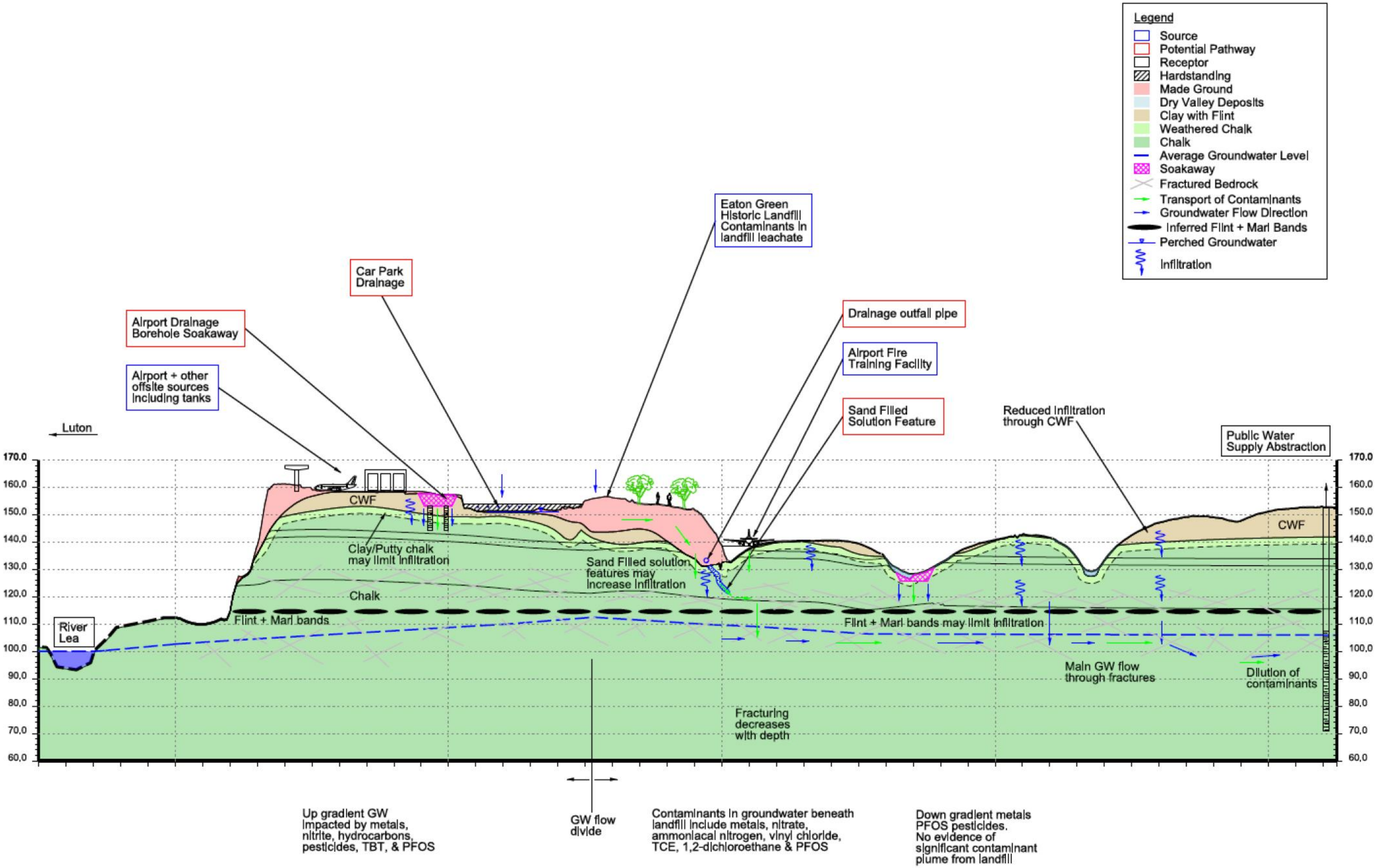


Figure 9 – Groundwater Hydrograph from Monitoring (Beneath WVG Landfill)

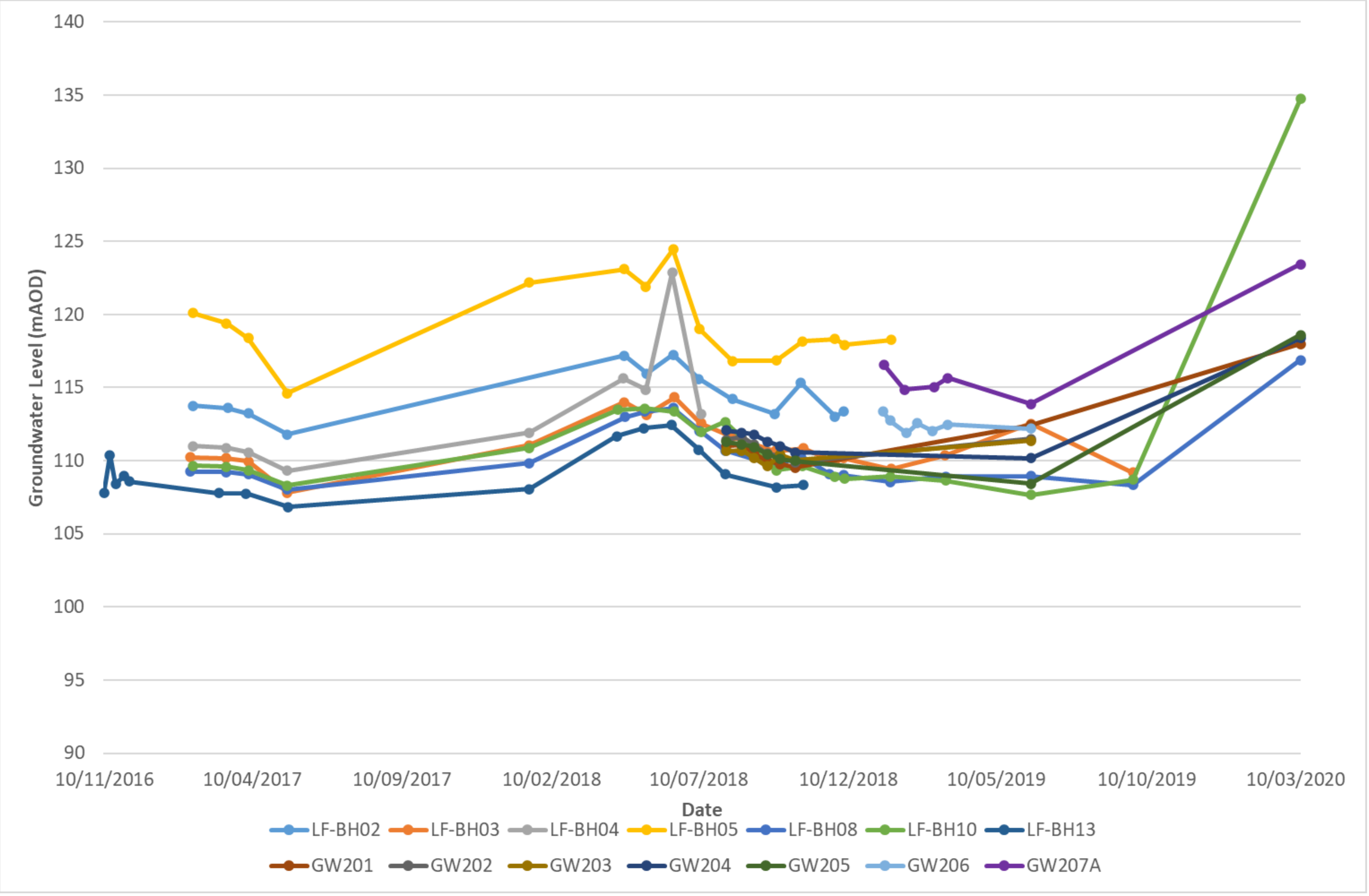
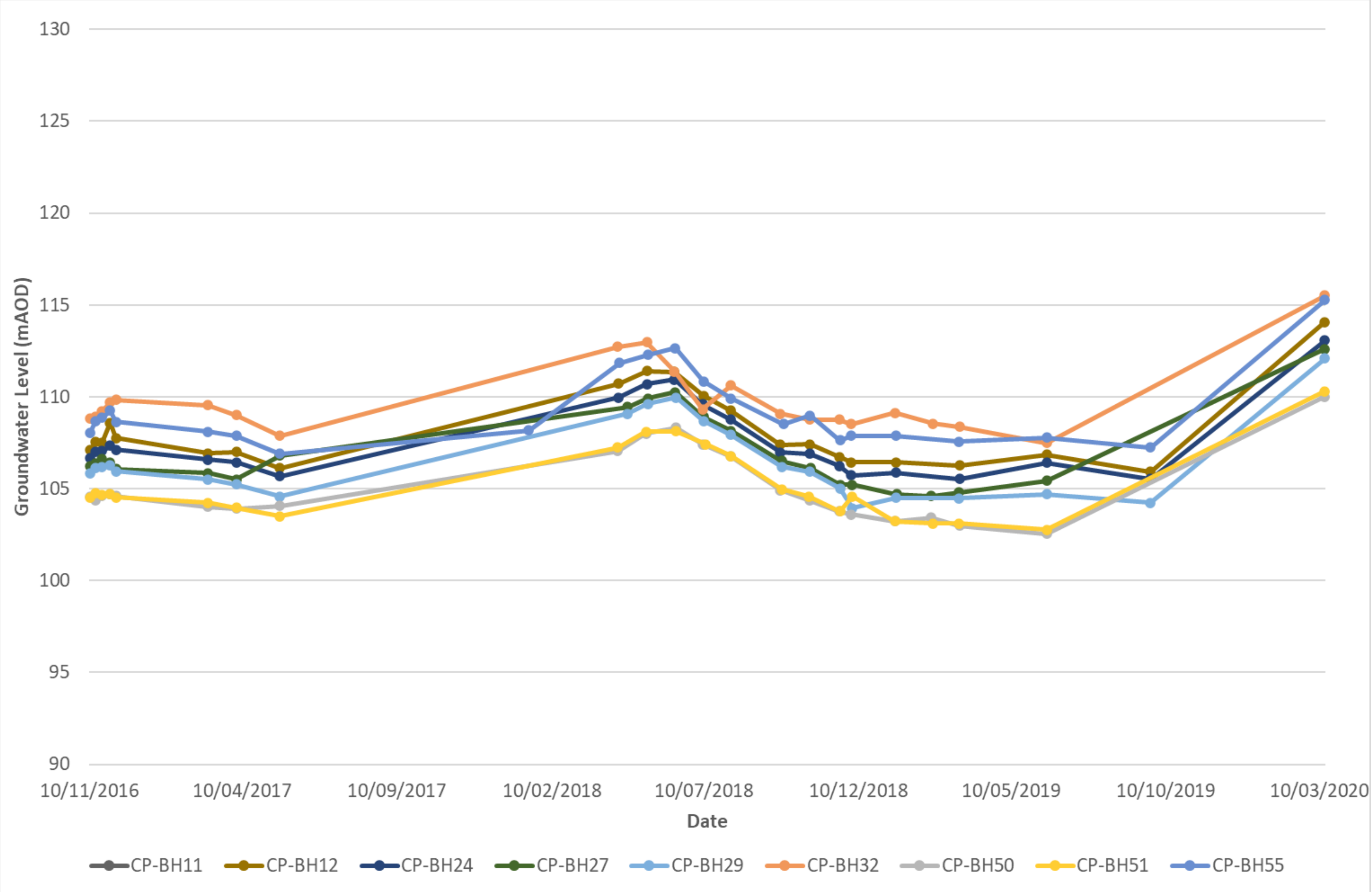
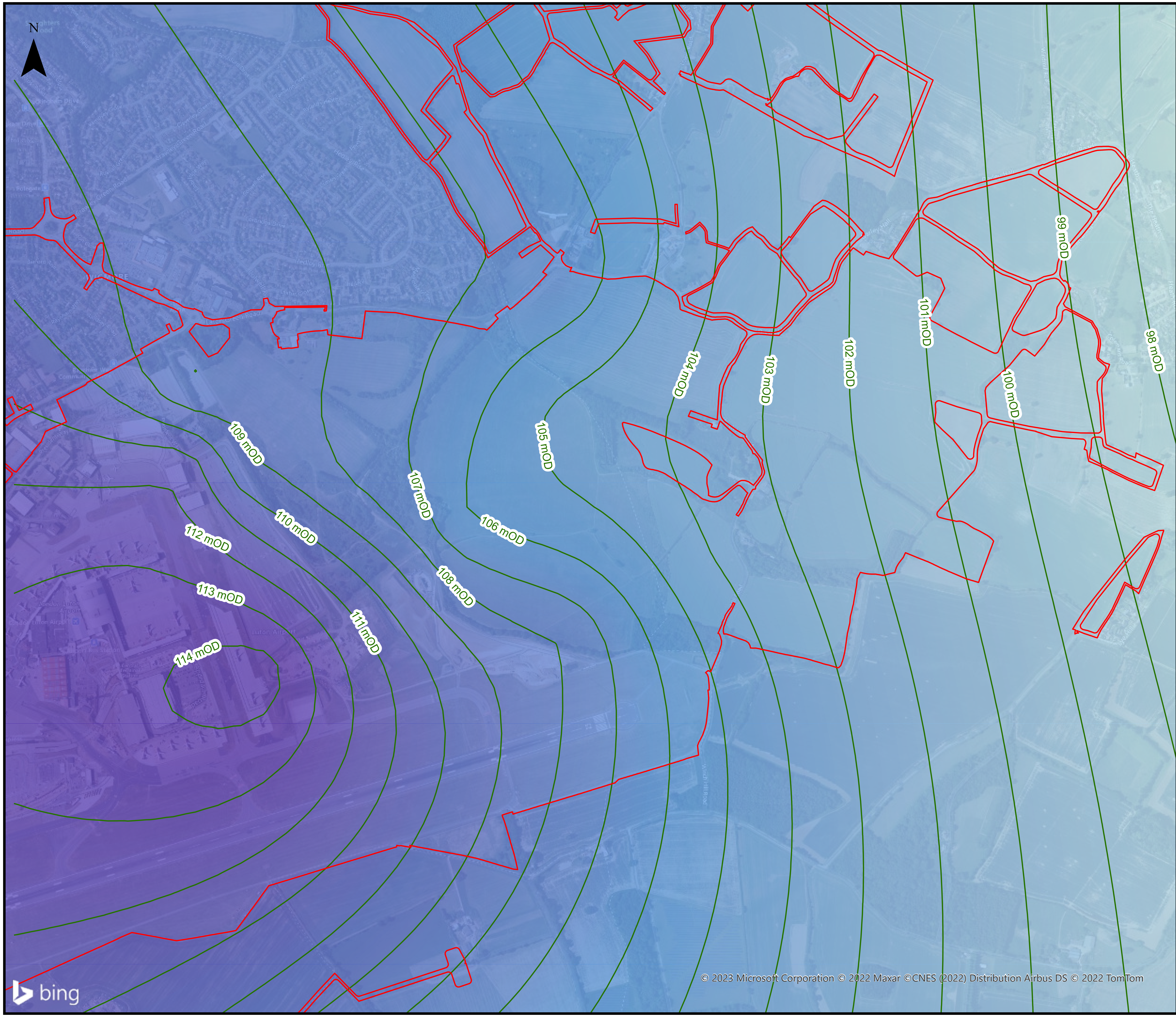


Figure 10 – Groundwater Hydrograph from Monitoring (External to WVG Landfill)





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Legend

Order Limits

1.0m Contours

Minimum Groundwater Levels

High: 114.46 mAOD

Low: 85.6 mAOD

mAOD: metres Above Ordnance Datum

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Figure 11
Minimum Groundwater Level
from Site Investigation

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SUITABLE FOR INFORMATION					S2

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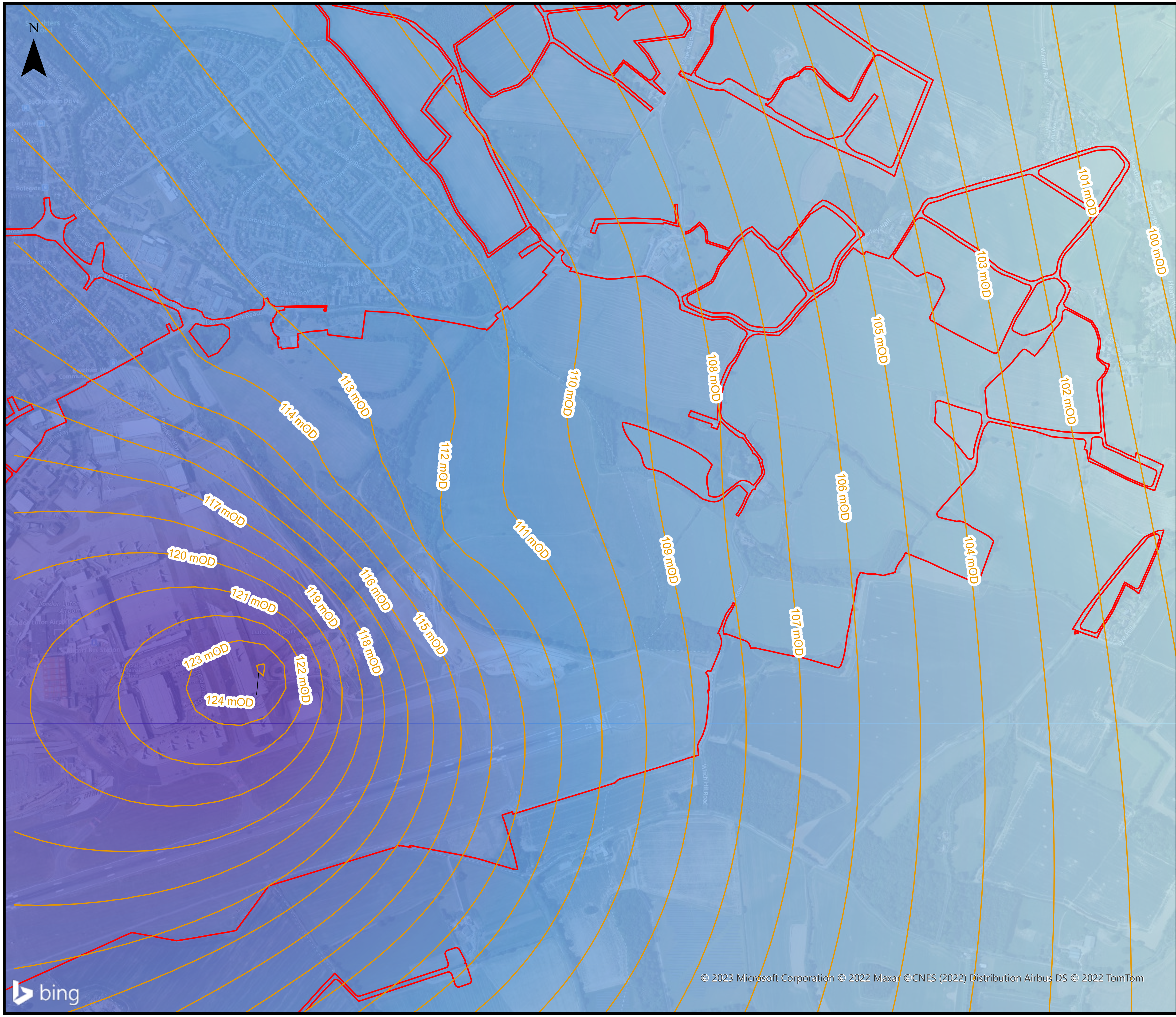

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Order Limits

1.0m Contour

Maximum Groundwater Levels

High: 124.177 mAOD

Low: 89.62 mAOD

mAOD: metres Above Ordnance Datum

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Figure 12
Maximum Groundwater Level
from Site Investigation

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SUITABLE FOR INFORMATION				S2	
Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:10,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
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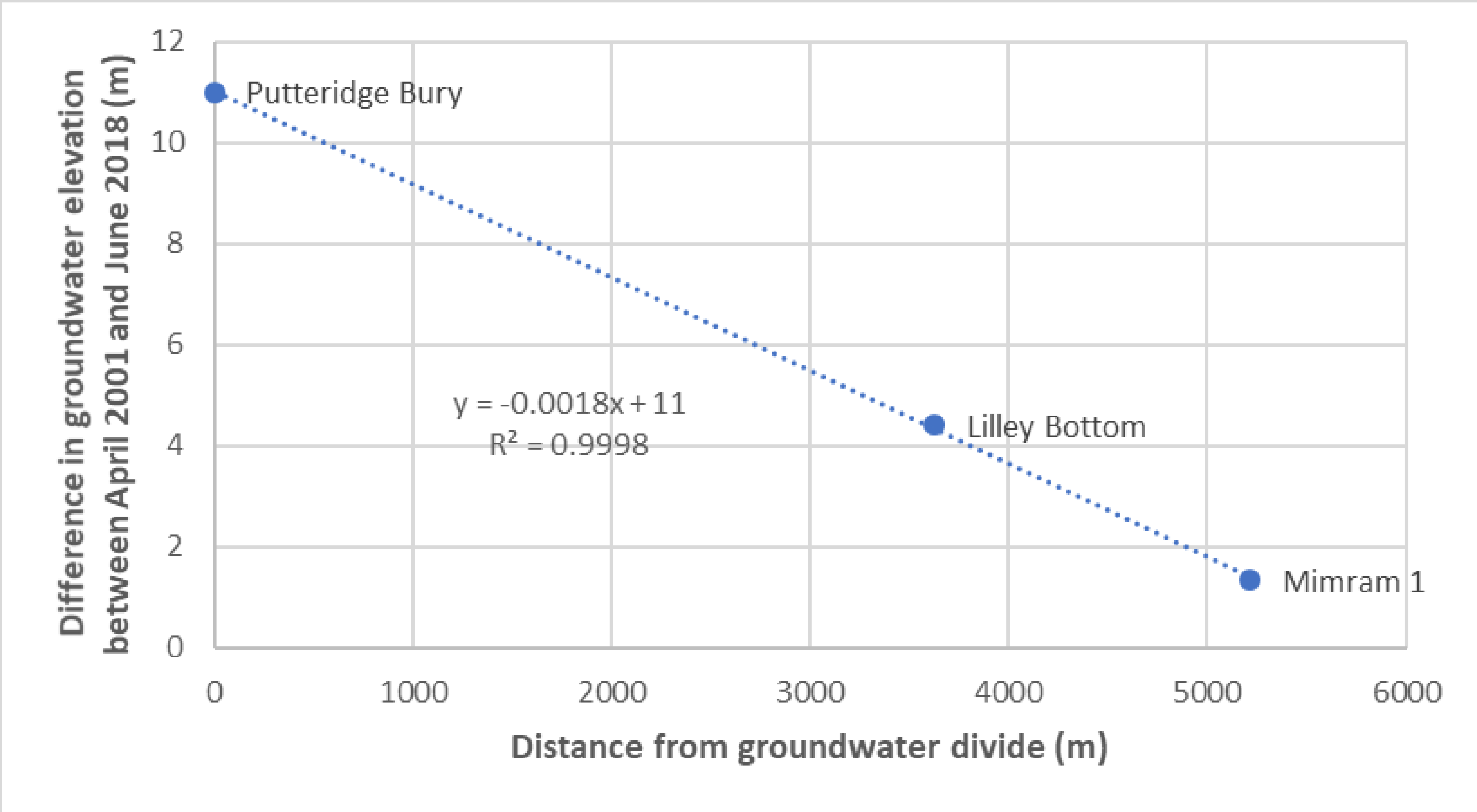
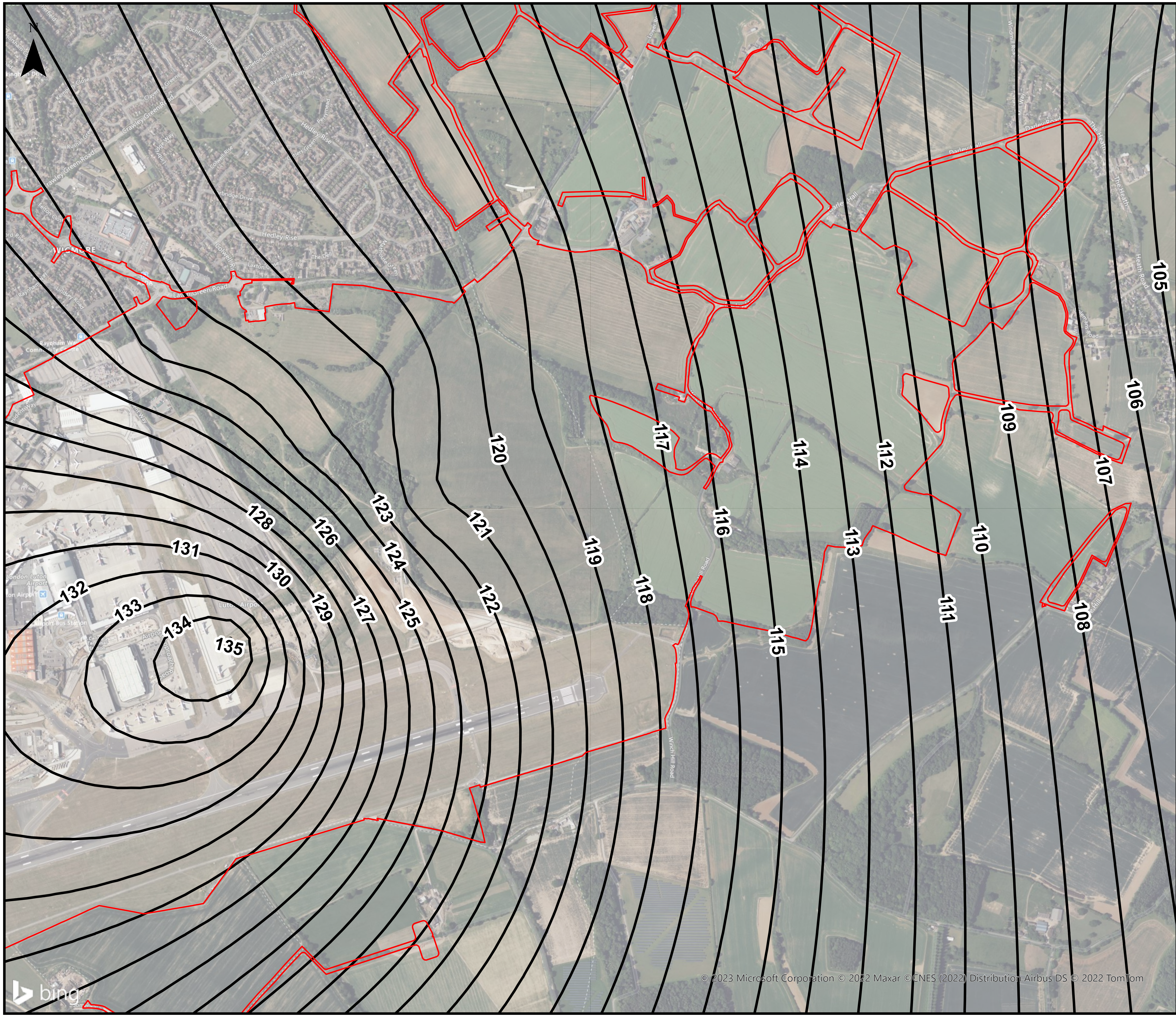
Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0230	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

Figure 13: Difference in groundwater level between the two months of April 2001 and June 2018 with the distance from the groundwater divide





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Legend

Order Limits

April 2001 1 in 100-year predicted groundwater levels (1m Contour)

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Figure 14
Predicted maximum groundwater contours for the Proposed Development based on EA offsite recorded data from extreme groundwater winter high of April 2001

Purpose of issue	Suitability
SUITABLE FOR INFORMATION	S2

Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:10,000	A3

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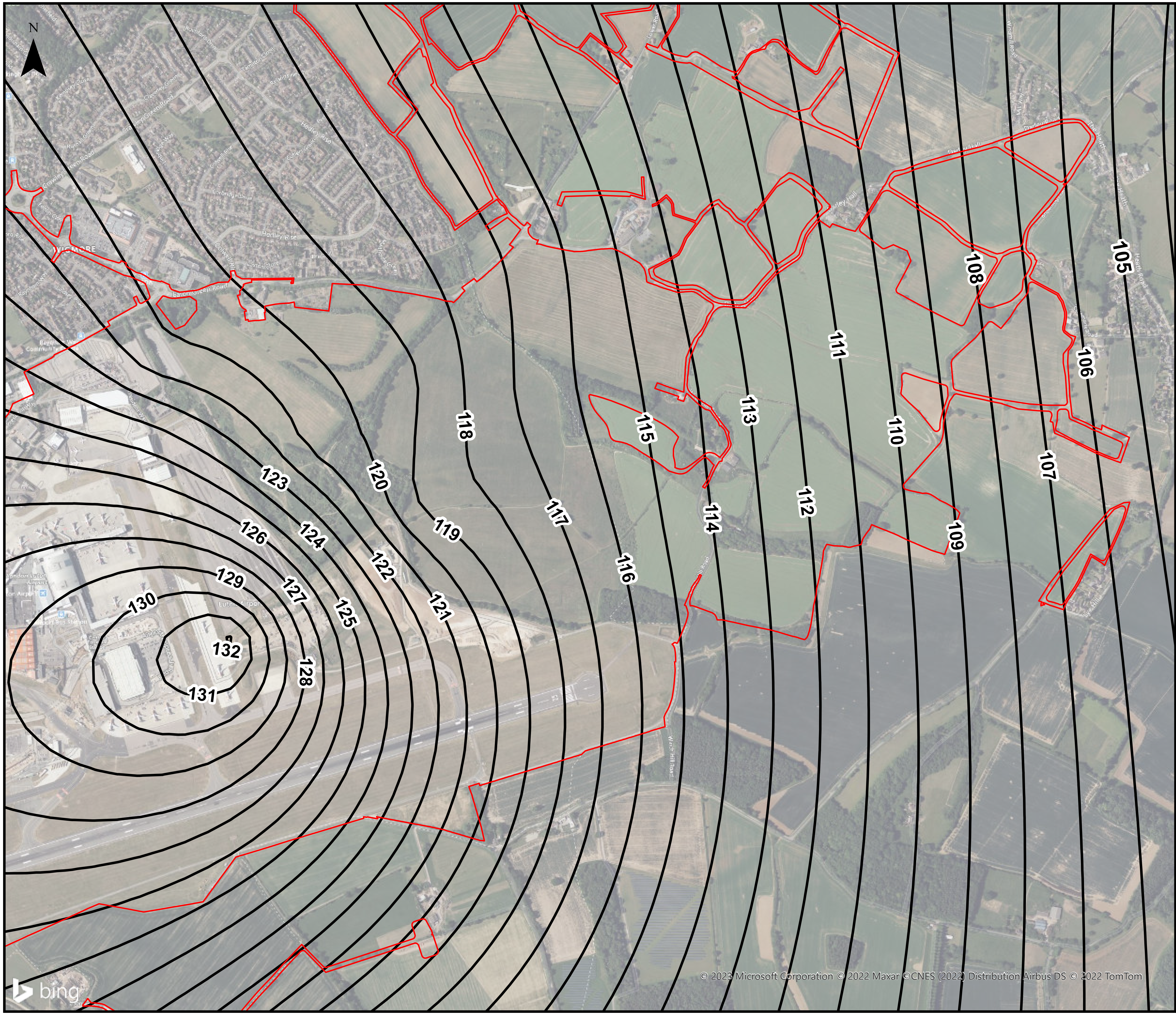
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0235	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number



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Legend

- Order Limits
- March 2014 1 in 25-year predicted groundwater levels (1m Contour)

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Drawing Title
Figure 15
1 in 25-year predicted maximum groundwater event contours (March 2014)

Purpose of issue					Suitability
SUITABLE FOR INFORMATION					S2
Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:10,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
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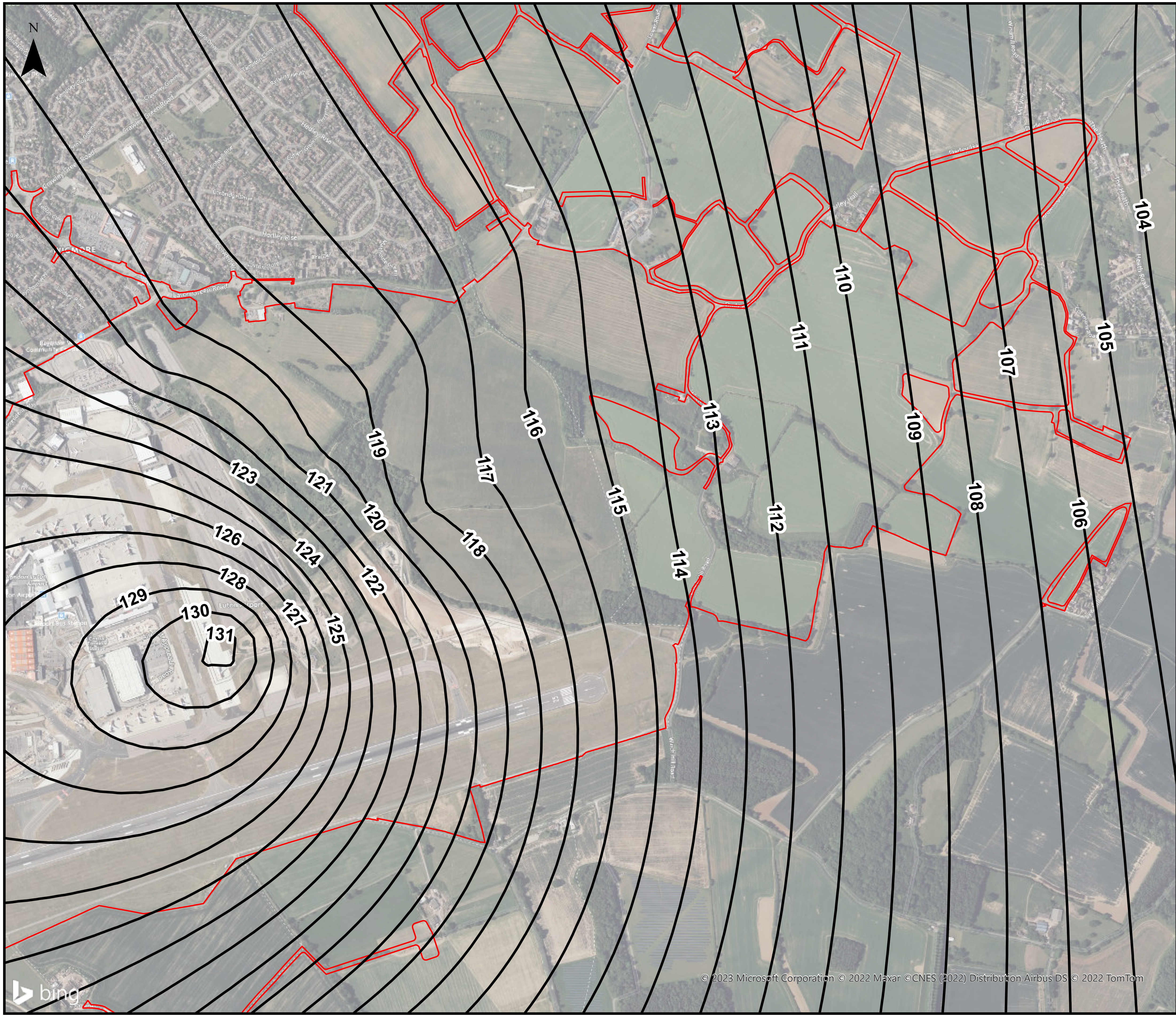
Project - Phase - Originator - Asset/Zone - Sub Asset - Type - Discp. - Number



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Legend

- Order Limits
- March 2003 1 in 10-year predicted groundwater levels (1m Contour)

Revision History	Drawn	Checked / Approved	Date	Rev.
First Issue	AB	TG / CS	17/02/23	P01

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Figure 16
1 in 10-year predicted maximum groundwater event contours (March 2003)

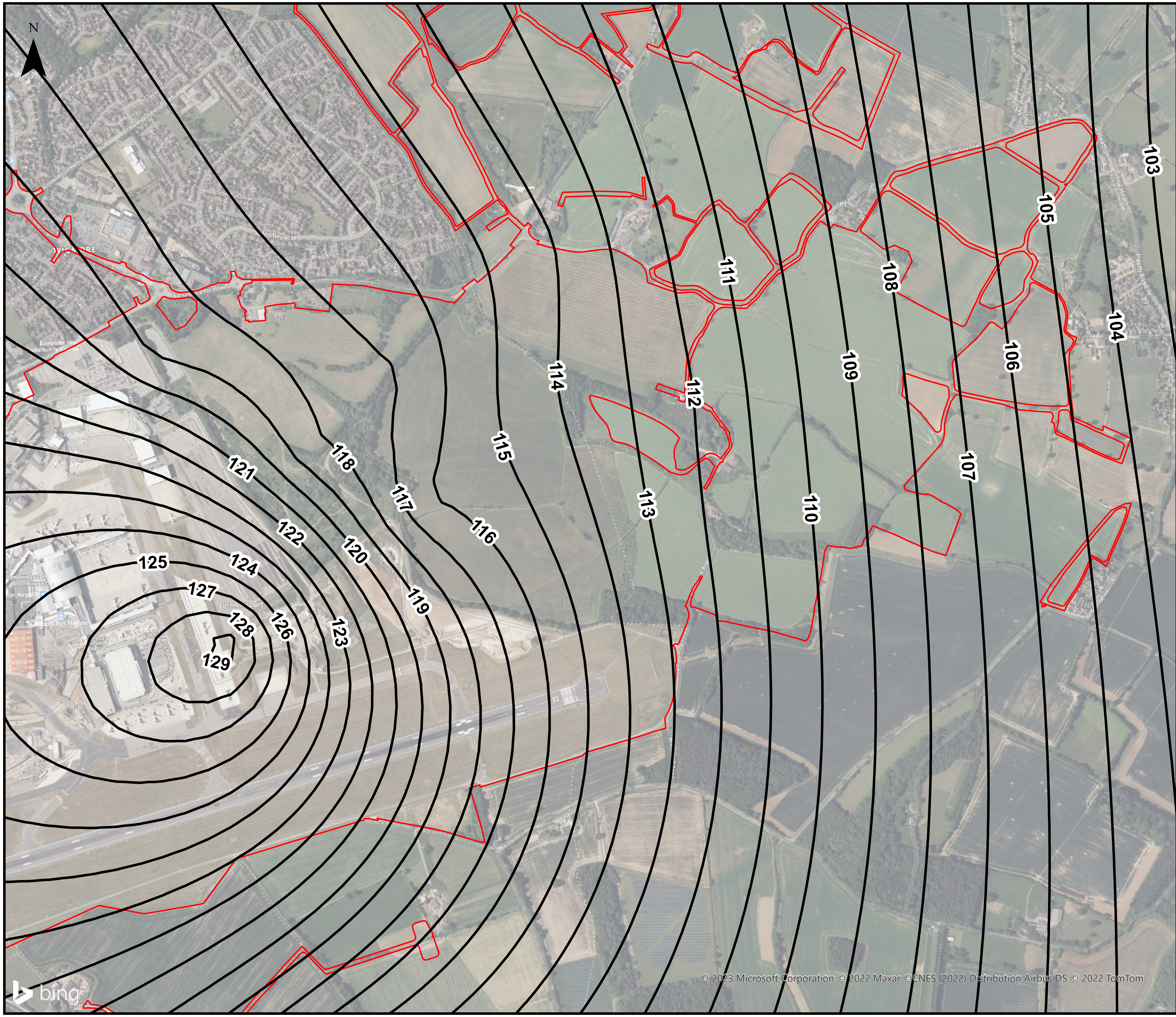
Purpose of issue					Suitability
SUITABLE FOR INFORMATION					S2

Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:10,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0239	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number



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Legend

- Order Limits
- March 2013 1 in 5-year predicted groundwater levels (1m Contour)

First Issue	AB	TG / CS	17/02/23	P01
Revision History	Drawn	Checked / Approved	Date	Rev.

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Figure 17
1 in 5-year predicted maximum groundwater event contours (March 2013)

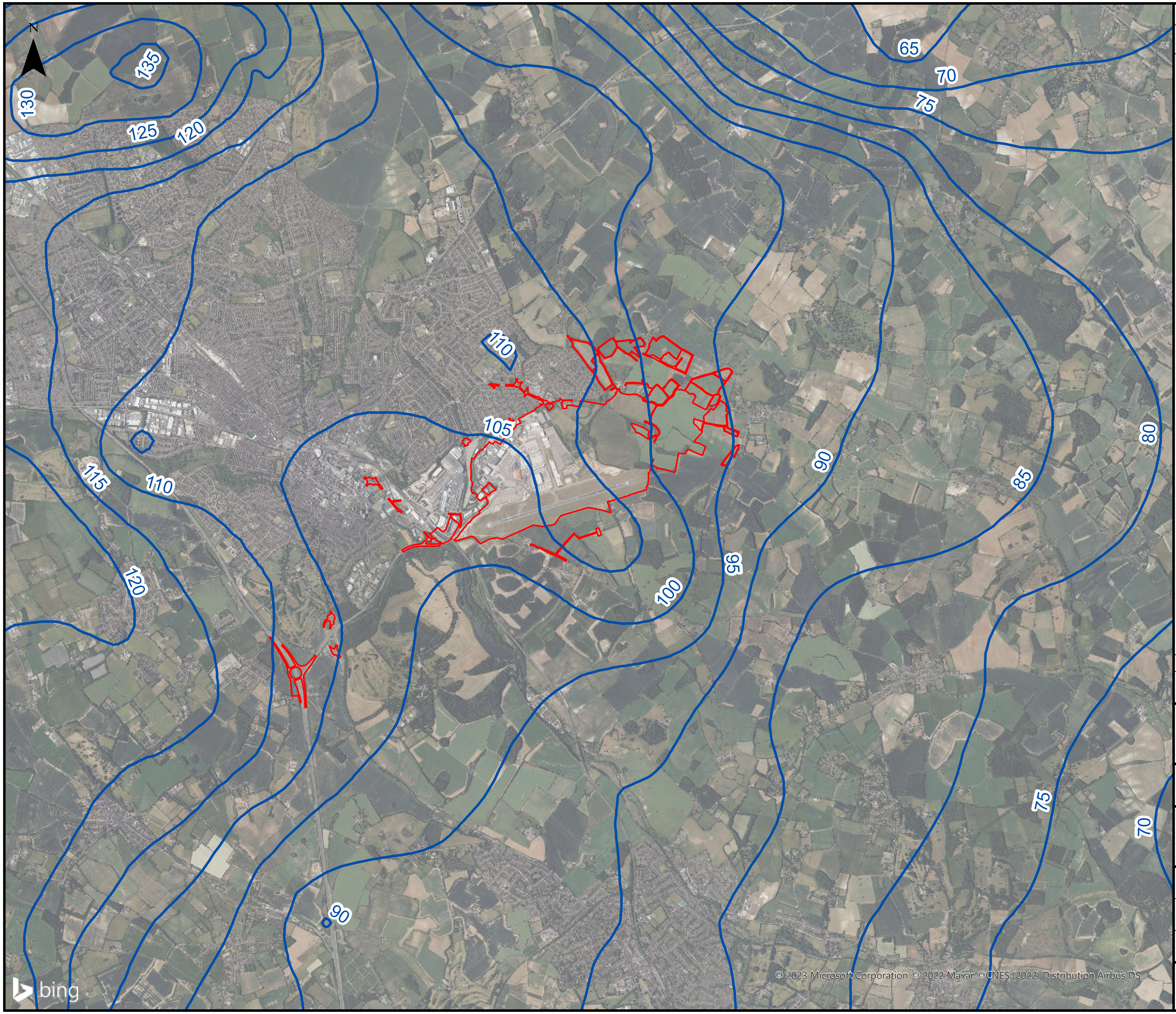
Purpose of issue					Suitability
SUITABLE FOR INFORMATION					S2
Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:10,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0240	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

APPENDIX B – HERTFORDSHIRE GROUNDWATER MODEL OUTPUTS



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Legend

Order Limits

March 1990 Ground Water (GW) Levels in mAOD (Chalk)

Revision History	Drawn	Checked / Approved	Date	Rev.
First Issue	AB	TG / CS	17/02/23	P01

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Hertfordshire Model - March 1990 Contours

Purpose of issue				Suitability	
SUITABLE FOR INFORMATION				S2	
Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:50,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
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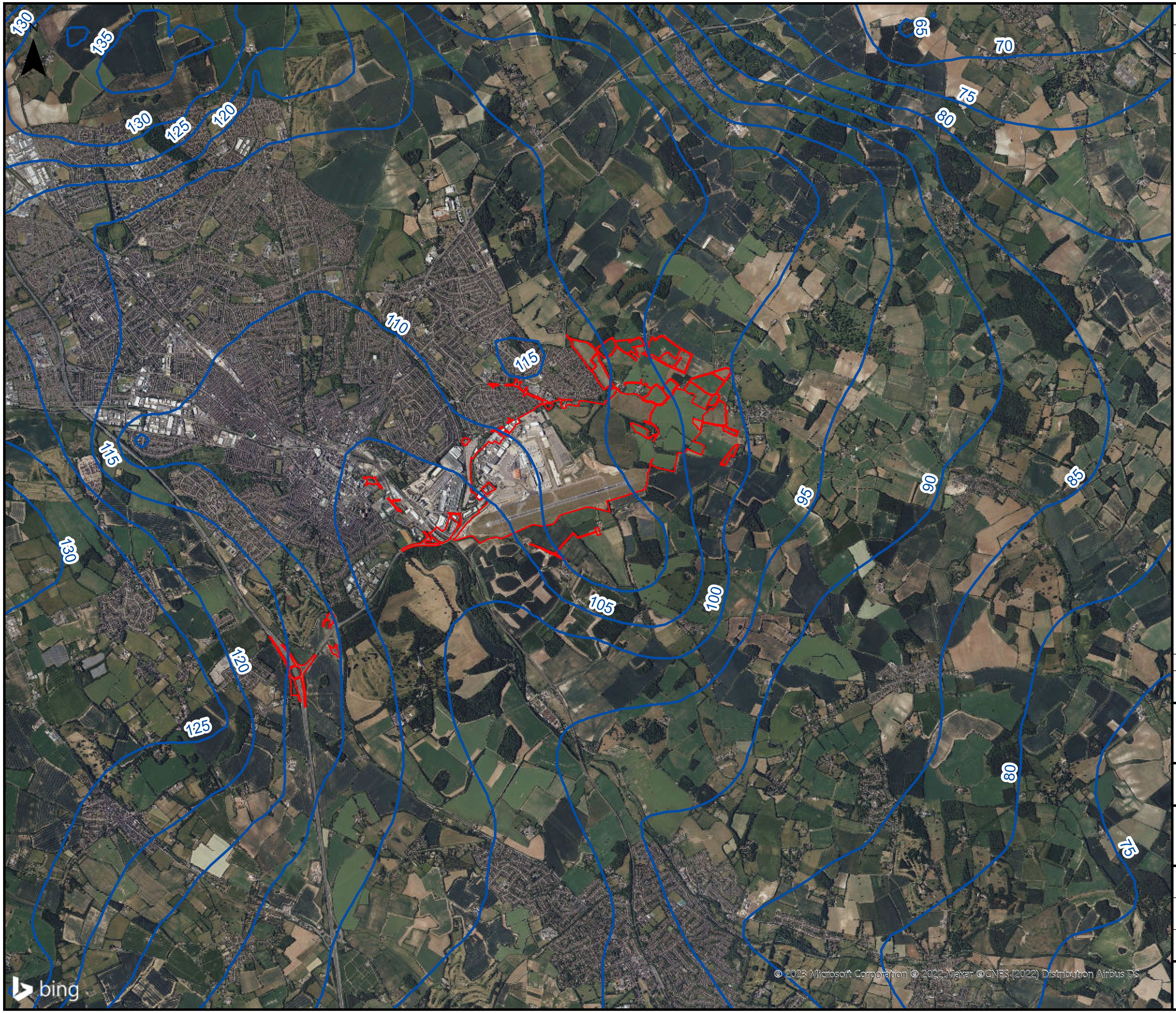

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Legend

Order Limits

October 1993 Ground Water (GW) Levels in mAOD (Chalk)

First Issue	AB	TG / CS	17/02/23	P01
Revision History	Drawn	Checked / Approved	Date	Rev.

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Hertfordshire Model - October 1993 Contours

Purpose of issue					Suitability
SUITABLE FOR INFORMATION					S2
Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:50,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
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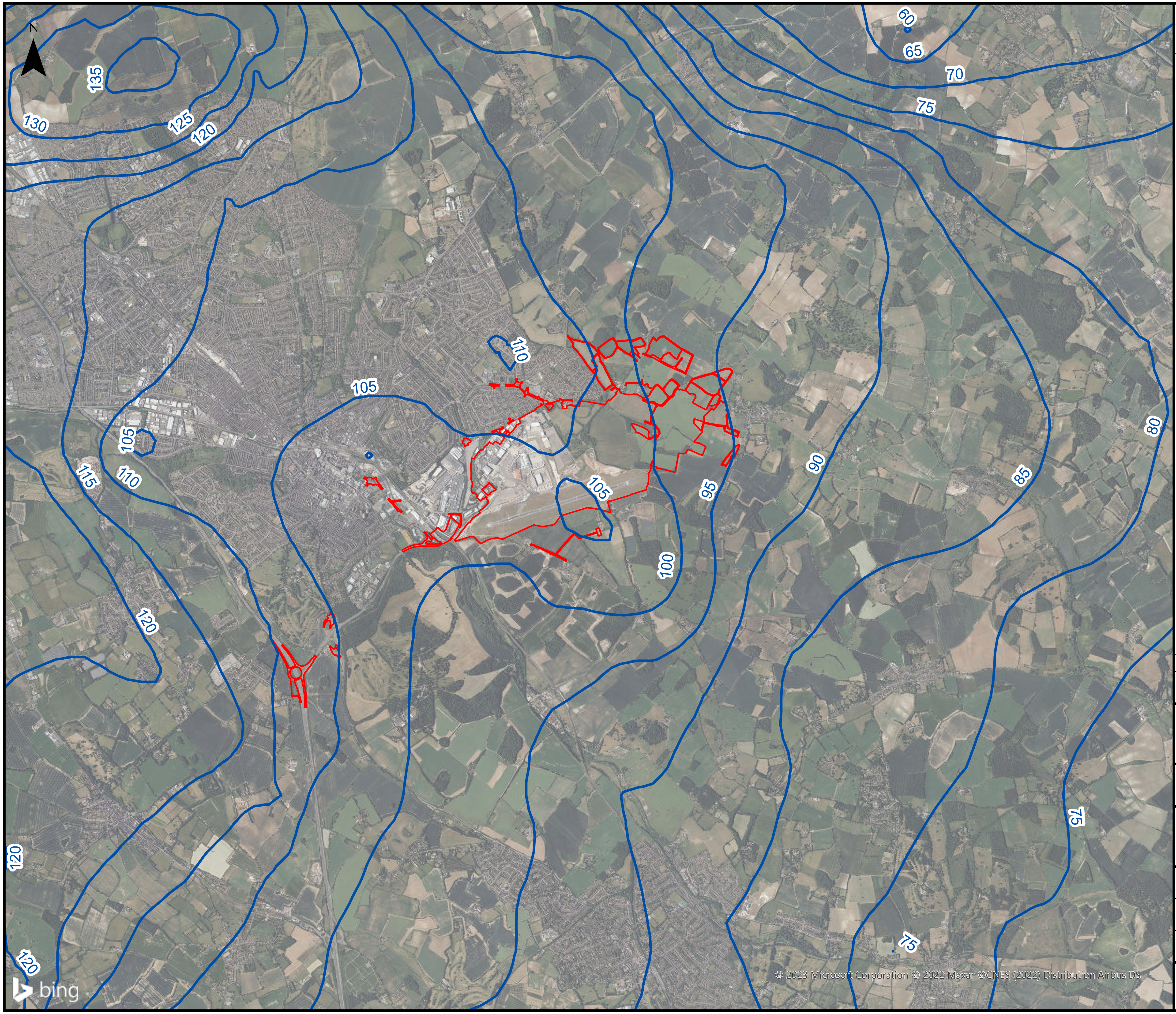

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Legend

- Order Limits
- December 1997 Ground Water (GW) Levels in mAOD (Chalk)

mAOD: metres Above Ordnance Datum

Revision History	Drawn	Checked / Approved	Date	Rev.
First Issue	AB	TG / CS	17/02/23	P01

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Hertfordshire Model - December 1997 Contours

Purpose of issue					Suitability
SUITABLE FOR INFORMATION					S2

Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:50,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0381	P01

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Legend

- Order Limits
- November 1999 Ground Water (GW) Levels in mAOD (Chalk)

Revision History	Drawn	Checked / Approved	Date	Rev.
First Issue	AB	TG / CS	17/02/23	P01

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Hertfordshire Model - November 1999 Contours

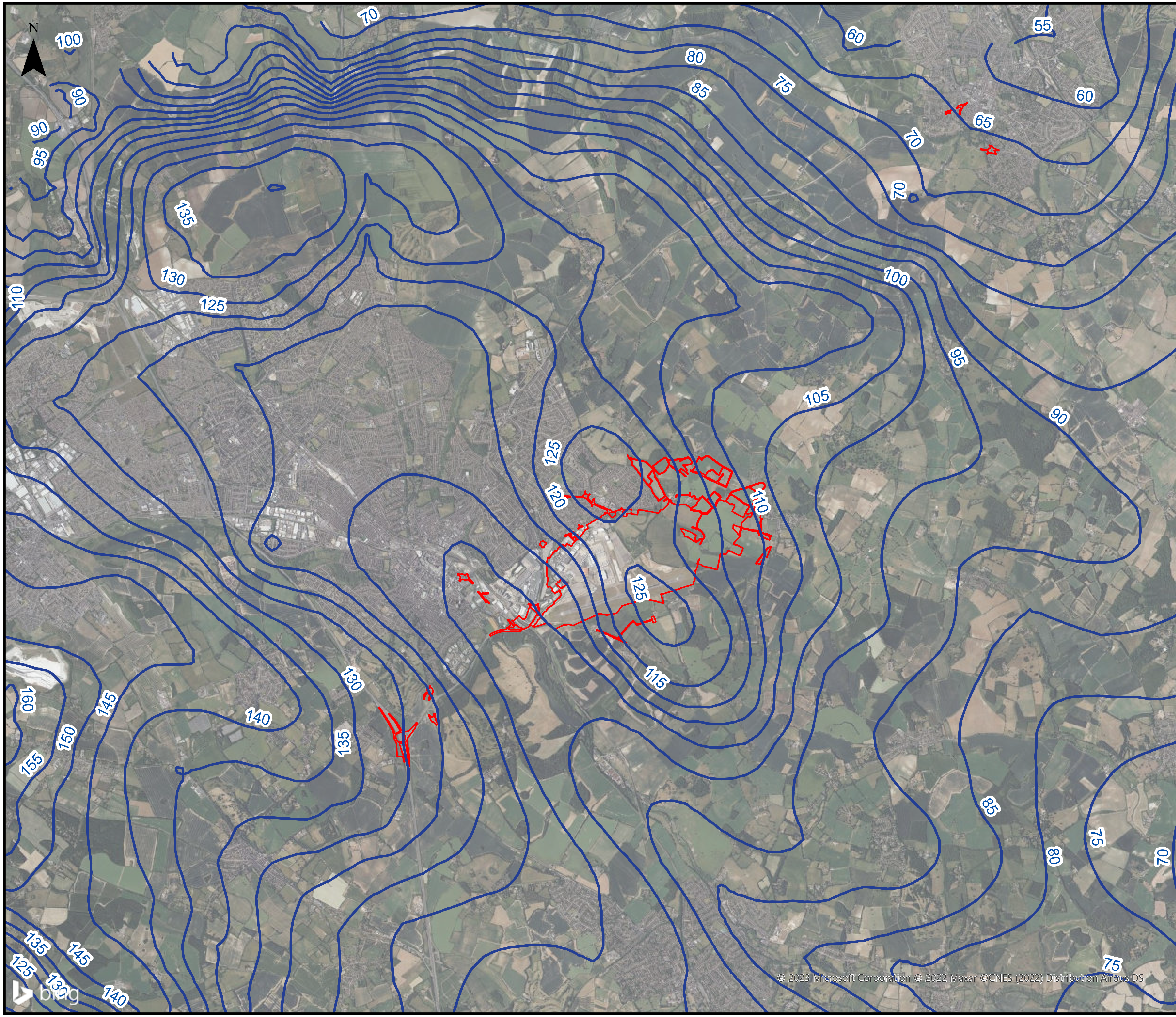
Purpose of issue	Suitability
SUITABLE FOR INFORMATION	S2

Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:50,000	A3

DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0383	P01

Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number



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Legend

Order Limits

April 2001 Ground Water (GW) Levels in mAOD (Chalk)

mAOD: metres Above Ordnance Datum

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Revision History	Drawn	Checked Approved	Date	Rev.

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Hertfordshire Model - April 2001 Contours

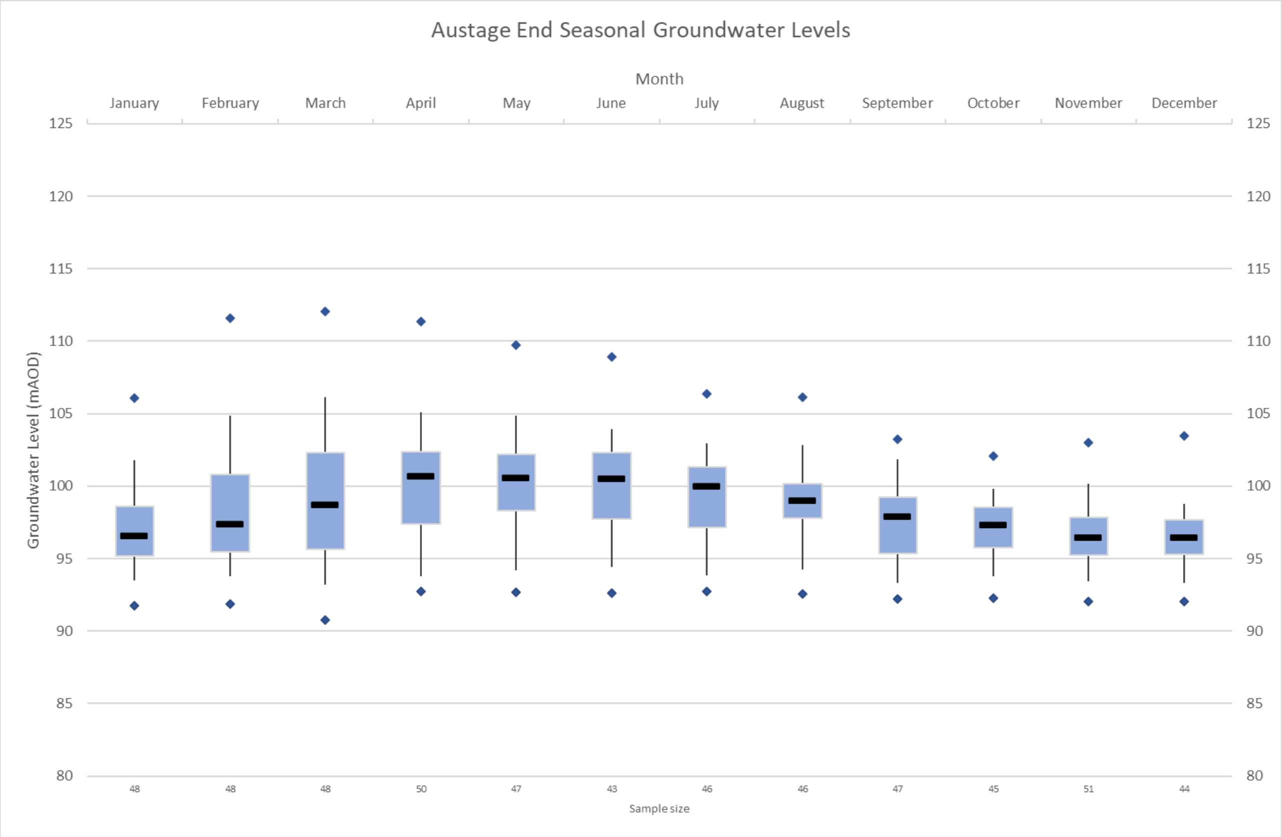
Purpose of issue				Suitability	
SUITABLE FOR INFORMATION				S2	
Drawn	Checked	Approved	Date	Scale	Size
AB	TG	CS	17/02/23	1:60,000	A3

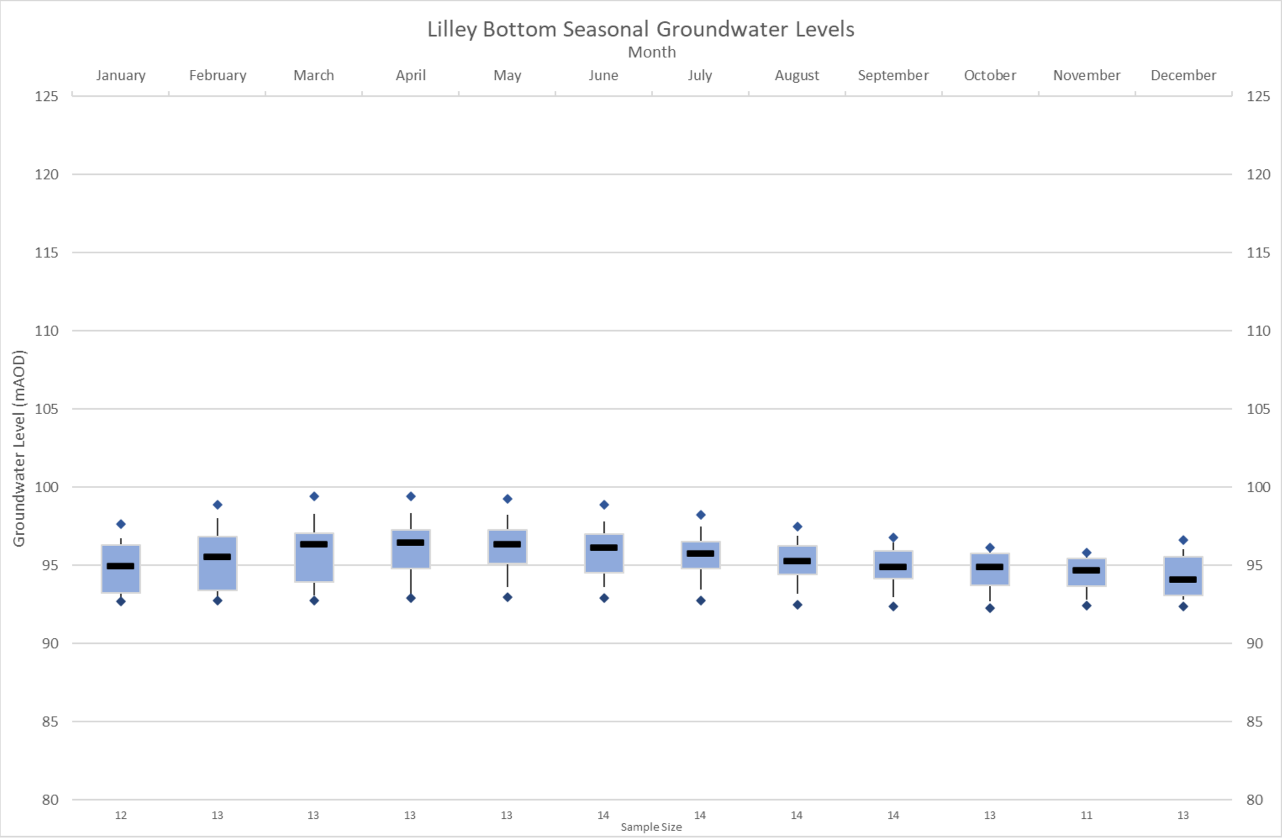
DCO Application Ref.	APFP Regulation	DCO Document Ref.
TR020001	APFP 5(2)(a)	TR020001/APP/5.02

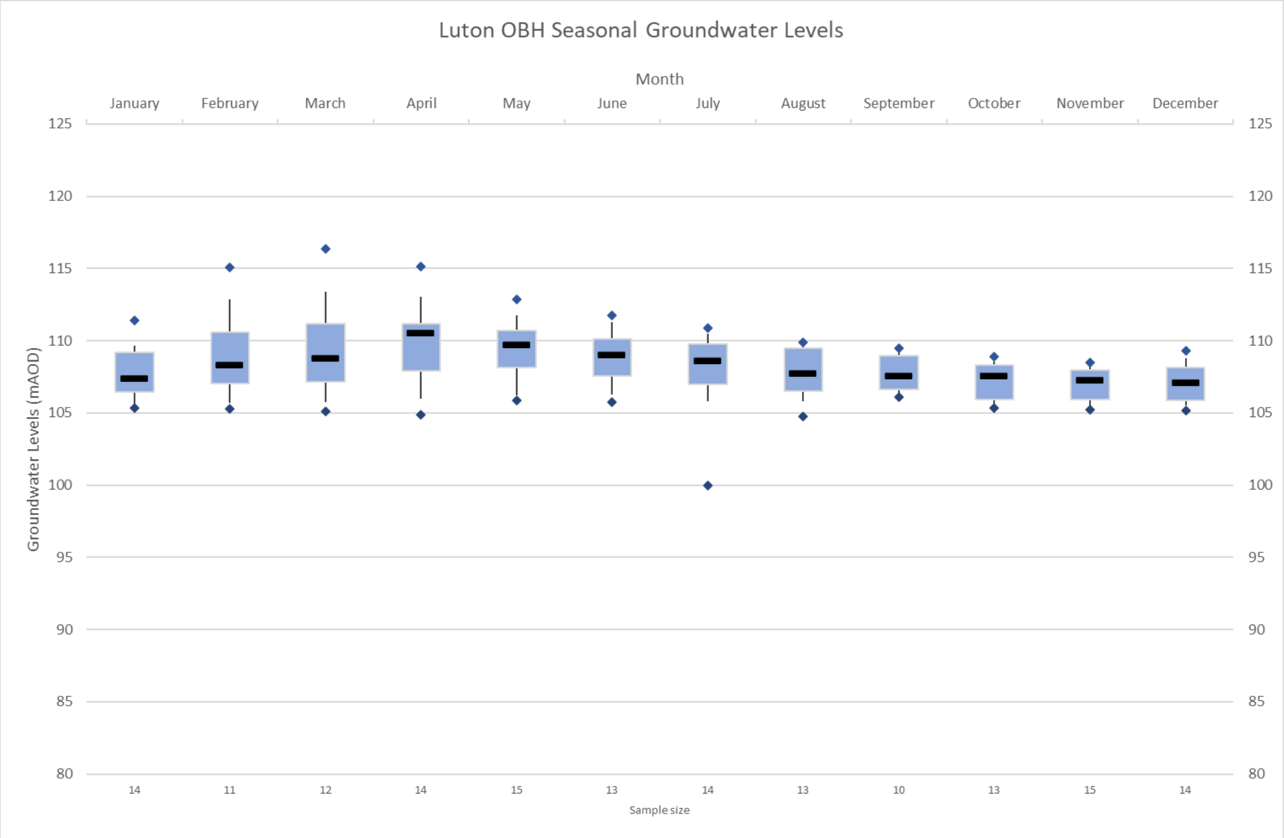
Drawing Number	Revision
LLADCO-3C-ARP-00-00-DR-YE-0380	P01

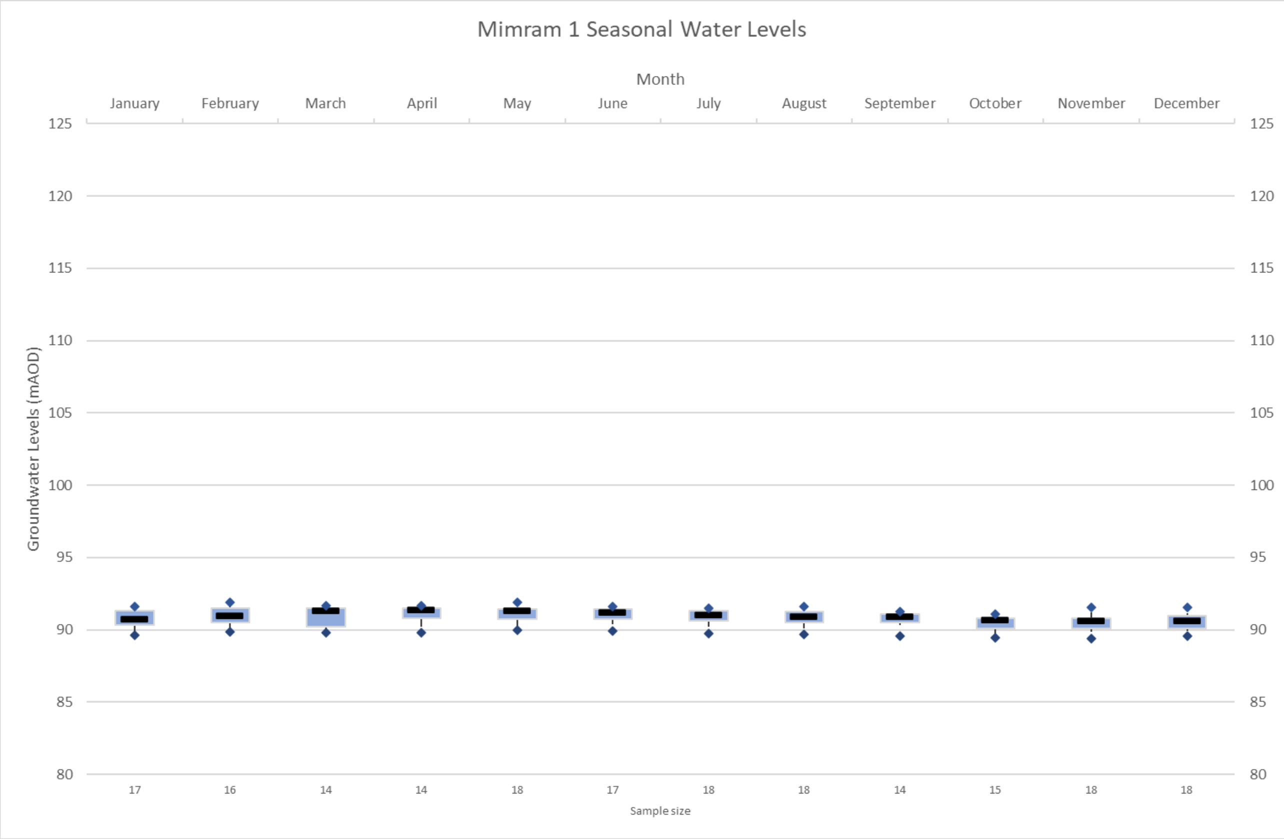
Project - Phase - Originator - Asset/Zone - Sub Asset - Type- Discp. - Number

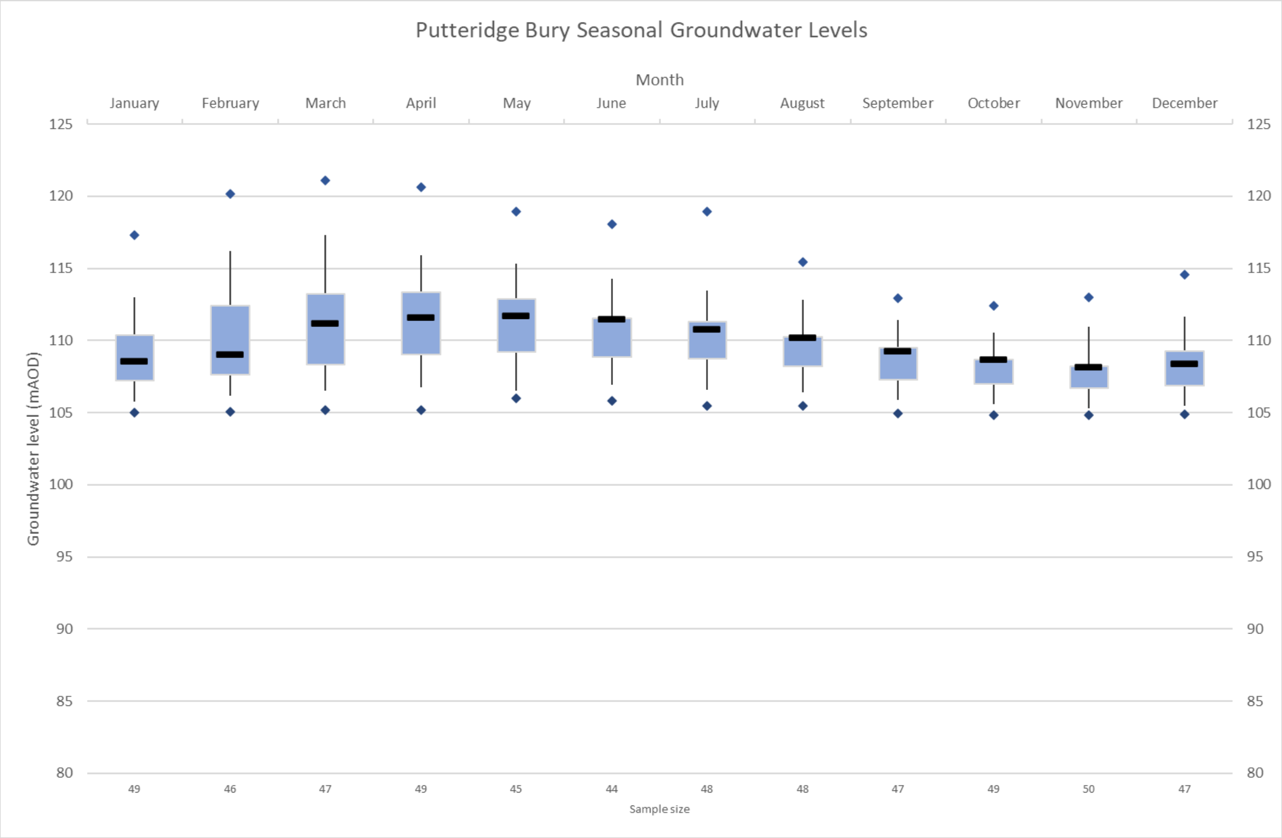
APPENDIX C – EA OBSERVATION BOREHOLES – BOX AND WHISKER PLOTS

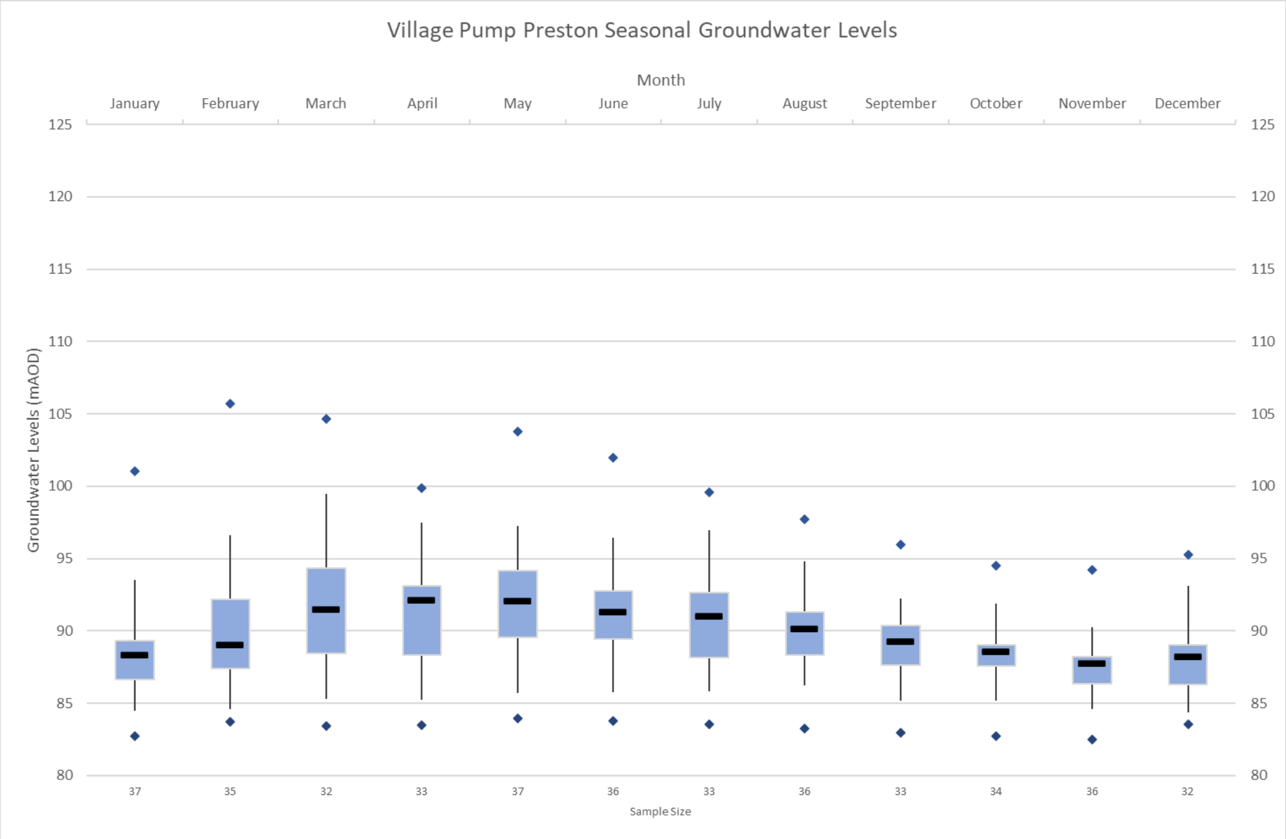

Austage End Seasonal Groundwater Levels
Month
January
February
March
April
May
June
July
August
September
October
November
December
Groundwater Level (mAOD)
125
120
115
110
105
100
95
90
85
80
48
48
48
50
47
43
46
46
47
45
51
44
Sample size


Lilley Bottom Seasonal Groundwater Levels
Month
January
February
March
April
May
June
July
August
September
October
November
December
Groundwater Level (mAOD)
125
120
115
110
105
100
95
90
85
80
12
13
13
13
13
14
14
14
14
13
11
13
Sample Size


Luton OBH Seasonal Groundwater Levels
Month
January February March April May June July August September October November December
Groundwater Levels (mAOD)
125 120 115 110 105 100 95 90 85 80
14 11 12 14 15 13 14 13 10 13 15 14
Sample size


Mimram 1 Seasonal Water Levels
Month
January February March April May June July August September October November December
Groundwater Levels (mAOD)
125 120 115 110 105 100 95 90 85 80
17 16 14 14 18 17 18 18 14 15 18 18
Sample size


Putteridge Bury Seasonal Groundwater Levels
Month
January
February
March
April
May
June
July
August
September
October
November
December
Groundwater level (mAOD)
125
120
115
110
105
100
95
90
85
80
49
46
47
49
45
44
48
48
47
49
50
47
Sample size


Village Pump Preston Seasonal Groundwater Levels
Month
January
February
March
April
May
June
July
August
September
October
November
December
Groundwater Levels (mAOD)
125
120
115
110
105
100
95
90
85
80
37
35
32
33
37
36
33
36
33
34
36
32
Sample Size

APPENDIX D – MAXIMUM GROUNDWATER LEVEL ASSESSMENT

File Note

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Project title	Luton Airport Expansion	Job number	245580-02
cc		File reference	Appendix D_GW contour methodology
Prepared by	Emma Watts (Leeds)	Date	28 June 2019
Subject	Groundwater Level Contours - Method		

1 Introduction

For the expansion of Luton Airport, several earthworks are needed. There is the potential for these earthworks activities to impact on the groundwater in the area. Therefore, it is important to understand the conceptual hydrogeological model within the development area, especially the maximum groundwater levels that are expected to be encountered beneath the site.

Previously the Environment Agency (EA) Vale of St Albans Model (VSAM) has been used in this project to determine the maximum groundwater levels underneath the site, as it was the best source of spatial groundwater level information pertaining to the April 2001 high groundwater levels event. However due to the large spatial scope of the VSAM, the levels in this numerical model were calibrated to groundwater levels within observation boreholes several kilometres from the site, resulting in uncertainty as to whether this model could accurately represent the groundwater levels beneath the Luton Airport site.

In order to produce site-specific maximum groundwater levels, groundwater monitoring at the site has been carried out since November 2016. However, as the on-site monitoring did not capture a high groundwater event, site-specific data has had to be compared to the levels from across the catchment from the April 2001 high groundwater event. This technical note details the methodology undertaken to convert these measured groundwater levels to maximum groundwater levels underlying Luton Airport.

2 On-site Monitoring

Groundwater levels from on-site monitoring boreholes at the time of writing this note were available from November 2016 to January 2019. During this period the highest recorded groundwater levels at the site were in June 2018.

However, as demonstrated in Figure 1, which shows the groundwater elevations in EA boreholes close to the Luton Airport site, June 2018 (and the full period of on-site monitoring) represents a period where groundwater elevations are relatively low in their historical record.

Figure 1: Groundwater elevations in EA boreholes close to the Luton Airport site

Therefore, in order to use the on-site monitoring data to calculate the absolute maximum groundwater conditions, a groundwater level uplift factor needs to be used to the observed June 2018 levels.

3 Maximum groundwater levels – April 2001 vs June 2018

According to the EA monitoring record, as well as the VSAM, the maximum groundwater conditions in recorded history were in April 2001.

To observe the level differences across the catchment between June 2018 and April 2001, four EA boreholes in close proximity to the site were utilised to compare the real-life level differences between the two months.

Table 1: EA borehole level comparison

EA Borehole Name	Distance and direction to the Luton Airport site	Groundwater level in June 2018 (mAOD)	Groundwater level in April 2001 (mAOD)	Difference in groundwater level (m)
Putteridge Bury	2.5km north-northwest of the Luton site Located along the axis of the groundwater divide*	110	121	11
Luton OBH	1.5km south of the Luton site	109	120#	7.5

	Located west of the groundwater divide towards the River Lee			
Lilley Bottom	1.3km north-east of the Luton site Located close to the dry valley to the east of the site	95	99.5**	4.5
Mimram 1	2.9km east of the Luton site Located within the Mimram river	90.5	92	1.5

*The location of the groundwater divide varies depending on the data source and its resolution, for this assessment the location of the groundwater divide is as suggested in the Vale of St Albans Model, the most recent data available.

#EA monitoring records for Luton OBH do not go back to April 2001, though typically where records are available Luton OBH groundwater levels are approximately 1m below Putteridge Bury levels during peak groundwater events.

**EA monitoring records for Lilley Bottom OBH also do not go back to April 2001. As the groundwater levels in Lilley Bottom do not display "peaky" responses, the maximum groundwater level observed in its record (from June 2005) is considered an appropriate value for use in this assessment.

The results in Table 2 suggest that close to the centre of the groundwater divide, in the Putteridge Bury borehole, the groundwater levels in April 2001 were up to 11m higher than in June 2018.

In comparison, the levels in the river Mimram 1 (within the valley bottom) are only 1.5m higher in April 2001. This confirms the previously presented conceptual understanding of the groundwater levels in the area, in that the levels in the interfluves (near the groundwater divide) vary more than those close to the valleys.

4 Head Uplift Factor

The difference in levels between the two months depend is on the location of the borehole within the catchment. Therefore, a single uplift value for all boreholes is considered inappropriate.

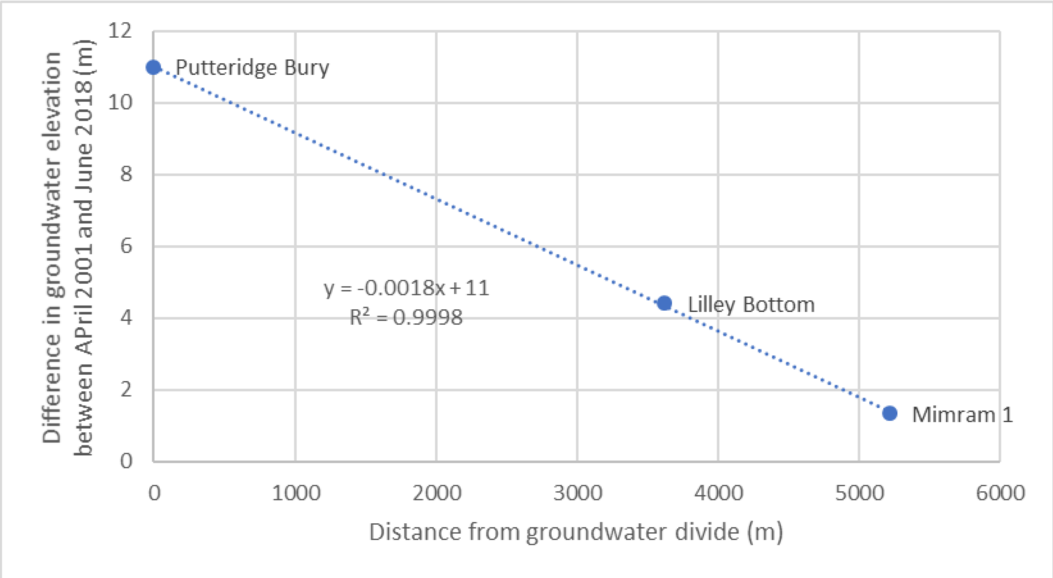
Therefore, a comparison between the distance from the groundwater divide and the corresponding difference in groundwater level between June 2018 and April 2001 was undertaken. Three boreholes, detailed below, were used to observe the relationship and determine how to apportion for the head uplift:

- Putteridge Bury was chosen as a representation of groundwater levels in the centre of the groundwater divide;
- Lilley Bottom as a mid-point between the groundwater levels in the divide and in the valley bottoms; and,

- Mimram 1 as a representation of groundwater levels in the valley bottoms.

As shown in Figure 2, the groundwater elevation difference between the June 2018 and April 2001 events show a linear decrease with distance from the groundwater divide (going east).

Figure 2: Relationship between distance from groundwater divide and groundwater elevation



Based on this, depending on the distance of the on-site monitoring boreholes from the groundwater divide (from VSAM), the head uplift to represent maximum groundwater conditions can be established using the equation:

$$Head\ uplift = (-0.0018 * distance\ from\ centre\ of\ groundwater\ divide) + 11$$

This gives the following results in Table 2:

Table 2: Groundwater level results from head uplift calculation

On-site Borehole Name	Distance from centre of Groundwater Divide (m)	Head Uplift (m)	Groundwater level in June 2018 (mAOD)	Calculated groundwater level in April 2001 (mAOD)
LF-BH02	628	9.87	117.26	127.13
LF-BH03	842	9.48	114.33	123.81
LF-BH05	0	11.00	124.46	135.46
LF-BH08	592	9.93	113.60	123.53
LF-BH10	545	10.02	113.37	123.39
LF-BH13	684	9.77	112.45	122.21
CP-BH12	691	9.76	111.36	121.12
CP-BH24	860	9.45	110.94	120.39
CP-BH27	1402	8.48	110.25	118.73

CP-BH29	1112	9.00	109.96	118.96
CP-BH32	862	9.45	111.38	120.83
CP-BH50	1292	8.67	108.33	117.00
CP-BH51	1460	8.37	108.12	116.49
CP-BH55	876	9.42	112.65	122.07

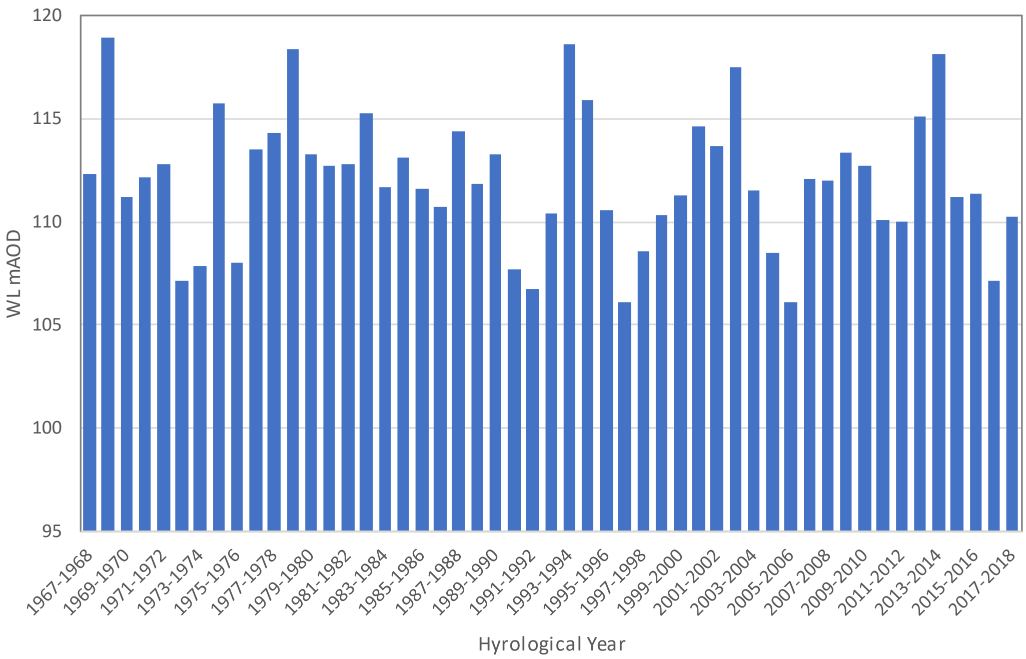
5 Other Maximum Events

5.1 Statistical interpretation of groundwater levels

The maximum groundwater levels taken from the Putteridge Bury borehole TL12SW101 (NGR 511920, 224780) were used to understand the return periods associated with the maximum recorded groundwater elevations in the region.

There are 51 years of gauged record associated with this borehole (Figure 3).

Figure 3: Maximum groundwater elevations at Putteridge Bury Borehole in each Water Year (Oct – Sep)



5.2 Methodology

Two methods were considered in the estimation of the return periods associated with the maximum groundwater elevations.

- The Gringorten plotting position; and
- WINFAP.

5.2.1 Gringorton

Making use of the Gringorten equation, the plotting position was calculated for the maximum groundwater elevation for each water year; the groundwater elevations for the specific return periods in Table 3 were then interpolated from these data:

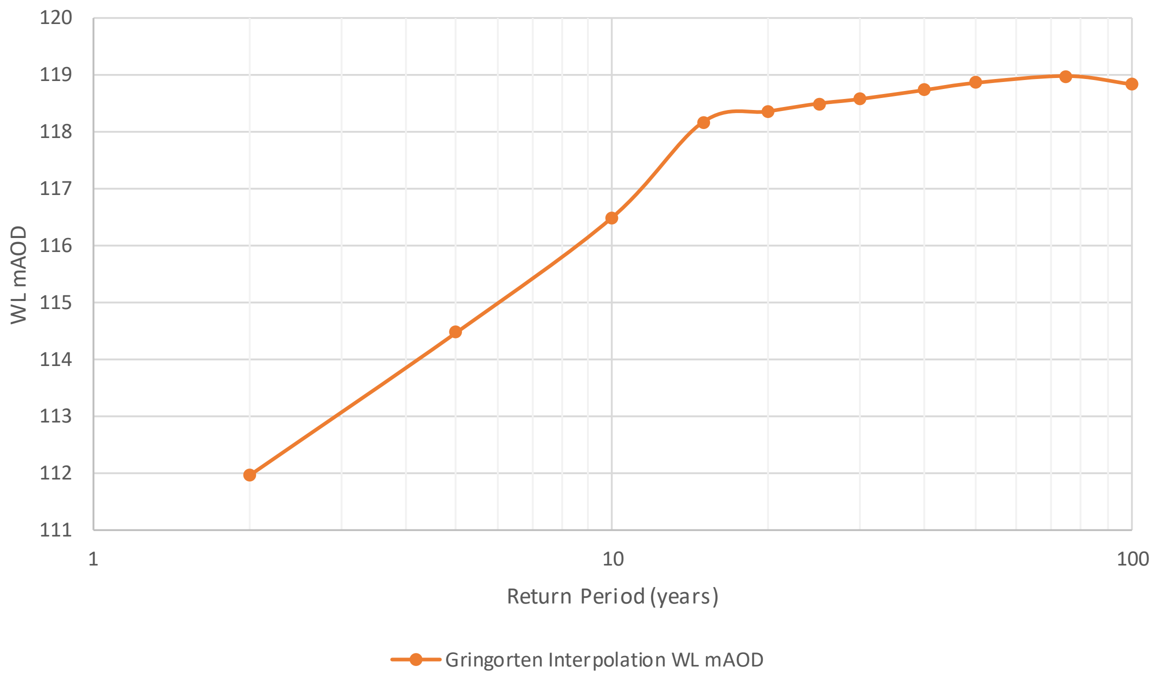
$$T = \frac{n + 0.12}{m - 0.44}$$

Equation 1: Equation used to estimate the recurrence interval (T) according to the Gringorten plotting position, where T = recurrence interval in years, n = total number of years in record used and m = magnitude or rank

Table 3: Gringorton derived groundwater elevations for each return period, based on a 51-year record

Return Period	Gringorten Interpolation WL mAOD
2	111.96
5	114.47
10	116.49
15	118.18
20	118.36
25	118.50
30	118.58
40	118.74
50	118.86
75	118.98
100	118.84

Figure 4: Plotted Gringorton derived groundwater elevations for each return period, based on a 51-year record



5.2.2 WINFAP

The second approach follows on from the initial Gringorten analysis. WINFAP 4 was developed by WHS to implement a range of the industry-standard FEH statistical analysis tools. It uses the latest FEH methods to provide estimates of peak flows and flood frequency curves for gauged and ungauged catchments.

This software was specifically designed for use in fluvial flood analysis, but the methodology used behind the analysis has been adopted here to make use of the groundwater elevations instead of gauged Annual Maximum (AMAX) flood flows.

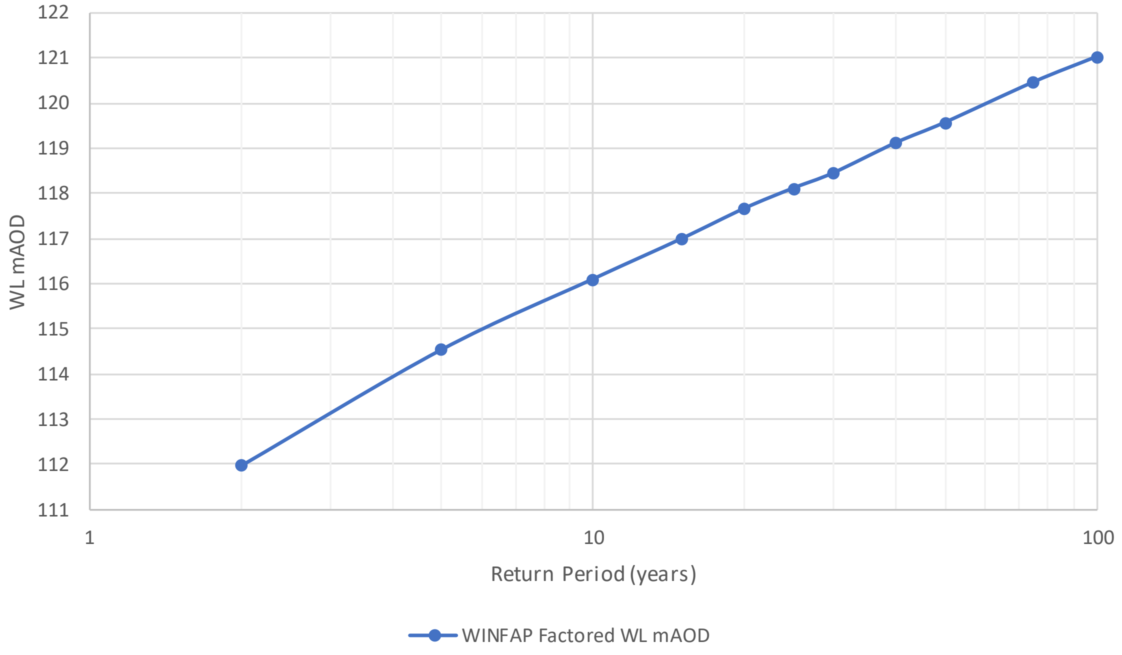
The Single Site Analysis was adopted for this assessment. This analysis is based on an observed flood series of Annual Maxima (AM) or Peaks Over Threshold (POT) flow data at the target catchment, in this case the AMAX flows were replaced with the maximum groundwater elevations recorded within each Water Year (Oct – Sep). The FEH recommends the Generalised Logistic distribution and the L-median fitting method for UK flood data, this distribution was also used for this assessment.

The growth factors were then applied to the QMED flood event to estimate flood peak flows for each return period. In this instance, the QMED has been replaced by the median groundwater elevation (Table 4).

Table 4: WINFAP growth curve factors and calculated groundwater elevations

Return Period	**Generalised Logistic (GL) - LMED Growth Curve**	**WINFAP Factored WL mAOD**
2	1.000	111.96
5	1.023	114.54
10	1.037	116.10
15	1.045	117.00
20	1.051	117.67
25	1.055	118.12
30	1.058	118.45
40	1.064	119.13
50	1.068	119.57
75	1.076	120.47
100	1.081	121.03

Figure 5: Plotted WINFAP peak flows for each return period

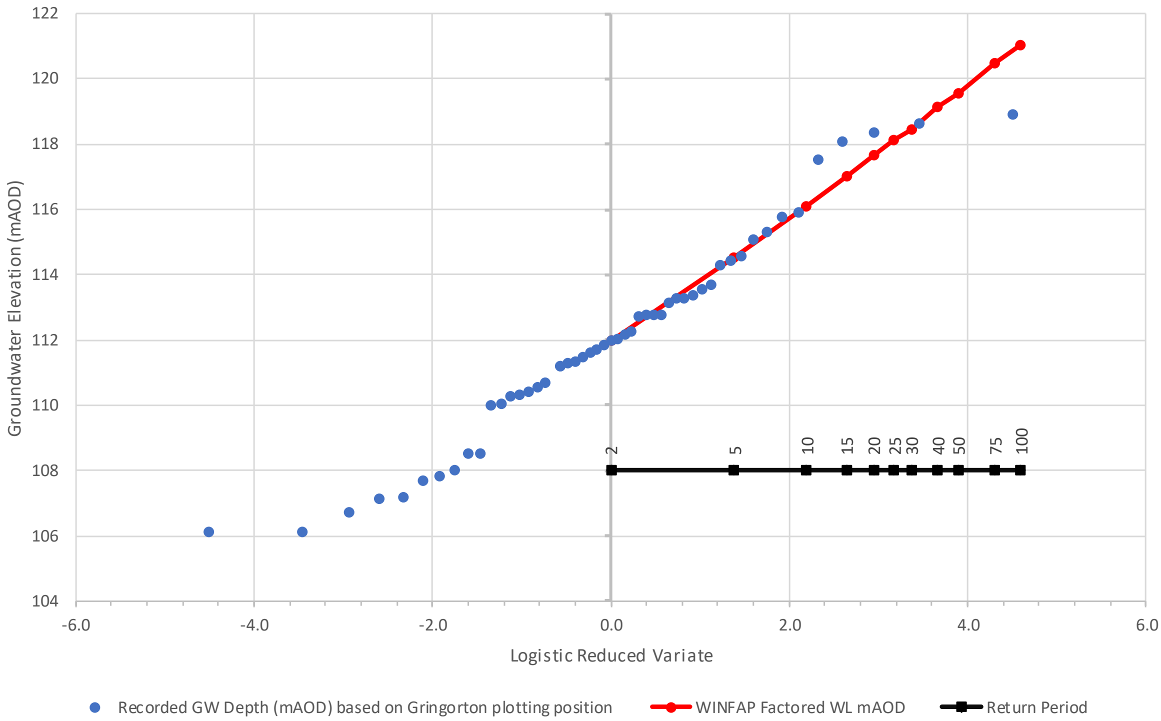


5.3 Summary

A comparison of the two methods has been presented in

Figure 6. The WINFAP data is much more linear when compared with the Gringorten levels.

Figure 6: Gringorten plotted gauged groundwater elevations and statistically derived WINFAP groundwater elevations



5.4 1 in 5, 1 in 10 and 1 in 25-year groundwater contours

In addition, 1 in 25-year, 1 in 10-year and 1 in 5-year maximum events were simulated. The same method was undertaken for each, however new head uplifts had to be calculated due to the hydraulic gradient changes. For each the nearest actual groundwater event in the historical record was used to the WINFAP predicted values, as shown in Table 5.

Table 5: Comparison between WINFAP predicted return period levels in Putteridge Bury and actual groundwater event used for contouring

Return Period year	WINFAP Factored Predicted WL in Putteridge Bury (mAOD)	Actual groundwater event in Putteridge Bury (mAOD)	Month and year actual groundwater event happened
2	111.96		
5	114.54	115.10	Mar-13
10	116.10	117.70	Mar-03
15	117.00		
20	117.67		
25	118.12	118.10	Mar-14
30	118.45		
40	119.13		
50	119.57		
75	120.47		
100	121.03	120.64	Apr-01

Figure 7: March 2014 (1 in 25-year event)

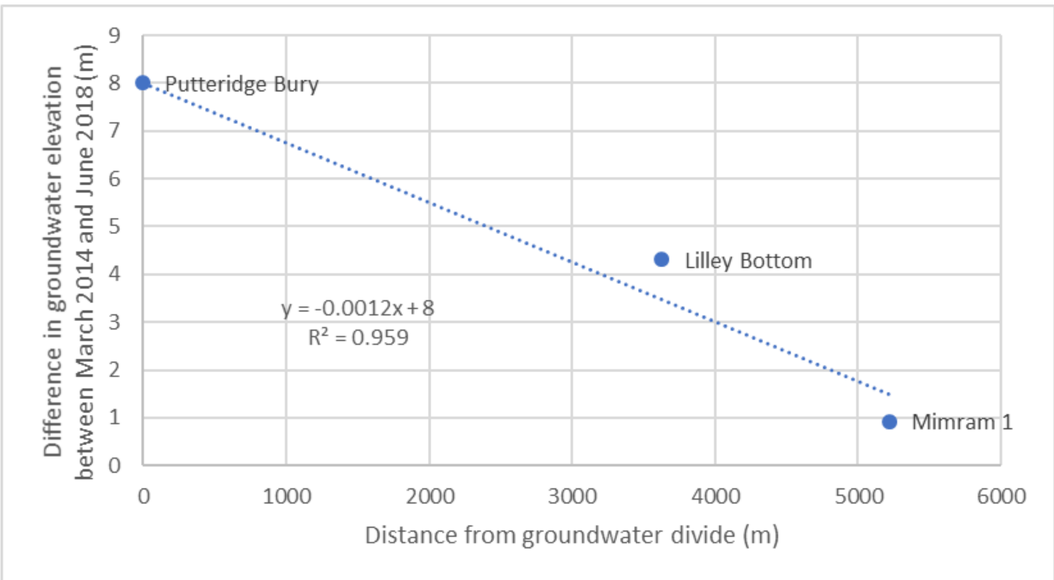


Figure 8: March 2003 (1 in 10-year event)

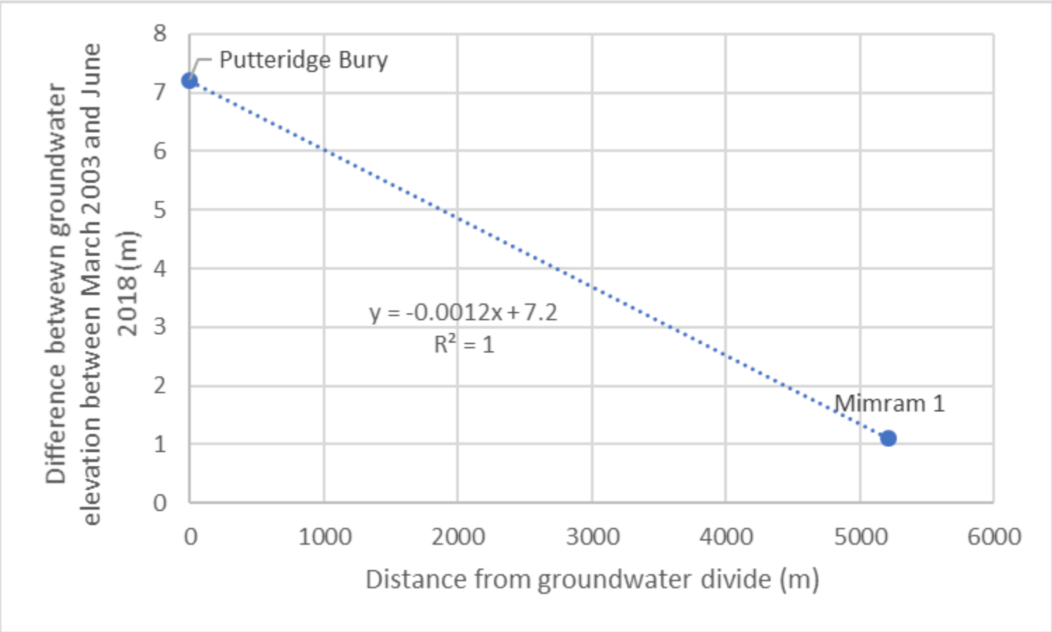


Figure 9: March 2013 (1 in 5-year event)

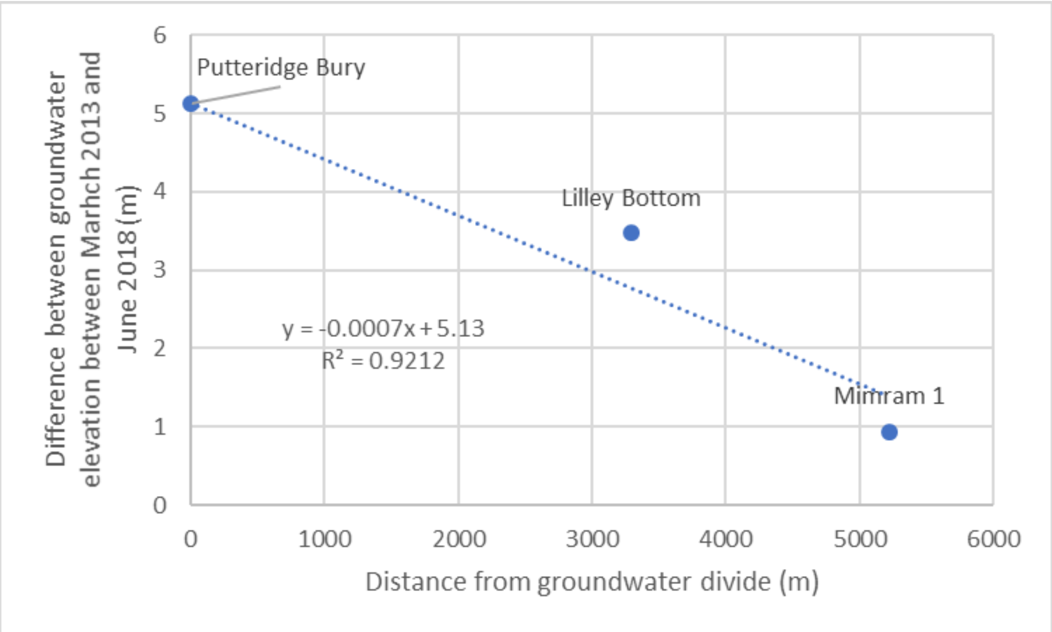


Table 6: Adjusted groundwater levels for the on-site boreholes during maximum events

On-site Borehole Name	1 in 25-year maximum event groundwater level (mAOD)	1 in 10-year maximum event groundwater level (mAOD)	1 in 5-year maximum event groundwater level (mAOD)
LF-BH02	124.51	123.71	121.95
LF-BH03	121.32	120.52	118.87

LF-BH05	132.46	131.66	129.59
LF-BH08	120.89	120.09	118.32
LF-BH10	120.72	119.92	118.12
LF-BH13	119.62	118.82	117.10
CP-BH12	118.53	117.73	116.01
CP-BH24	117.91	117.11	115.47
CP-BH27	116.57	115.77	114.40
CP-BH29	116.63	115.83	114.31
CP-BH32	118.35	117.55	115.91
CP-BH50	114.78	113.98	112.56
CP-BH51	114.37	113.57	112.23
CP-BH55	119.60	118.80	117.17

DOCUMENT CHECKING (not mandatory for File Note)

	Prepared by	Checked by	Approved by
Name	Emma Watts (Leeds)	Gerd Cachandt (Leeds) / Yolande Macklin (Campus)	Yolande Macklin (Campus)
Signature	[redacted]	[redacted]	[redacted]

APPENDIX E – HANTUSH ASSESSMENTS

This spreadsheet will calculate the height of a groundwater mound beneath a stormwater infiltration basin. More information can be found in the U.S. Geological Survey Scientific Investigations Report 2010-5102 "Simulation of groundwater mounding beneath hypothetical stormwater infiltration basins".

The user must specify infiltration rate (R), specific yield (Sy), horizontal hydraulic conductivity (Kh), basin dimensions (x, y), duration of infiltration period (t), and the initial thickness of the saturated zone (hi(0), height of the water table if the bottom of the aquifer is the datum). For a square basin the half width equals the half length (x = y). For a rectangular basin, if the user wants the water-table changes perpendicular to the long side, specify x as the short dimension and y as the long dimension. Conversely, if the user wants the values perpendicular to the short side, specify y as the short dimension, x as the long dimension. All distances are from the center of the basin. Users can change the distances from the center of the basin at which water-table aquifer thickness are calculated.
Cells highlighted in yellow are values that can be changed by the user. Cells highlighted in red are output values based on user-specified inputs. The user MUST click the blue "Re-Calculate Now" button each time ANY of the user-specified inputs are changed otherwise necessary iterations to converge on the correct solution will not be done and values shown will be incorrect. Use consistent units for all input values (for example, feet and days)

use consistent units (e.g. feet & days **or** inches & hours)

Input Values

0.2700	*R*	**Recharge (infiltration) rate (feet/day)**
0.020	Sy	**Specific yield, Sy (dimensionless, between 0 and 1)**
6.70	K	**Horizontal hydraulic conductivity, Kh (feet/day)***
426.500	x	**1/2 length of basin (x direction, in feet)**
196.850	y	**1/2 width of basin (y direction, in feet)**
1.000	t	**duration of infiltration period (days)**
131.240	hi(0)	**initial thickness of saturated zone (feet)**

Conversion Table

inch/hour	**feet/day**
0.67	1.33
2.00	4.00

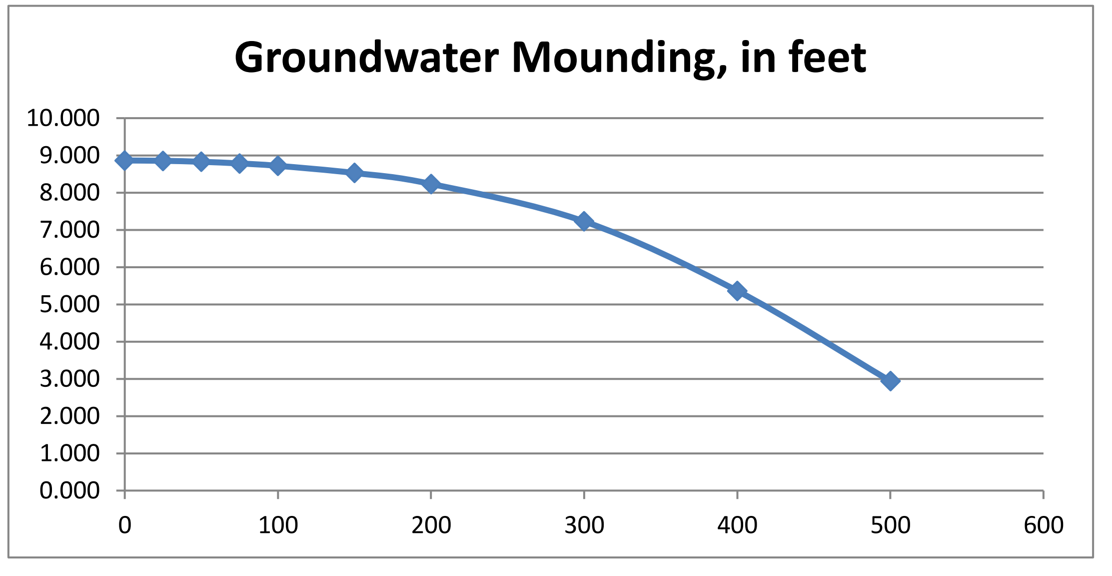
hours	**days**
36	1.50

In the report accompanying this spreadsheet (USGS SIR 2010-5102), vertical soil permeability (ft/d) is assumed to be one-tenth horizontal hydraulic conductivity (ft/d).

140.103	h(max)	**maximum thickness of saturated zone (beneath center of basin at end of infiltration period)**
8.863	Δh(max)	**maximum groundwater mounding (beneath center of basin at end of infiltration period)**

Ground-water Mounding, in feet	**Distance from center of basin in x direction, in feet**
8.863	**0**
8.855	**25**
8.828	**50**
8.784	**75**
8.720	**100**
8.528	**150**
8.234	**200**
7.227	**300**
5.364	**400**
2.940	**500**

Re-Calculate Now



Disclaimer

This spreadsheet solving the Hantush (1967) equation for ground-water mounding beneath an infiltration basin is made available to the general public as a convenience for those wishing to replicate values documented in the USGS Scientific Investigations Report 2010-5102 "Groundwater mounding beneath hypothetical stormwater infiltration basins" or to calculate values based on user-specified site conditions. Any changes made to the spreadsheet (other than values identified as user-specified) after transmission from the USGS could have unintended, undesirable consequences. These consequences could include, but may not be limited to: erroneous output, numerical instabilities, and violations of underlying assumptions that are inherent in results presented in the accompanying USGS published report. The USGS assumes no responsibility for the consequences of any changes made to the spreadsheet. If changes are made to the spreadsheet, the user is responsible for documenting the changes and justifying the results and conclusions.

This spreadsheet will calculate the height of a groundwater mound beneath a stormwater infiltration basin. More information can be found in the U.S. Geological Survey Scientific Investigations Report 2010-5102 "Simulation of groundwater mounding beneath hypothetical stormwater infiltration basins".

The user must specify infiltration rate (R), specific yield (Sy), horizontal hydraulic conductivity (Kh), basin dimensions (x, y), duration of infiltration period (t), and the initial thickness of the saturated zone (hi(0), height of the water table if the bottom of the aquifer is the datum). For a square basin the half width equals the half length (x = y). For a rectangular basin, if the user wants the water-table changes perpendicular to the long side, specify x as the short dimension and y as the long dimension. Conversely, if the user wants the values perpendicular to the short side, specify y as the short dimension, x as the long dimension. All distances are from the center of the basin. Users can change the distances from the center of the basin at which water-table aquifer thickness are calculated.
Cells highlighted in yellow are values that can be changed by the user. Cells highlighted in red are output values based on user-specified inputs. The user MUST click the blue "Re-Calculate Now" button each time ANY of the user-specified inputs are changed otherwise necessary iterations to converge on the correct solution will not be done and values shown will be incorrect. Use consistent units for all input values (for example, feet and days)

use consistent units (e.g. feet & days **or** inches & hours)

Input Values				
1.1900	*R*	**Recharge (infiltration) rate (feet/day)**		
0.020	Sy	**Specific yield, Sy (dimensionless, between 0 and 1)**		
6.70	K	**Horizontal hydraulic conductivity, Kh (feet/day)***		
196.860	x	**1/2 length of basin (x direction, in feet)**		
98.430	y	**1/2 width of basin (y direction, in feet)**		
1.000	t	**duration of infiltration period (days)**		
131.240	hi(0)	**initial thickness of saturated zone (feet)**		

Conversion Table

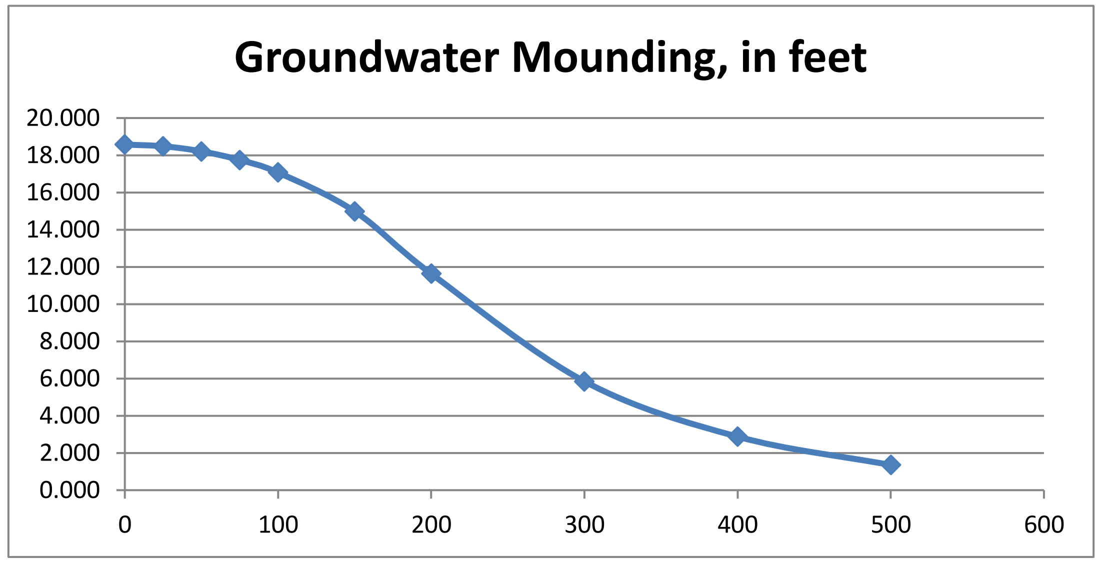
inch/hour	**feet/day**
0.67	1.33
2.00	4.00
hours	**days**
36	1.50

In the report accompanying this spreadsheet (USGS SIR 2010-5102), vertical soil permeability (ft/d) is assumed to be one-tenth horizontal hydraulic conductivity (ft/d).

149.814	h(max)	**maximum thickness of saturated zone (beneath center of basin at end of infiltration period)**
18.574	Δh(max)	**maximum groundwater mounding (beneath center of basin at end of infiltration period)**

Ground-water Mounding, in feet	**Distance from center of basin in x direction, in feet**
18.574	0
18.484	25
18.211	50
17.746	75
17.072	100
14.986	150
11.633	200
5.847	300
2.875	400
1.352	500

Re-Calculate Now



Disclaimer

This spreadsheet solving the Hantush (1967) equation for ground-water mounding beneath an infiltration basin is made available to the general public as a convenience for those wishing to replicate values documented in the USGS Scientific Investigations Report 2010-5102 "Groundwater mounding beneath hypothetical stormwater infiltration basins" or to calculate values based on user-specified site conditions. Any changes made to the spreadsheet (other than values identified as user-specified) after transmission from the USGS could have unintended, undesirable consequences. These consequences could include, but may not be limited to: erroneous output, numerical instabilities, and violations of underlying assumptions that are inherent in results presented in the accompanying USGS published report. The USGS assumes no responsibility for the consequences of any changes made to the spreadsheet. If changes are made to the spreadsheet, the user is responsible for documenting the changes and justifying the results and conclusions.