Cambridge Waste Water Treatment Plant Relocation Project
Anglian Water Services Limited

Appendix 20.9 Hydrological Impact [redacted] Assessment (Site Selection Stage)

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Cambridge WWTP Relocation Project

Hydrogeological Impact Assessment

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Hydrogeological Impact Assessment

March 2021

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Executive summary

Cambridge City Council and South Cambridgeshire District Council are leading the regeneration of North East Cambridge (NEC). The existing Cambridge Waste Water Treatment Plant (WWTP) occupies a significant part of the North East Cambridge area designated for regeneration.

To facilitate the regeneration, Cambridge City Council, in partnership with Anglian Water Services Limited (Anglian Water), applied for funding from the Housing Infrastructure Fund. The funding will enable the relocation of Cambridge WWTP which is owned and operated by Anglian Water. The Government announced in March 2019 that funding would be granted for the relocation of Cambridge WWTP and, as a result, Anglian Water is currently planning the relocation. The project is classified as a Nationally Significant Infrastructure Project (NSIP). Therefore, in order for the development to be authorised, Anglian Water must make an application to the Secretary of State for Environment, Food and Rural Affairs (DEFRA) for a Development Consent Order (DCO).

The relocated Cambridge WWTP will treat waste water from:

- the Cambridge drainage catchment area, currently treated at the existing Cambridge WWTP; and
- the Waterbeach drainage catchment area located on the north side of the Cambridge drainage catchment area.

Three potential sites were selected following a staged appraisal for the relocation of the WWTP. The final preferred site is being chosen following public consultation and a further, more detailed, appraisal in Stage 4 – Final Site Selection. The WWTP relocation scheme comprises the following main components:

- A new WWTP at one of the three sites, as shown on Figure A.1.
- A tunnel transferring waste water from the existing Cambridge WWTP to the new WWTP.
- Shafts associated with the waste water transfer tunnel.
- A pipeline bringing waste water from the existing WWTP at Waterbeach to the new WWTP, and replacement of the existing Waterbeach WWTP with a pumping station.
- For Sites 1 and 2, a pipeline diversion to the existing waste water transfer network near the A14.
- Discharge pipelines, or a tunnel (with associated shafts), transferring the treated effluent from the new WWTP to an outfall on the River Cam.
- For discharge of the treated effluent, use of a new outfall either:
  - close to the location of the existing outfall on the River Cam, just downstream of the A14 river crossing to the north of Cambridge (associated with Sites 1, 2 or 3); or,
  - at a location on the river about 2 km downstream of the crossing (for Sites 1 or 2 only).

The precise routes of proposed tunnels and pipelines, and locations of the outfalls, are not available at present. However, potential corridors or zones for these components of the scheme have been identified, as shown on Figure A.1.

A preliminary water resources statement was produced, and submitted to the Environment Agency, to provide an overview of the potential impacts on surface water and groundwater

water resources of relocating the Cambridge WWTP to one of the three selected site areas (Mott MacDonald Ltd, 2020b).

The Environment Agency provided a consultation response in relation to the project proposals shared in the first phase of public consultation. The response detailed a number of points in relation to the potential impact of the proposals on the water environment. In addition, the Environment Agency indicated that a Hydrogeological Impact Assessment (HIA) should be carried out to support the selection of a final site for the relocation of Cambridge WWTP.

As a result, Anglian Water commissioned the production of a Hydrogeological Impact Assessment (HIA), the subject of this report. The HIA builds on the previous assessment of the potential impacts on the water environment provided in the Water Resources Statement (Mott MacDonald Ltd, 2020b).

This report has therefore been prepared to provide an initial hydrogeological assessment of the potential impacts that could arise at Sites 1, 2 or 3 as a result of construction or operation of a new Waste Water Treatment Plant (WWTP). The report does not consider detailed assessments, such as the use of numerical modelling, to provide supporting evidence, but uses data available at the time of writing, together with approximate calculations and analytical models, to support the conclusions. This document is intended to support the Stage 4 - Final Site Selection assessment and to provide further analysis for the Environment Agency. The hydrogeological impact assessment will, however, develop as the project progresses, although no significant changes are anticipated to the current conclusions.

The principal objectives of this HIA are to:

- Determine baseline conditions in relation to the groundwater and groundwater-dependent environment for each of the proposed sites and their surroundings.
- Identify any potential impacts of the proposed development on groundwater and these groundwater-dependent environments.
- Discuss the likely magnitude and significance of potential impacts.
- Consider appropriate mitigation measures for any identified potential impacts.
- Identify any potential residual impacts which may not be fully mitigated.

To cover these objectives, the HIA includes the following sections and assessments:

- Summary descriptions of the surface water features, geology, groundwater resources and use of groundwater for the area in and around the three potential sites. The extent and characteristics of the two Principal bedrock aquifers are described, comprising the Lower Greensand and the Grey Chalk. In addition, information is provided on Water Framework Directive (WFD) water bodies in the area, together with designated nature conservation sites, particularly those sites which include groundwater-dependent features.
- Hydrogeological conceptual models for each of the three sites. The Lower Greensand aquifer, confined below the Gault Formation, would be encountered in shaft excavations at Sites 1 and 2. The Grey Chalk is found in outcrop at Site 3. The conceptual models utilise the geological and groundwater information available for the area, as well as detailed geological logs obtained from the construction of boreholes at each of the sites as part of ground investigations undertaken in 2020. Aquifer hydraulic properties derived from borehole testing are also discussed, together with potential links between groundwater and surface water in and around the sites. The Quy Fen Site of Special Scientific Interest (SSSI), which is located on Grey Chalk, is discussed in detail in relation to Site 3.

- Potential temporary impacts on groundwater resources could result from the dewatering required during construction of shafts. The rates of dewatering required for the deepest shaft at each site, associated with the transfer tunnel, are estimated based on the geological logs and permeability test results for the bedrock/Principal aquifers present at the sites. In addition, permeability data from other sources, for example, existing test data for local boreholes and regional geotechnical and hydrogeological studies, are included in the assessment, leading to a range of potential dewatering rates. The calculations assume a theoretical worst case in which the shaft is unlined throughout the period of excavation. The dewatering rates are then used to estimate the potential aquifer drawdown which might occur temporarily in the area, including the potential drawdown at existing groundwater sources and groundwater-dependent nature conservation sites and surface water features.
- Actual methods of shaft construction are then considered, together with the effect these construction methods should have in reducing dewatering rates, and the resulting impact on groundwater resources and groundwater-dependent features.
- Mitigation and residual impacts are discussed in relation to dewatering, as well as the strategy for monitoring during shaft construction. The impacts of discharging to surface water drainage features during shaft dewatering are also assessed, together with the mitigation measures that would be required.
- The potential impacts on groundwater of other components of the proposed scheme are assessed in a further chapter of the HIA. These components include:
  - The tunnel transferring waste water from the existing Cambridge WWTP to the new WWTP;
  - Waste water transfer pipelines and the effluent discharge pipeline or tunnel; and
  - Permanent foundations and below-ground structures at the new WWTP site.
- Construction methods, potential impacts, mitigation methods and any residual impacts are considered for each scheme component. Dewatering rates for construction of sections of the pipelines are also estimated in order to provide a very approximate indication of potential dewatering requirements.

One of the sites (Site 2) is close to a major landfill site, located in the Gault Formation near Milton. As a result, concerns have been expressed by the Environment Agency that temporary dewatering during shaft construction could mobilise groundwater contaminants originating from the landfill. As a result, contaminant modelling was undertaken for Site 2, and also Site 1 at a greater distance from the landfill, to assess:

- the potential for contaminant migration from the landfill through the Gault Formation;
- the risk of contamination in dewatering discharge for deep excavations in shafts, or of encountering contamination during tunnelling; and
- the risk posed to groundwater quality in the Lower Greensand aquifer.

Construction and operation of a WWTP at Site 3 could give rise to the potential for contamination from the site to migrate in shallow groundwater through the Grey Chalk to a watercourse (the Black Ditch) located down-gradient of the site. Contaminant transport modelling was also undertaken, therefore, to provide a better understanding of the risks during operation of a new WWTP at Site 3 to nearby nature conservation sites connected or located close to the Black Ditch. These conservation sites comprise Stow Cum Quy Fen SSSI and Allicky Farm Pond County Wildlife Site.

Overall conclusions

Overall conclusions are provided for all the assessments of impact and risk to the groundwater system covered in the HIA. These assessments have been taken into account in the Stage 4 – Final Site Selection process. It is considered that all potential impacts on groundwater, as well as abstractions, surface water and nature conservation sites dependent on groundwater, are either insignificant or can be mitigated for by means described in the HIA, with the following exceptions:

- Providing mitigation for any loss or reduction of spring discharges, seepages and flows in watercourses in the Lower Greensand outcrop, resulting from construction of the shaft at Site 1 or Site 2 by underpinning, is unlikely to be practicable in the event that test pumping indicates that dewatering could affect these features. The impact would, however, be temporary, for up to four months during dewatering, together with a subsequent period of aquifer recovery.
- It is possible that the water level in the Cottenham Moat County Wildlife Site (CWS) could be affected temporarily as a result of dewatering in the Lower Greensand aquifer at Site 1 or Site 2, although any connection between the moat and the aquifer is unproven and may not exist. At this stage it is also uncertain whether mitigation, if required at the CWS, would be practicable. Further assessment may be appropriate to determine whether there could be a link between the CWS and the aquifer.

The Environment Agency has suggested that any groundwater removed during temporary dewatering for shaft construction, particularly from shafts in the Lower Greensand, should be recharged to the aquifer. However, this would be a complex undertaking and is not considered practicable or worthwhile to mitigate for a temporary impact.

If the potential impacts of dewatering of the shaft at Site 1 or 2 indicate that abstraction at an existing groundwater source may not be possible during dewatering, actions can be taken to ensure the required supply can be maintained. Such actions could include:

  - lowering the pump in the borehole, or combining this action with provision of a replacement pump; or,
  - providing an alternative water supply for the duration of dewatering and the period of subsequent groundwater level recovery.

Dewatering requirements would be minimal for shaft construction at any of the three sites using secant piles. As a result, with secant piling, the impact on groundwater in the Lower Greensand or Grey Chalk aquifer, and any abstractions dependent on or supported by groundwater, would be negligible. The impact of shaft dewatering required during construction by underpinning should also be minimal in the Grey Chalk at Site 3.

Contaminant modelling has provided supporting information to confirm that:

- construction of the new WWTP at Site 1 or 2 near the existing landfill at Milton is very unlikely to affect groundwater quality; and
- In the unlikely event of any potential spills or leaks of contaminants associated with a WWTP at Site 3, they would be unlikely to affect water quality at Quy Fen SSSI and Allicky Farm Pond CWS.

1 Introduction

1.1 Project background

Cambridge City Council and South Cambridgeshire District Council are leading the regeneration of North East Cambridge (NEC). The existing Cambridge Waste Water Treatment Plant (WWTP) occupies a significant part of the North East Cambridge area designated for regeneration.

To facilitate the regeneration, Cambridge City Council, in partnership with Anglian Water Services Limited (Anglian Water), applied for funding from the Housing Infrastructure Fund. The funding will enable the relocation of Cambridge WWTP which is owned and operated by Anglian Water. The Government announced in March 2019 that funding would be granted for the relocation of Cambridge WWTP and, as a result, Anglian Water is currently planning the relocation.

The relocation project will deliver benefits of national significance and regional and local importance and is a project that is complex in its nature and scale. The Secretary of State for Environment, Food and Rural Affairs has made a direction under section 35 of the Planning Act 2008 confirming that the relocation of Cambridge WWTP is to be treated as a development of national significance for which development consent under that Act is required. Anglian Water will therefore, in due course, submit an application for a development consent order (DCO) for the relocation project.

As part of the DCO application and related Environmental Impact Assessment (EIA) process, Anglian Water will demonstrate the robust process it has undertaken to identify a suitable location for the new WWTP.

Anglian Water commissioned a detailed site selection study, to investigate and assess potential locations for the new WWTP. The early proposals for the project, including the three potential locations for the new WWTP identified in Stage 3 - Fine Screening (Mott MacDonald Ltd, 2020a) of the site selection process, were shared with the public and stakeholders in phase one non-statutory consultation in July 2020.

This phase of non-statutory consultation aimed to gain local knowledge from the public and identify any concerns that technical stakeholder may have in relation to the proposals. This additional information would then be considered during the final stage of the site selection process (Stage 4 – Final Site Selection), in order to identify the site that will be taken forward to the next phase of statutory consultation and the subsequent DCO application.

1.2 Purpose of this report

The Environment Agency provided a consultation response in relation to the project proposals shared in the first phase of public consultation. The response detailed a number of points in relation to the potential impact of the proposals on the water environment, and indicated that a Hydrogeological Impact Assessment (HIA) should be carried out to support the selection of a final site for the relocation of Cambridge WWTP.

Therefore, Anglian Water commissioned the production of a HIA, the subject of this report, which builds on the previous high-level assessment of the potential impacts on the water environment provided in the Water Resources Statement (Mott MacDonald Ltd, 2020b). The HIA also utilised the results of a preliminary phase of ground investigation, a site visit to Quy Fen SSSI and further desk based study to improve the understanding of local hydrogeology and assess potential impacts.

The HIA has been undertaken to provide an initial hydrogeological assessment of the potential impacts that could arise at Sites 1, 2 or 3 as a result of construction or operation of a new WWTP. The report does not consider detailed assessments, such as the use of numerical modelling, to provide supporting evidence, but uses data available at the time of writing, together with approximate calculations and analytical models, to support the conclusions. This document is intended to support the Stage 4 - Final Site Selection assessment and to provide further analysis for the Environment Agency. The hydrogeological impact assessment will, however, develop as the project progresses, although no significant changes are anticipated to the current conclusions.

The principal objectives of this HIA are to:

- Determine baseline conditions in relation to the groundwater and groundwater-dependent environment at each of the proposed sites and their surroundings.
- Identify any potential impacts of the proposed development on these environments.
- Discuss the likely magnitude and significance of potential impacts.
- Consider appropriate mitigation measures for any identified potential impacts.
- Identify any potential residual impacts which may not be fully mitigated.

1.3 Scheme description

The proposed scheme comprises the following components:

- A new WWTP at one of the three sites, indicated on Figure A.1. The WWTP will include inlet works, several sets of above-ground tanks and buildings for various purposes in the treatment process, digesters, a gas holder and flare stack, as well as offices. The relocated WWTP will require an operational footprint of up to 22 hectares (22ha). Any landscaping around the site would be in addition to this area.
- A tunnel transferring waste water from the existing Cambridge WWTP to the new WWTP[1].
- Shafts associated with the transfer tunnel. The shafts would be located at the existing Cambridge WWTP and at the new WWTP, and at intermediate locations as required for tunnel construction.
- A pipeline bringing waste water from the existing WWTP at Waterbeach to the new WWTP, and replacement of the existing Waterbeach WWTP with a pumping station.
- A pipeline diversion to the existing waste water transfer network, intercepting waste water in an existing pipeline close to the A14 and diverting the flow to either Site 1 or Site 2.
- Discharge pipelines, or a tunnel (with associated shafts), transferring the treated effluent from the new WWTP to the outfall on the River Cam.
- A new outfall for discharge of the treated effluent, either:
  - close to the location of the existing outfall on the River Cam, just downstream of the A14 crossing (associated with options for Sites 1, 2 or 3); or,

[1] The new waste water transfer tunnel will intercept the tunnel that currently conveys waste water into he existing WWTP and divert hem to the new WWTP. The location of this interception point will be within the footprint of the existing WWTP site. The existing WWTP will be decommissioned once the new WWTP is operational.

- at a location about 2 km downstream of the crossing (associated with options for Sites 1 or 2 only).

The precise routes of proposed tunnels and pipelines, and locations of the outfalls, are not available at present. However, potential corridors or zones for these components of the scheme have been determined and these are shown on Figure A.1. The area within 1 km of scheme components is also included on this figure. This area is provided for scale to assist with a visual assessment of the location of water resources features in relation to scheme components. However, it does not comprise the study area for this HIA. Depending on potential impacts of different scheme components, the study area takes into account more distant groundwater dependent features.

The construction of the waste water transfer tunnel could be completed using a bored segmental lining or pipe-jacking method. The main difference between these construction methods is that pipe-jacking would require intermediate shafts at regular intervals of about 500 m along the tunnel route. It is currently assumed that pipe-jacking would only be a viable option for constructing the waste water transfer tunnel to Site 3. It is considered by Anglian Water that it would not be feasible to site multiple intermediate shafts within the waste water transfer tunnel corridors to Sites 1 and 2 without significant adverse impacts on surface receptors. Therefore, in this HIA, pipe-jacking of the waste water transfer tunnel is only considered for Site 3.

2 Water resources background

2.1 Location and surface water features

The sites are within the area defined by the Environment Agency in the Anglian River Basin Management Plan (RBMP) as the Cam and Ely Ouse management catchment (Department for Environment Food & Rural Affairs, 2015). The management catchment comprises two operational catchments. Sites 1 and 2 are divided between the Lower Cam operational catchment, and the South Level and Cut-Off Channel operational catchment located to the west and north. Site 3 is situated wholly within the Lower Cam operational catchment. Sites 1 and 2 are located to the west of the River Cam, with Site 3 to the east of the river. A location plan is shown in Figure A.1.

The precise routes of proposed tunnels and pipelines, and locations of the outfalls, are not available at present. However, potential corridors or zones for these components of the scheme have been determined and these are shown on Figure A.1. The area within 1 km of scheme components is also included on this figure. This area is provided for scale to assist with a visual assessment of the location of water resources features in relation to scheme components. However, it does not comprise the study area for this HIA. Depending on potential impacts of different scheme components, the study area takes into account more distant groundwater dependent features.

The area of all three sites is relatively low lying with little variation in elevation. Sites 1 and 2 are located at about 10 mAOD, with maximum elevations of 12 mAOD indicated on Ordnance Survey (OS) 1:25000 scale mapping in the area. The River Cam is located 2 to 3 km to the east of Sites 1 and 2. Downstream (to the north) of the A14 crossing at Milton, the elevation of the River Cam is below 5 mAOD.

Ordnance Survey mapping indicates that Site 3 is located around the 10 mAOD contour on the east side of the River Cam. There is a general reduction in elevation from west to east across Site 3, towards a set of drainage features connected to Black Ditch. The Black Ditch discharges to the north along the boundary of Stow-cum-Quy Fen to Bottisham Lode. Quy Water, located to the west of Site 3 and the Black Ditch, is the main watercourse contributing to Bottisham Lode. Bottisham Lode discharges to the River Cam near Waterbeach, about 5 km downstream of the A14 crossing, as shown on Figure A.1.

Some substantial still water bodies are also present in the area, including the following:

- at Milton Country Park between Milton and the A14;
- adjacent to the north side of the A14 and south of the busway NCN51; and
- Leyland Water between Waterbeach and Landbeach.

Several smaller lakes and ponds are also located in the area.

Data from the UK National River Flow Archive (NRFA) indicates that the annual rainfall in the Quy Water catchment, upstream of Lode, averaged 620mm over the past 20 years (National River Flow Archive, 2020).

Surface water flood risk zones are shown on Figure A.2.The main areas of flood risk concern are associated with the River Cam and tributaries. One of the criteria used in selecting Sites 1, 2 and 3 was that the sites were located entirely within Flood Zone 1 (less than 1 in 1,000 annual probability of river flooding) as defined by the Environment Agency.

Much of the land across the areas of the three sites is used for arable agriculture. Site 2 is located adjacent to an extensive area of landfill on the east side of the site. Belts of woodland are also present in and around Site 2.

2.2 Geology and groundwater

2.2.1 Bedrock

The bedrock geology in the area of the three sites is shown in Figure A.3. It comprises the following sequence, listed from youngest to oldest formations:

- Grey Chalk, comprising:
  - Zig Zag Chalk Formation
  - Totternhoe Stone
  - West Melbury Marly Chalk Formation
- Gault Formation
- Lower Greensand (Woburn Sands Formation)
- Kimmeridge Clay Formation (underlain by the Ampthill Clay and West Walton Formations)

The bedrock formations dip gently (at approximately 0.5°) to the south east, with the youngest beds, the Zig Zag Chalk Formation and Totternhoe Stone present at outcrop in the south east corner of the area shown on Figure A.3. The Totternhoe Stone is a hard band in the Grey Chalk and an important aquifer flow horizon in south Cambridgeshire. Several Chalk springs are located close to the outcrop of the Totternhoe Stone to the east of Quy Water and the low lying areas along the River Cam. Watercourses fed by these springs contribute to channels, including Bottisham Lode, which drain to the River Cam. However, the Totternhoe Stone does not extend any further into the area than the outcrop between the Zig Zag Chalk Formation and the West Melbury Marly Chalk Formation.

The West Melbury Marly Chalk Formation is located towards the base of the Chalk (in the Grey Chalk Sub-group) and is described as grey, or dark grey, and marly in several borehole logs in the vicinity of Site 3, available on the British Geological Survey (BGS) website. The Cambridge Greensand Member (previously known as the Upper Greensand) may possibly be present in the formation at the boundary with the underlying Gault Formation.

The Cambridge Greensand Member is not present in outcrop in the Cambridge area but is described by British Geological Survey (BGS) in the Hydrogeological Map of the area between Cambridge and Maidenhead (British Geological Survey , 1984) as comprising glauconitic, micaceous, calcareous, fine grained sandstones or siltstones elsewhere in the region. There is, however, no indication of any distinctive sandstone or siltstone in geological logs for existing boreholes which have been drilled previously through the contact between the Grey Chalk and Gault Formation in the vicinity of Site 3 (British Geological Survey, Geology of Britain viewer, 2020a).

BGS indicates that there are no significant abstractions solely from the Cambridge Greensand Member in the region covered by the hydrogeological map. In addition, no significant aquifer horizons would be expected in the marly chalk forming the West Melbury Marly Chalk Formation, as described in boreholes near Site 3. Both the permeability of the marly chalk and the transmissivity of the formation are expected to be low. As a result, groundwater yields and any discharges from the West Melbury Marly Chalk Formation are also likely to be small.

The Gault Formation, comprising a pale grey marl to dark grey silty clay, with a basal bed of glauconitic or phosphatic nodules, underlies Sites 1 and 2. The total thickness of the Gault Formation in the area is about 35m based on geological logs for boreholes close to the contact with the overlying Grey Chalk.

The Lower Greensand (Woburn Sands Formation) is present in a narrow outcrop in the north west of the area and underlies the Gault Formation. It is also present in an anticlinal structure along the River Cam to the north east of Sites 1 and 2. BGS describes the formation generally as comprising a fine to coarse-grained rounded marine quartz sandstone (or loose sand), glauconitic in part, commonly silty with few clay seams, typically grey or greenish grey, weathering to ochreous yellow-brown.

Geological logs available on the BGS website indicate the Lower Greensand is about 8 to 10 m thick in the vicinity of Sites 1 and 2 (British Geological Survey, Geology of Britain viewer, 2020a). In a borehole located close to Site 2, drilled as part of site investigations for construction of the A14, the formation is described as comprising grey to black sandy clay or clayey sand or sandstone, specked with orangy-brown in places, with occasional pockets or laminated bands of grey clay and frequent phosphate nodules in some horizons. Test pumping of a borehole in the Lower Greensand at Sunclose Farm on Butt Lane, between Sites 1 and 2, produced a yield of 1.5 l/s with a drawdown of 12.3 m over 24 hours, indicating that the formation is low yielding in the area.

The Lower Greensand is underlain by the Kimmeridge Clay which is present in outcrop to the west of Cottenham.

Both the Chalk and the Lower Greensand are classified by the Environment Agency as Principal aquifers. However, based on available geological logs in the study area, significant aquifer horizons are unlikely to present in the West Melbury Marly Chalk Formation which underlies Site 3. In the case of the Lower Greensand, the materials are fine and variable, and the aquifer is of limited thickness. Therefore, neither aquifer is likely to produce substantial yields at any groundwater abstraction sites nearby. Seepages from the West Melbury Marly Chalk Formation may, however, contribute to local drains and watercourses.

The Gault Formation is classified by the Environment Agency (EA) as an unproductive aquifer (effectively a non-aquifer). An extensive borrow pit from which Gault Formation was excavated for development of the A14 is located on the east side of Site 2. The borrow pit has subsequently been used for landfill (Milton Landfill).

There are five EA monitoring boreholes located within a 2 km radius of the proposed WWTP sites, all of which are within the Lower Greensand Formation. The groundwater level in these boreholes ranges from about 2.6 mAOD to 6.5 mAOD, or 1.5 mbgl to 7.1 mbgl.

There are no EA monitoring boreholes penetrating the Chalk which could assist in this HIA as the closest is over 6 km from the proposed sites.

2.2.2 Superficial deposits

Superficial river terrace deposits, comprising sand and gravel, overlie the bedrock across most of Site 2, together with small areas of Site 1 and 3, as shown in Figure A.4. The river terrace deposits have a thickness of up to about 2.5 m in the area around Site 2, as indicated by logs for boreholes held by BGS (British Geological Survey, Geology of Britain viewer, 2020a). Otherwise the three sites are located directly over bedrock below the soil/sub-soil.

BGS mapping indicates that alluvium, comprising clay, silt, sand and gravel, is present along the valley floor of the River Cam, with extensive river terrace deposits at a slightly higher elevation, particularly along the western flank of the valley. Borehole logs indicate that sandy clay and peat are present to a depth of 6 to 7 m in parts of the valley floor at the A14 crossing, overlying sand and gravel to a depth of up to about 8 m. About 0.5 km further downstream however, the superficial deposits have a depth of approximately 3.2 m, indicating that there is considerable variability in thickness (and composition) of superficial deposits along the valley. The river terrace deposits on the western side of the valley have a recorded depth of nearly 7 m at one location, but are more typically 2.5 to 4 m in depth. Peat is present in some areas to the east of Waterbeach.

River terrace deposits and alluvium are classified by the Environment Agency as Secondary A aquifers. Peat is classified as an unproductive aquifer.

The major still water bodies are located in areas of river terrace deposits underlain by Gault Formation. These water bodies have, presumably, resulted from, or been created by, excavation of river terrace deposits. They are likely to be fed by groundwater, acting as seepage pits intercepting the perched water table in the superficial deposits above the Gault Formation, as well as direct rainfall and possibly some surface water flow via drains.

2.3 Groundwater abstraction and aquifer vulnerability

There are no groundwater abstractions for public water supply within the area shown on Figure A.1. No part of the area in and around the sites, including the area within 1 km of scheme components on Figure A.1, is within an Environment Agency designated source protection zone (SPZ) for any public water supply groundwater abstraction. There is a source protection zone in the Chalk outcrop (SPZ3, the total contributing recharge catchment around a source) that extends into the south east corner of the study area shown on Figure A.3, which is approximately 3 km from Site 3. No source protection zones associated with the Lower Greensand aquifer are located in the study area.

The Environment Agency has mapped aquifer vulnerability nationally using information on recharge, soil leaching properties, superficial cover and the unsaturated zone above the groundwater table. Aquifer vulnerability for the three sites is classified as follows:

- Most of the area of Site 1 is classified as unproductive with minor areas of medium to low risk. The Agency's technical summary indicates that '*Activities in areas of unproductive strata are unlikely to represent a risk to groundwater due to the low permeability of the deposits and the protection which they provide to any aquifers that may be present beneath. However, water run-off from these areas may represent a risk to surface water or adjacent groundwaters*.' Areas of low risk '*provide the greatest protection to groundwater from pollution. They are likely to be characterised by low leaching soils and/or the presence of low permeability superficial deposits*'. Medium risk indicates that conditions provide some groundwater protection.
- Site 2 – mostly medium to low risk with some unproductive areas.
- At Sites 1 and 2, the medium to low risk category corresponds generally to areas with superficial deposits comprising river terrace deposits (sand and gravel). Areas classified as unproductive have no superficial deposits overlying the Gault Formation.
- Site 3 is located directly on the Grey Chalk in a high-risk area which the Agency identifies as being '*able to easily transmit pollution to groundwater*'. High risk areas are '*characterised by high leaching soils and the absence of low permeability superficial deposits*'. Site 3 is also identified as being in an area with '*soluble rock risk*' in which '*solution features that enable rapid movement of a contaminant may be present*'.

2.4 Abstraction licensing strategy (ALS) status

The proposed sites are covered by the Cam & Ely Ouse ALS, which includes both the Cam and Ely Ouse Chalk and the Woburn Sands. The ALS status for all three sites is '*water not available*'. This status means that the '*groundwater unit balance shows more water has been abstracted based on recent amounts than the amount available*' and that further licences will not be granted (Environment Agency, 2020a).

2.5 Water Framework Directive (WFD) status

The site areas are located on the *Cam and Ely Ouse Chalk* and the *Cam and Ely Ouse Woburn Sands* groundwater bodies.

The Chalk is a Principal aquifer and a Drinking Water Protected Area. Both the qualitative and chemical status of this groundwater body are classed as poor by the WFD. The reasons for not achieving good status include impacts of groundwater abstraction, pollution from agriculture, transport drainage and sewage discharge.

The Cam and Ely Ouse Woburn Sands is also both a Principal aquifer and a Drinking Water Protected Area. Groundwater flow is generally dominated by matrix flow with some fracture flow. The limited outcrop area to the west is particularly vulnerable to any potentially polluting activity at the surface. The qualitative and chemical status of the Woburn Sands are both classed as good, with concerns for agricultural and rural land management pollution. However, these concerns are not significant enough to reduce the classification of the overall groundwater body (Environment Agency, 2020b).

2.6 Surface water and groundwater dependent nature conservation sites

This section describes the designated nature conservation sites which may be partly or wholly dependent on surface water and groundwater, and could, potentially, be affected by the construction or operation of the scheme. The locations of the sites are shown on Figure A.2. The sites include:

- statutory designated sites, comprising Ramsar, Special Areas of Conservation (SAC), Sites of Special Scientific Interest (SSSI), National Nature Reserves (NNR) and Local Nature Reserves (LNR); and,
- non-statutory designated sites, comprising County Wildlife Sites (CWS).

All the statutory and non-statutory designated nature conservation sites shown on Figure A.2 were initially considered in the HIA.

Some additional nature conservation sites are present within the area of the map in Figure A.2, but are not shown for the following reasons:

- the citations for the sites indicate there is no dependency on a surface water or groundwater source;
- the sites are at locations in catchments up-gradient of the 1 km buffer to the south of the existing WWTP and should not be affected; or,
- the sites are to the north of the Great Ouse at locations which are beyond any potential impact on the river due to the scheme and should not, therefore, be affected.

In addition, some SSSIs in the area are designated for geology rather than reasons related to nature conservation.

There are no Special Protection Areas (SPA) or ancient woodlands with groundwater or surface water dependent features which would be affected by the construction or operation of the scheme.

Summary details are provided in the following section for all the designated nature conservation sites shown on Figure A.2.

2.6.1 Statutory designated sites

There are seven statutory designated nature conservation sites, which may be partly or wholly dependent on surface water or groundwater shown in Figure A.2. Summary details are provided in Table 2.1.

Table 2.1: Statutory Designated Sites

Designated site	Area (ha)	Description	Water dependent features
Wicken Fen Ramsar, SAC, NNR, SSSI	254	Designated as a remnant of the East Anglian peat fens, Wicken Fen has been preserved as a flood catchment area with water levels controlled by sluice gates. Traditional management has created a mosaic of habitats from open water to sedge and litter fields. The site supports hydrologically dependent species including nationally scarce plants and Red Data Book invertebrates, There are reedbeds and pools attractive to breeding wetland birds and wintering wildfowl. Note that only a part of Wicken Fen is shown on Figure A.2– the site extends out to the north-east of the area shown on the map.	This site is highly dependent on surface water and is subject to winter flooding, potentially indirectly connected to the River Cam.
Cam Washes SSSI	169	Designated for a series of low lying pastures which are subject to seasonal flooding. Includes damp grassland and wet tussocky fields, with associated pools, ditches and river margins. The relative freedom from disturbance makes this an important site for numbers and diversity of wintering and breeding wildfowl and waders. Important at times of flood when birds may be unable to feed on the internationally important Ouse Washes further downstream.	Located along the banks of the River Cam. Hydrological elements of the site include pools, ditches and river margins.
Stow-cum-Quy Fen SSSI	30	Designated for floristically rich calcareous loam pasture.	Pools formed on Chalk Marl support a range of aquatic plants including some uncommon species. The open water habitats are particularly attractive to dragonflies and damselflies. Pools on the periphery of the site add to the general variety of habitats in the locality.
Upware North Pit SSSI	1.1	The site consists of a series of flooded pits divided by bunds. The freshwater habitats include one of only two localities in the country for the water germander *Teucrium scordium*, a plant listed in the British Red Data Book. In addition, the pits are of general value for the wetland communities present and some other uncommon plants are recorded.	Flooded pits/ freshwater habitat, possibly connected to the River Cam.
Wi braham Fens SSSI	61	Designated for a large area of fen and neutral grassland with associated scrub and open water communities.	Fen designation. Drainage ditches throughout the site are relatively unpolluted, thus being rich in emergent plants.
Worts Meadow LNR	5.7	Designated for species-rich hedgerows and three ponds containing great crested newts *Triturus cristatus.*	Ponds with great crested newts.
Bramblefields LNR	2.8	Habitat is a mixture of grassland and scrub.	Includes a pond which contains newts.

2.6.2 Non-statutory designated sites

There are 13 non-statutory designated sites (CWS), which may be partly or wholly dependent on surface water or groundwater shown in Figure A.2. All these sites either have some reference to water dependency in their citation, or potential water dependency is apparent from the site name. The CWSs that could potentially be affected by the construction or operation of the scheme are discussed in the following sections. It is noted that all 13 sites will be considered further in the water resources assessment in the environmental statement.

2.6.3 Initial comments on water dependency and potential impacts

The following points are noted for some of the sites in relation to water dependency, location and the potential impacts of the scheme:

- The River Cam CWS is a part of the river (together with adjacent semi-natural habitat) which has not been grossly modified by canalisation and/or poor water quality. Additionally, the CWS includes areas with concentrations of mature pollard willows. The proposed discharge locations for treated effluent, and the crossing point to Site 3 for the pipeline from the Waterbeach WWTP, are located within the CWS. Another section of the River Cam CWS is approximately 1 km downstream from the crossing point to Site 3 for the pipeline from the Waterbeach WWTP.
- As indicated in Section 2.1, the elevation reduces from west to east across Site 3, towards a set of drainage features connected to Black Ditch. Further downstream, Black Ditch is connected to a part of the Stow-cum-Quy Fen SSSI (see Section 3.2.2.5 for further details). The Allicky Farm Pond CWS is also located close to Black Ditch, and within Flood Zone 3 along the ditch, as indicated on Figure A.2. Any surface water from Site 3, or possibly seepages originating from groundwater in the West Melbury Marly Chalk Formation underlying Site 3, may discharge to Black Ditch. As a result, the discharge from the area of Site 3 may drain through a part of Stow-cum-Quy Fen and, depending on surface water connections and water level conditions, might affect Allicky Farm Pond.
- Cottenham Moat CWS is approximately 4 km from the nearest site (Site 1), with no direct surface water connection evident from OS mapping. The moat is shown by BGS to be located on Kimmeridge Clay. However, the location is adjacent to the contact with the Lower Greensand outcrop which overlies the Kimmeridge Clay. It is possible, therefore, that a connection might exist between the moat and the Lower Greensand aquifer. Potential impacts of temporary dewatering in the Lower Greensand aquifer during shaft construction may need to be considered at Cottenham Moat. The moat supports great crested newts.
- Worts Meadow LNR is located about 330 m west of the Waterbeach pipeline route to Sites 1 and 2. As indicated in Table 2.1, the LNR includes three ponds. The site is underlain partially by river terrace deposits which may support the ponds. River terrace deposits are also present along the pipeline route at the closest location, and temporary dewatering of these deposits may be necessary during trench excavation and installation of the pipeline. However, at a distance of more than 300 m, any dewatering required would not be expected to affect the groundwater in the river terrace deposits at the LNR.
- Wilbraham Fens SSSI is located upstream and more than 1 km from Site 3 at the nearest point. No impact on surface water or groundwater would be expected at the SSSI as a result of the construction and operation of the scheme.
- Ditton Meadows CWS and Little Wilbraham River CWS are likely to be upstream of any impact of the construction and operation of the scheme.

All the statutory and non-statutory designated nature conservation sites shown on Figure A.2 were initially considered in the HIA. Some have been eliminated as a result of location, as indicated above. In addition, other sites shown on Figure A.2 are likely to have no direct connection to water features which could be affected by the scheme. These sites include Cambridge Road Willow Pollards CWS and Clayhithe Pollard Willows CWS. Although the reference to willows indicates water dependency, any groundwater or surface water features in the vicinity are not likely to be affected by the scheme.

As can be seen in Figure A.2, many of the nature conservation sites are located a substantial distance downstream of the area in which activities associated with the construction and operation of the scheme will take place. These sites include Cam Washes SSSI, Wicken Fen Ramsar, SAC, NNR and SSSI, and Upware North Pit SSSI. The closest of these sites, Cam Washes SSSI, is located about 6 km from the potential downstream location for the treated effluent discharge, and 3 km downstream of the possible river crossing for the pipeline to Site 3.

For these three sites, and other downstream sites at a substantial distance from the scheme, it will be assumed that potential impacts during the construction of the scheme, such as pollution of watercourses, will be avoided by application of measures included in the code of construction practice (CoCP). As a result, no special mitigation measures are likely to be needed for these downstream sites.

Based on the above information, nature conservation sites have been identified which require further consideration within this HIA. Table 2.2 provides a list of these sites, the elements of the scheme that could affect them and references to the relevant assessment sections.

Table 2.2: Nature conservation sites to be assessed

Nature conservation site	Scheme elements
Stow-cum-Quy Fen SSSI	Drainage from site 3 (See Section 6.2) Temporary dewatering in the West Melbury Marly Chalk Formation at site 3 (See Section 4)
Allicky Farm Pond CWS	Drainage from site 3 (See Section 6.2) Temporary dewatering in the West Melbury Marly Chalk Formation at site 3 (See Section 4)
Cottenham Moat CWS	Temporary dewatering in the Lower Greensand aquifer during shaft construction at sites 1 and 2 (See Section 4)
River Cam CWS	Treated effluent discharge pipeline for all three sites (See Section 5.3) Waterbeach transfer pipeline (See Section 5.3)

It is noted that the potential impacts on all surface water and groundwater dependant nature conservation sites indicated in this report will also be considered in the water resources assessment in the environmental statement. As part of the biodiversity assessment required for the project, a Habitat Regulations Assessment (HRA) screening will be undertaken in relation to Wicken Fen Ramsar, SAC, NNR and SSSI.

3 Conceptual hydrogeology for site areas

3.1 Ground investigation

A preliminary ground investigation, comprising dynamic sampling and rotary cored boreholes, was carried out to assess the geological, hydrogeological and geotechnical conditions at the three sites, and the treated effluent tunnel corridor from Sites 1 and 2. In-situ testing and sampling was carried out with standpipes installed within each borehole. The investigation was carried out between August and October 2020 and consisted of five wireline rotary cored boreholes, referenced BH01 to BH05. The final depths of these boreholes range between 30.0 and 40.5mbgl (A F Howland Associates, 2020). Details of the strata encountered, piezometer installations, in-situ and laboratory testing, and groundwater monitoring were all recorded.

The cored borehole locations can be seen in Figure A.5. The boreholes at the potential WWTP sites were labelled as follows, in reverse order to the site reference numbers:

- BH01 at Site 3;
- BH02 at Site 2; and,
- BH03 at Site 1.

A 3D geological model was constructed with Leapfrog Works software utilising borehole data obtained from freely available BGS data (British Geological Survey, Geology of Britain viewer, 2020a) and the additional five boreholes drilled during the ground investigation. Two cross sections were drawn perpendicularly through the centre of each site, based on the model, and then annotated as shown in Figure A.7 to Figure A.12. These cross sections assist with predicting what ground conditions could be expected during construction.

The details from these site investigations are used, together with the existing hydrogeological data available from reference sources and other studies, to describe the conceptual hydrogeology for each site in the following sections.

3.2 Site specific conceptual hydrogeological models

3.2.1 Sites areas 1 & 2

3.2.1.1 Site specific hydrogeology

Borehole logs for BH02 and BH03 from the ground investigation can be used to predict at what depth geological layers will be encountered on each site. BH03 is located on the eastern edge of Site 1. BH02 is located towards the west of Site 2. Locations are shown on Figure A.5.

Table 3.1: Ground investigation borehole summary - Sites 1 and 2

	BH03 – Site 1			BH02 – Site 2		
Formation	Top of formation (mAOD)	Top of formation (mbgl)	Thickness of formation (m)	Top of formation (mAOD)	Top of formation (mbgl)	Thickness of formation (m)
Topsoil	8.7	Ground level	0.3	11.4	Ground level	0.4
Superficial deposits	8.4	0.3	1.4	11.0	0.4	1.7
Gault Formation	7.0	1.7	26.1	9.3	2.1	19.9
Lower Greensand	**-18.1**	**26.8**	**7.7**	**-10.6**	**22.0**	**9.3**
Kimmeridge Clay	-25.8	**34.5**	5.5 (to base of borehole, completed at 40.0 mbgl)	-19.9	**31.3**	9.2 (to base of borehole, completed at 40.5 mbgl)

The geological descriptions for the layers encountered in the borehole logs for these sites are summarised as follows:

- Topsoil – Dark brown silty, sandy Clay with some gravel.
- Superficial deposits – Clayey fine to coarse Sand, some with gravel, and sandy Clay.
- Gault Formation – Firm to stiff, pale grey, silty calcareous Clay.
- Lower Greensand Formation
  - Site 1 (BH03): predominantly very weak, greenish grey, silty, fine to coarse grained Sandstone, some of which is extremely weakly cemented, with evidence of generally close spaced, sub-horizontal fractures in some sections.
  - Site 2 (BH02): predominantly very weak, greenish grey to mottled grey-brown, silty, fine to coarse grained Sandstone, some of which is cemented or includes a little quartzite gravel, with bands of silty clay and silty sand. Close to wide spaced, sub-horizontal fractures noted in some sections.
- Kimmeridge Clay – Stiff fissured grey silty calcareous Clay and Mudstone.

There is a marked variation in the level of the top of the Lower Greensand Formation between the sites as can be seen in Table 3.1. At Site 2 the Lower Greensand Formation was encountered at -10.6 mAOD, while at Site 1 it was at -18.1 mAOD. The thickness of the Lower Greensand also varied between the sites (7.6 m at Site 1 and 9.3 m at Site 2). The top of the Lower Greensand therefore declines at an angle of about 0.2° in an east-north-easterly direction between BH02 at Site 2 and BH03 at Site 1. As would be expected, this is lower than the general dip of bedrock formations (at approximately 0.5°) to the south east.

Cross sections taken from the Leapfrog model beneath these sites can be seen in Figure A.8 to Figure A.10.

3.2.1.2 Groundwater levels and aquifer properties

Groundwater levels recorded in boreholes which penetrate into the Lower Greensand Formation at the Milton Landfill, adjacent to Site 2 and approximately 500 m from Site 1, show levels of between 4 and 6 mbgl. At this depth, the water table lies within the Gault Clay at both sites, hence confirming the confined condition of the Lower Greensand aquifer.

As indicated in Section 2.2.1, test pumping of a borehole in the Lower Greensand at Sunclose Farm on Butt Lane, between Sites 1 and 2, produced a yield of 1.5 l/s with a drawdown of 12.3 m over 24 hours. Applying the Logan approximation (Logan, J., 1964) to the yield/drawdown data gives:

- a transmissivity of about 13 m^2/d; and,
- an average permeability of 1.6 m/d, equivalent to 1.9 x 10^{-5} m/s, based on the aquifer thickness of 8 m given in the geological log for the site.

For comparison, in the Cam Bedford Ouse groundwater model (AMEC Environmental & Instrastructure UK Ltd, 2015), the Environment Agency applied a hydraulic conductivity/ permeability value of about 14 m/d to the area between Sites 1 and 2, equivalent to 1.6 x 10^{-4} m/s. It is not known how this value, which is almost an order of magnitude higher than the value derived by the Logan approximation using data for Sunclose Farm, was determined. However, the BGS website provides summary information on test pumping for a borehole constructed near Horningsea, about 2 km east of Site 2, which includes a transmissivity of 69 m^2/d calculated by the Cooper-Jacob method (Kruseman, G.P. and de Ridder, N.A., 2000). The information from BGS indicates that there was a moderate level of confidence in the value derived by the analysis. The thickness of the aquifer is not, however, available for the site. Assuming a thickness of 8 m, as at Sunclose Farm, this would give an average hydraulic conductivity/ permeability value of about 8.6 m/d for the aquifer at this location, equivalent to 1.0 x 10^{-4} m/s.

On the basis of location, it is considered that the transmissivity and permeability derived from the data for Sunclose Farm could provide a better representation of aquifer properties in the vicinity of Sites 1 and 2 than the value for permeability used in the Cam Bedford Ouse groundwater model (AMEC Environmental & Instrastructure UK Ltd, 2015).

3.2.1.3 Surface water features

As indicated in Section 2.1, Sites 1 and 2 are located at about 10 mAOD, with maximum elevations of 12 mAOD indicated on Ordnance Survey (OS) 1:25000 scale mapping in the area. The River Cam is located 2 to 3 km to the east of the sites. Downstream of the A14 crossing at Milton, the elevation of the River Cam is below 5 mAOD.

One kilometre to the south west of Site 2 is a lake which is bound on its southern edge by the A14. There are two water bodies between Sites 1 and 2, one is a pond located at Green Gates Farm and the other is a reservoir at Sunclose Farm. Another reservoir is located between Site 1 and the A10 at Rectory Farm. All water bodies can be seen on Figure A.2.

There are several drainage ditches running across and in close proximity to both proposed sites. However, taking into account the flat lying terrain, exact drainage connections and directions of drainage discharge are difficult to determine from the mapping. The area of Site 2 is likely to drain to the west, towards a watercourse in Histon which is a tributary of The Old West River and Cottenham Lode. Site 1 drains partly to the River Cam to the east and partly to tributaries which may discharge eventually to the Great Ouse to the north.

3.2.1.4 Surface water and groundwater interaction

Still water bodies in the vicinity of Sites 1 and 2 are located in areas of river terrace deposits underlain by Gault Formation. As indicated in Section 2.2.2, these water bodies have, presumably, resulted from, or been created by, excavation of river terrace deposits. They may be fed by groundwater which is perched in the superficial deposits above the Gault Formation, as well as direct rainfall and possibly some surface water flow.

The Lower Greensand aquifer is confined and separated from the superficial deposits by Gault Clay in the vicinity of Sites 1 and 2. As a result, there is no interaction between the aquifer and surface water features. There may, however, be interaction between the Lower Greensand aquifer and surface water at the Lower Greensand outcrop approximately 2 to 3 km to the north-west of the sites, where springs/seepages located on the aquifer could contribute to minor watercourses in the area.

3.2.2 Site area 3

3.2.2.1 Site specific hydrogeology

BH01 was completed to a total depth of 30.2 mbgl at Site 3 as part of the ground investigation in 2020. BH01 is located towards the north-western side of Site 3, as shown on Figure A.5.

The geological description in the borehole log for BH01 at Site 3 is summarised as follows:

- Topsoil and Superficial deposits (to 0.8 mbgl) – Brown slightly clayey or silty, gravelly fine to medium sand.
- West Melbury Marly Chalk Formation (to 10.9 mbgl) – Weak, low to medium density, off white chalk with infilled fractures. Areas of extremely weak rock throughout, although the geological log does not refer specifically to any marl being recovered in the core.
- Gault Formation (to base of borehole, completed at 30.2 mbgl) – Stiff fissured grey silty calcareous clay.

The Lower Greensand was not encountered in BH01 at Site 3. Based on BGS records, the closest borehole to Site 3 which encountered the Lower Greensand Formation was located approximately 1 km to the north of BH01. The BGS records indicate that the Lower Greensand was encountered at 44.8 mbgl at this location. However, as the bedrock in the area dips gently to the south east, the Lower Greensand is likely to be deeper at Site 3.

Cross sections taken from the Leapfrog model beneath Site 3 are shown in Figure A.11 and Figure A.12. Based on the modelling, the top of the Lower Greensand should be about 50 to 51 mbgl at Site 3.

3.2.2.2 Surface water network

The River Cam is located approximately 1 km to the west of Site 3. Quy Water is located about 600 m to the south east of Site 3.

There is a series of parallel drainage ditches which extend in an easterly direction from Low Fen Drove Way, shown on Figure A.2, and discharge to the Black Ditch. A drainage ditch which originates within Site 3 appears to connect into the Black Ditch to the north-east of Low Fen Drove Way. The Black Ditch drains in a northerly direction from the area to the south-east of Site 3 towards Stow Cum Quy Fen. The course of the ditch continues in a north-easterly direction along the boundary of Quy Fen before discharging into Bottisham Lode. The network of drainage ditches, including the Black Ditch, are managed by Swaffham Internal Drainage Board (IDB). The drains within the Swaffham IDB catchment area[2] gravitate to the IDB's pumping station site at Upware, where land drainage water is discharged to the River Cam[3].

[2] Ely Group of Internal Drainage Boards, Swaffham Internal Drainage Board District Map. Available at: http://www.elydrainageboards.co.uk/wp-content/uploads/2019/04/Swaffham.pdf

[3] Ely Group of Internal Drainage Boards, description of Swaffham Internal Drainage Board. Available at: http://www.elydrainageboards.co.uk/internal-drainage-boards/swaffham/

At the closest point, Stow Cum Quy Fen is situated about 1.1 km north east of the boundary of Site 3. As indicated in Section 2.6, Stow Cum Quy Fen is a Site of Special Scientific Interest (SSSI) designated for floristically rich calcareous loam pasture. It includes pools formed on the West Melbury Marly Chalk Formation, which support a range of aquatic plants including some uncommon species.

3.2.2.3 Groundwater levels and aquifer properties

Permeability testing of the whole of the Chalk saturated section, carried out by a rising head test method on BH01 in Site 3 (see Section 4.2 for further details), gave a value of 7.0×10^{-8} m/s, equivalent to 0.006 m/d (A F Howland Associates, 2020). Given the very low permeability indicated by testing at BH01, there is likely to be very little groundwater flow in the lowermost section of the Grey Chalk in the area. As indicated in Section 2.3, Site 3 is identified as being in an area with '*soluble rock risk*' in which '*solution features that enable rapid movement of a contaminant may be present*'. However, the very low permeability determined by testing indicates that no such features enabling rapid movement of a contaminant are likely to be present at the location of BH01.

The position of BH01 in Site 3 is shown on Figure A.5. BH01 is located at about 10.3 mAOD. The groundwater level in the Grey Chalk in BH01 was recorded at 5.7 m depth (approximately 4.6 mAOD) on 16th October 2020.The water level was higher in two measurements taken in November 2020, and was recorded at 3.9 m depth (approximately 6.4 mAOD) on 14th December 2020, indicating an overall rise of about 1.8 m in a period of two months. The rise in water level presumably resulted from autumn rainfall giving rise to aquifer recharge over this period. Aquifer recharge is likely to continue into late winter or spring 2021 and, therefore, the groundwater level in BH01 could continue to rise above the level recorded in December 2020.

Ordnance Survey mapping indicates that Site 3 spans the 10 mAOD contour on the east side of the River Cam. The topographic elevation reduces from west to east across much of the site, towards the set of drainage features connected to the Black Ditch. As the Chalk is at outcrop, any groundwater flow would be expected to follow topography, with groundwater levels at a shallower depth in lower parts of the site. It is likely, therefore, that most of the groundwater flow is towards the network of land drainage feeding into the Black Ditch. OS mapping indicates that the 5 mAOD contour is located in this area of land drains, with the elevation declining overall to the north.

Based on the topography, any groundwater in the Grey Chalk on the western-most side of the site (to the west of the location of BH01) would be expected to flow towards ditches which discharge to the River Cam.

There are no Environment Agency observation boreholes in close proximity to Site 3. However, groundwater levels in an Environment Agency observation borehole in the Grey Chalk, in an area several kilometres to the north east of the site, vary generally by about two metres over the period of record from 1969, with occasional higher groundwater levels in particularly wet winters. The observation borehole is at a slightly higher elevation than BH01, at about 15 mAOD, with the uppermost strata likely to comprise the Totternhoe Stone. Based on this data, groundwater levels might be expected to vary generally by up to two metres at Site 3. However, the observed variation of 1.8 m in BH01 in the latter part of 2020, described above, indicates that, in some areas of Site 3, the general seasonal variation in groundwater levels may be more than two metres.

Taking into account the elevation of the water table at BH01, and the variations in ground levels across Site 3, groundwater levels might, at times, be present close to ground level in lower areas of the site. Higher groundwater levels would be expected in the period from late autumn to spring, but would also be dependent on the hydrological conditions in any particular year.

3.2.2.4 Surface water and groundwater interaction

Given the groundwater level below site area 3, and assuming relatively flat hydraulic gradient which is consistent with the topography, indicates that groundwater in the Grey Chalk over much of the area of Site 3 is likely to be in hydraulic connectivity with surface water in the area of drainage ditches that feed into the Black Ditch. However, due to the very low permeability of the lowermost section of the Grey Chalk present in this area, it is considered that the baseflow contribution to the Black Ditch is also likely to be low.

There are a number of open waterbodies in the area, including Allicky Farm Pond CWS, and pools at Stow Cum Quy Fen SSSI that are formed partly or wholly on the Grey Chalk, and may therefore be in hydraulic continuity with groundwater. In the case of Allicky Farm Pond CWS, the site is also located across a ribbon of peat deposits which appears to have formed the original course of the Black Ditch. Hence this feature may be dependent on groundwater in the superficial deposits, or on a combination of groundwater in the superficial deposits and bedrock.

The hydrology and hydrogeology of the Stow Cum Quy Fen SSSI are discussed in detail below.

3.2.2.5 Stow Cum Quy Fen SSSI

Stow Cum Quy Fen SSSI is an important site in relation to the assessment of the potential impacts of a WWTP on site area 3. As noted in the Water Resources Statement (Mott MacDonald Ltd, Cambridge WWTP Relocation Project - Water Resources Statement, 2020b) there was some uncertainty as to whether there is any hydraulic connection between the area of Site 3 and the SSSI. Therefore, further investigation was undertaken to determine if any potential pathways for impact existed.

Further investigation included the following:

- Reconnaissance visits to the Fen on 7th and 31st December 2020 (photographs of the visit are provided in Appendix B) to assess the extent and approximate depth of ponds within Stow Cum Quy Fen, the connection of the ponds with the Black Ditch, and any indications that the ponds are groundwater fed;
- Discussion with the Quy Fen Trust (during a meeting with the Secretary and Chairman of the Trust on 10th December 2020) to clarify observations recorded during the first reconnaissance visit; and,
- Further desk top assessment to collate and assess the above findings.

A map of the Fen and all the features described in the following section is provided in Figure A.6.

The main water features of the SSSI comprise a number of ponds, some of which are formed in the Grey Chalk. The ponds were excavated during the 19th Century for the extraction of coprolite. There is also a more recent pond on the southern boundary of the site, which was constructed in 2017[4] The ponds vary in size and depth, the largest of which, an approximately

[4] Stow Cum Quy Fen pond survey, Fenland Botanical surveys, 2017, http://quyfen.uk/wp-content/uploads/Stow-Cum-Quy-Fen-pond-survey_July-2017.pdf

200 m long rectangular waterbody known as the Cut (Photo B.1 and Photo B.2), is located towards the centre of the Fen. Several smaller isolated ponds are also located around the Fen.

The western end of the Cut is connected to a ditch via a culvert (Photo B.3). This ditch (Photo B.4) extends to the west where it opens into a shallow pond, comprised of two areas separated by a raised footpath. During the reconnaissance visit, a small upwelling of water was evident in the western section of the pond close to the raised footpath. The upwelling may indicate there was some flow at the time from the eastern pond area under the footpath.

The western section of the pond extends to the bank of the Black Ditch (Photo B.5), and is connected to the Black Ditch via a culvert and one way valve (Photo B.6). The valve allows flow from the ponds into the Black Ditch during periods of high water levels in the Fen, but is closed to flow from the Black Ditch back into the ponds. However, it is understood that, during periods of particularly high flow in the Black Ditch, the water level can increase above the level of bank allowing overflow onto the Fen. This is understood to have occurred following a period of prolonged, heavy rainfall in December 2020. The occurrence of overflow from the Black Ditch was apparent during the reconnaissance visit on 31st December 2020 from the significant extent of standing water on the Fen, and the elevated water level in the Black Ditch (Photo B.8).

It is understood that there are no other surface water features connected to the Cut, or the adjacent ditch and pond areas between the Cut and the Black Ditch. The Cut is estimated to be approximately 2-3 m deep, which is likely to mean it is in hydraulic continuity with groundwater in the Grey Chalk. Marly chalk material is evident in parts of the banks of the Cut. Therefore, it seems likely that the Cut is mainly groundwater fed with, some contribution of run-off from the surrounding area.

There is also a pond on the western bank of the Black Ditch, which is located within the SSSI boundary but outside of the area controlled by the Quy Fen Trust. It is understood there is no control structure connecting this pond with the Black Ditch, but that the elevation of the bank is lower adjacent to this pond. Therefore, there should also be a connection between the Black Ditch and the pond during periods of high water levels, although it was not possible to confirm this during the reconnaissance visits.

The Black Ditch (Photo B.7) flows along the western boundary of the SSSI and into an online pond within the northern extent of the SSSI. From this point, the ditch, known as the Commissioners Drain, flows in a north easterly direction towards Bottisham Lode.

During the reconnaissance visit on 7th December 2020, flow in the Black Ditch was estimated visually and, hence, very approximately to be of the order of 50 l/s, just downstream of a track crossing to the south-west of Stow Cum Quy Fen.

There is a drainage ditch along the north-eastern boundary of the SSSI, which connects into the pond at the northern extent of the SSSI. This ditch includes some on-line ponds in relatively deep depressions, although at the time of visiting (7th December 2020) no water was evident in the ditch and ponds.

4 Dewatering for shaft construction

4.1 Introduction

Initial design considerations assume that the infrastructure for each site would include a circular shaft with an external diameter of 20 m to be excavated to a depth of about 39 m below ground level (mbgl). The shaft would accommodate the terminal pumping station for the waste water, received at the WWTP site via the transfer tunnel from the existing WWTP. The depth of excavation includes for a 12.7 m deep, mass fill, concrete base plug, located below the inlet from the tunnel and the pumping station. The plug is required to balance the potential impacts of flotation on the shaft, although the depth of the plug is likely to be reduced substantially once the design is progressed. The initial design also includes for a concrete collar with an external diameter of about 30 m, installed to a depth of 4 m around the top of the shaft, which provides additional dead weight as a further measure to prevent flotation.

Depending on the method of construction, dewatering will be required during excavation of the shaft. Preliminary calculations of the potential ranges of dewatering rates have been undertaken by geotechnical methods assuming the shaft is unlined throughout the period of excavation. The rates have then been used to assess the possible extent of drawdown resulting from this dewatering.

Potential methods of construction, for example with lining of the shaft as excavation progresses, have also been considered in relation to the drawdown impacts. The methods might vary between:

- Sites 1 and 2, where excavation is required through the Gault Formation and into the Lower Greensand aquifer; and,
- Site 3, where excavation is required through the lowermost section of the Grey Chalk and into the Gault Formation.

Presentation of the assessment of impacts of dewatering has been based on the HIA methodology outlined by the Environment Agency (Environment Agency, 2007). The Agency's methodology has been adapted to include the following topics:

- Groundwater features and potential impacts;
- Surface water features and potential impacts;
- Mitigation methods; and,
- Residual impacts and monitoring recommendations.

The guidance indicates that the details to be included are not prescriptive. The detail provided may be changed in order to match different situations.

The following additional shafts have also been considered in the assessment:

- Two shafts required at the existing Cambridge WWTP to intercept the existing tunnel that conveys waste water into the WWTP;
- If pipe jacking is used as the method of construction of the tunnel transferring waste water from the existing Cambridge WWTP to a new WWTP site at Site 3, three intermediate shafts would also be required along the route of the tunnel;
- A final pumping station shaft at each of the sites, connecting to the effluent discharge pipeline or tunnel; and

- If the effluent discharge is transferred to the River Cam via a tunnel, a pumping station shaft would also be required near to the river.

None of these additional shafts would be deep enough to encounter, or have an impact on, the Lower Greensand aquifer during construction. They would all be completed in the Gault Formation. However, the shaft connecting to the effluent discharge pipeline at Site 3 would be excavated through the lowermost section of the Grey Chalk and into the Gault Formation. In addition, two of the intermediate shafts required for pipe jacking along the route of the tunnel to Site 3 would also be excavated through the lowermost section of the Grey Chalk. These three additional shafts are therefore discussed with the shaft constructed to accommodate the terminal pumping station for the waste water at Site 3.

4.2 Assessment of dewatering rates

4.2.1 Introduction

Preliminary geotechnical investigations were undertaken at the three potential WWTW sites in 2020 and a draft factual investigation report submitted (A F Howland Associates, 2020). The investigation boreholes at the sites confirmed the stratigraphy and provided some data relating to groundwater and permeability. One borehole was constructed at each site. For the investigation, the boreholes at Sites 1, 2 and 3 were numbered BH03, BH02 and BH01 respectively (note that this is in reverse order to the site numbering).

Rising head permeability tests were carried out in BH02 and BH03 during drilling, and also in the standpipes in BH01, BH02 and BH03 during monitoring visits.

The de-watering calculations rely primarily on an understanding of three essential aspects:

- Stratigraphical succession;
- Groundwater levels (piezometric levels); and,
- Permeability of the relevant strata.

The recent investigations provided some limited information on permeability values. This information has been supplemented by other published data and some previous local project experience based on Mott MacDonald records.

4.2.2 Permeability of formations

Grey Chalk

The West Melbury Marly Chalk Formation in the lower part of the Grey Chalk sub-group consists generally of a ‘muddy’ or marly chalk often lacking in structure. It has a distinctive light grey (rather than white) colour.

The lack of structure in the West Melbury Marly Chalk Formation, and its generally higher clay or silt content relative to other Chalk formations, means it has a lower permeability. As indicated in Section 3.2, the recent investigations included one rising head permeability test in the West Melbury Marly Chalk Formation giving a permeability of 7.0×10^{-8} m/s. Mott MacDonald also has experience of engineering works in this formation at sites in Cambridge, including various ground investigations spanning the period 2006 to 2016. From these previous investigations, we have noted five permeability test results in the West Melbury Marly Chalk Formation, giving permeability values between 9.8×10^{-9} and 3.1×10^{-6} m/s. This range spans the value obtained from BH01 at Site 3.

Gault Formation

The Gault Formation is a stiff to very stiff, fissured, bluish grey, silty Clay with rare shell fragments. Fissures are horizontal and vertical, often closely spaced, planar and smooth. Clay materials are generally considered to have very low permeability. However, fissured clays can have non-negligible permeability due to fissure flow.

The recent investigations did not include any permeability tests in the Gault Formation. However, a paper by Butcher and Lord (Butcher A.P. & Lord, J.A., 1993) brings together a substantial amount of data from construction sites around Cambridge. Using some of the graphed data from this paper, based on laboratory test samples, it was possible to back-calculate permeability values for the Gault Formation. The permeability values derived in this way were consistent, varying between 1.1 x 10^{-8} m/s and 4.5 x 10^{-8} m/s, with an average of 3.0 x 10^{-8} m/s. However, it would be expected that field values, which would be fully affected by the presence of fissures, could be somewhat higher. A range of values has therefore been adopted in the analysis to include the possibility of higher permeability in the Gault Formation.

Lower Greensand

The Lower Greensand is typically a very weak, weakly cemented, greenish grey, silty, fine to coarse grained Sandstone with close to medium spaced horizontal fractures, sometimes with some interbeds of firm to stiff, silty, sandy Clay. The Lower Greensand is designated as a Principal aquifer, although the permeability of the formation is variable and can be quite low.

In-situ tests were undertaken during the site investigations in 2020. Analysis of results from the tests indicates possible permeability values of 1.9 x 10^{-7} m/s and 1.6 x 10^{-6} m/s for the Lower Greensand aquifer. BGS also publishes some general information on the permeability of the Lower Greensand (British Geological Survey, 2020b), quoting a wide range of values from 1.2 x 10^{-9} m/s to 1.2 x 10^{-4} m/s, with median value of 6.1 x 10^{-6} m/s.

As indicated in Section 3.1, analysis of test pumping data for a borehole in the Lower Greensand at Sunclose Farm on Butt Lane, between Sites 1 and 2, indicated an average permeability of 1.6 m/d, equivalent to 1.9 x 10^{-5} m/s.

Summary of permeabilities

A summary of the permeability values used in the de-watering analysis is given in Table 4.1. The values used take account of the range of permeability data available but also, in some cases, take a cautious view of the likely permeabilities (i.e. the values selected may be on the high side based on the data available). For example, for the Grey Chalk, all values for permeability, including the lower bound (1.0 x 10^{-7} m/s), are higher than the value obtained for the permeability from the test on BH01 at Site 3 (7.0 x 10^{-8} m/s). The *more likely to be representative* value (1.0 x 10^{-6} m/s) is more than an order of magnitude higher than the value obtained from data for BH01, while the upper bound (1.0 x 10^{-5} m/s) is more than two orders of magnitude higher.

Table 4.1: Permeability values used in analysis

Stratum	Potential or likely values					
	Lower Bound		More likely to be representative, or derived from local data		Upper Bound	
	m/s	m/d	m/s	m/d	m/s	m/d
Grey Chalk	1.0×10^{-7}	8.6×10^{-3}	1.0×10^{-6}	0.09	1.0×10^{-5}	0.9
Gault Clay	1.0×10^{-8}	8.6×10^{-4}	5.0×10^{-8}	4.3×10^{-3}	1.0×10^{-7}	8.6×10^{-3}
Lower Greensand	1.0×10^{-7}	8.6×10^{-3}	6.1×10^{-6} / 1.9×10^{-5}	0.5 / 1.6	5.0×10^{-5}	4.3

4.2.3 Method of assessment

The method of assessment has been taken from standard equations presented in CIRIA report 113 (Somerville, S.H., 1986) together with the updated version in CIRIA Report C750 (Preene, Roberts, & Powrie, 2016). Dewatering rates have been calculated using analysis of full penetration by a single well for:

- an unconfined aquifer for the Grey Chalk at Site 3, and the Gault Formation at Sites 1 and 2; and,
- a confined aquifer for the Gault Formation below the Chalk at Site 3, and the Lower Greensand below the Gault Formation at Sites 1 and 2.

These calculations employ the following equations:

For the unconfined aquifer:

$$Q = \pi k \, (H^2 - h_w^2) \, / \, \ln(R_0/r_e)$$

(Equation 6.8 in CIRIA C750), where:

- k is the permeability
- H is the initial piezometric level in the aquifer
- h_w is the piezometric level or water level in the equivalent well (see discussion below)
- R_0 is the radius of influence, and
- r_e is the 'equivalent well radius', as discussed below.

For the confined aquifer:

$$Q = 2\pi k D \, (H - h_w) \, / \, \ln(R_0/r_e)$$

(Equation 6.7 in CIRIA C750), where:

- D is the thickness of the confined aquifer, and
- other parameter definitions are as above.

The Radius of Influence, R_0, was calculated from:

$$R_0 = C \, (H - h_w) \, k^{½}$$

(Equation 6.9 in CIRIA C750), where:

- C is an empirical calibration factor; and
- other parameter definitions are as above.

The empirical calibration factor, C is usually taken as 3000. However, as the radius of influence, R_0, is a component of a logarithmic function in the above equations, the outcome is not especially sensitive to this parameter. In certain cases, for example for a large diameter 'equivalent well' in a low permeability stratum, R_0 could theoretically be less than r_e. In this case, the above equations cannot be applied as $\ln[R_0/r_e]$ becomes negative. In such circumstances, however, it is common practice to take an equivalent radius of influence, R_0^*, as being the sum of R_0 and r_e. This implies that the drawdown starts from the ring of dewatering wells, not from the centre of the shaft, which is a reasonable assumption for the calculation. Furthermore, the calculated dewatering volumes are not especially sensitive to this assumption.

In all cases, initial groundwater pressures or water levels have been assumed to be at ground level. Groundwater level monitoring data from the recent investigations suggest this is a conservative assumption, as the recorded depths to equilibrium water levels were typically between 2 m and 5 mbgl.

In addition, it was necessary to calculate an 'equivalent well radius' assuming that a number of deep well points may need to be used for dewatering in a ring around each shaft. The 'equivalent well radius' was calculated using the method in CIRIA C750. This advises that the equivalent well radius of a de-watering system, comprising several wells or well points arranged in a circle, can be taken as the overall radius of the well dewatering system.

The following assumptions are also made in undertaking the calculations of dewatering discharges:

- Each shaft is formed to its deepest extent, assuming excavation to the maximum depth of the base is required to prevent uplift;
- Steady state conditions are reached;
- The strata analysed are laterally extensive and horizontally oriented;
- The strata are isotropic, i.e. permeability is the same in all directions;
- An equivalent radius of influence for flow calculation, is assessed for a large well with a radius equal to the 'equivalent well radius' as discussed above;
- The strata give up storage instantaneously with no drop in head; and,
- The whole of the shaft is part of the "well" i.e. water can flow into the shaft at any point, and the calculations ignore any lining during construction.

4.2.4 Results of the dewatering assessment

The total dewatering requirements calculated for shafts at the three sites are presented in Table 4.2. The requirements were calculated separately using lower bound and upper bound permeabilities for the formations, as well as best indicator values based on permeabilities which are more likely to be representative of the site, as detailed in Table 4.1.

Applying the best indicator dewatering values for Sites 1 and 2, more than 95% of the inflow is calculated to occur from the Lower Greensand, with the remainder from the overlying Gault Formation. For Site 3, about 60% of the inflow occurs from the Grey Chalk, with the remainder, about 40%, from the Gault Formation below the Grey Chalk.

Table 4.2: Summary of range of inflow estimates

	Site 1		Site 2		Site 3	
Estimate	l/s	m3/d	l/s	m3/d	l/s	m3/d
L kely Lower Bound	0.2	20	0.2	20	0.2	20
Best indicator	4 to 10	350 to 900	4 to 10	350 to 900	0.8	70
L kely Upper Bound	23	2000	23	2000	3	250

Note: Variations in equivalent figures reflect rounding.

As indicated in the table, there is a substantial difference between the best indicator and likely upper bound dewatering quantities for the Lower Greensand at Sites 1 and 2 and the Grey Chalk at Site 3. In addition, if the permeability of the Lower Greensand applied in the Cam Bedford Ouse model in the area of Sites 1 and 2, as discussed in Section 3.2.1.2, is used, the estimate of open shaft dewatering increases to more than 60 l/s (greater than 5 000 m^3/d). However, there is no evidence from borehole testing at the sites, or at Sunclose Farm between the sites, to indicate that this permeability value is applicable to the area of the two sites.

4.2.5 Assessment of drawdown

Approximate assessments of the potential drawdown, as a result of shaft dewatering, at nature conservation sites, some surface water features and licensed and unlicensed protected groundwater sources were undertaken for Sites 1, 2 and 3. The assessments were made using the Theis equation (Kruseman, G.P. and de Ridder, N.A., 2000), together with estimated aquifer parameters for the Lower Greensand and the Grey Chalk.

In all cases it was assumed that, in theory, the shaft would be excavated and remain completely unlined over a period of a few months. A period of four months was used for calculating drawdown. In practice, however, the construction methods would not involve the shaft remaining unlined throughout the period of excavation, as discussed below in Section 4.3.

At Sites 1 and 2, excavation would be expected to take place in the Lower Greensand over a shorter period than allowed for in the calculation of drawdown. However, dewatering of the aquifer would also be required at least part of the time during excavation in the overlying Gault Formation to reduce the upward pressure on the base of the excavation. Therefore, a period of four months of dewatering has also been assumed in the calculations for the Lower Greensand at Sites 1 and 2.

Analysis using the Theis equation is appropriate for assessing the impacts of abstraction/ dewatering at a point source on the surrounding aquifer. It is generally applied in assessing the impact of pumping from a relatively small diameter borehole and not a shaft with a diameter of 20 m. Nonetheless, as the distance to the locations at which impacts are considered are at least one or two orders of magnitude greater than the diameter of the shaft, use of the Theis equation should give some indication of the order of magnitude of the impact.

Lower Greensand at Sites 1 and 2

The following parameters were used in the assessment of dewatering of the Lower Greensand:

- The best indicator dewatering rates of 4 and 10 l/s, indicated in Table 4.2, ignoring the minor component of less than 5% contribution from the Gault Formation.
- Transmissivities (4.5 and 13 m^2/d) related to permeabilities used in deriving the respective dewatering rates, calculated as follows:
  - The transmissivity of 4.5 m^2/d was based on the permeability value of 6.1 x 10^{-6} m/s considered *more likely to be representative* in the dewatering analysis. An aquifer

thickness of 8.5 m was also assumed, the average thickness for the Lower Greensand for BH1 (7.7 m) and BH2 (9.3 m), in calculating the transmissivity.
  - The transmissivity of 13 m^2/d was the value obtained by the Logan approximation applied to the test pumping summary data for the borehole at Sunclose Farm.
- A confined aquifer storage coefficient value is also needed for the Theis equation calculations. A value of 1×10^{-4} was applied to the whole of the Lower Greensand in the Cam Bedford Ouse groundwater model (AMEC Environmental & Instrastructure UK Ltd, 2015). This value was, therefore, used in the calculations.
- In order to assess the sensitivity of the results to the storage coefficient, a second value, 5×10^{-4}, was also used in the assessment. This value was chosen based on an analysis of storage coefficient values by BGS for the Woburn Sands formation in the Bedford/ Cambridge area (British Geological Survey, 1997). Analysis of data for 19 sites indicated that 5.4×10^{-4} was the 25 percentile value for the distribution of storage coefficient values, with the lower values in the range expected to represent confined aquifer conditions.

The potential impacts of dewatering from an open excavation for the shaft in the Lower Greensand were assessed at the following locations:

- The location given on the BGS website for the borehole at Sunclose Farm, about 350 m and 450 m from the nearest points on Sites 1 and 2 respectively.
- Other groundwater abstraction licences and private groundwater abstractions within about 5 km of Sites 1 and 2.
- The outcrop of the Lower Greensand about 2 to 3 km to the north-west of Sites 1 and 2.
- Cottenham Moat county wildlife site (CWS) located close to the Lower Greensand outcrop about 4 km from Site 1.

In addition, the maximum extent of impact in the Lower Greensand aquifer was calculated. Applying the best indicator dewatering rates of 4 and 10 l/s, together with the corresponding transmissivities, gave an approximate theoretical radius of drawdown around the shaft site of about 11 km and 18 km respectively. Increasing the storage coefficient from 1×10^{-4} to 5×10^{-4} reduced the radius of drawdown to about 5 km and 8 km. This indicates clearly the potentially extensive impact of dewatering and the sensitivity of the calculations to the storage coefficient which is chosen.

Borehole at Sunclose Farm

Use of the Theis equation indicates that dewatering of an unlined shaft excavation at Sites 1 or 2 over a period of four months could, in theory, produce a drawdown of 25 to 30 m in the borehole at Sunclose Farm, sufficient to dewater the borehole entirely. In practice, complete dewatering is unlikely to occur as, if the water level was drawn down into the Lower Greensand aquifer, the dewatering would start to draw on storage in the aquifer itself. However, the indications are that the impact would be severe. It would probably make the borehole unusable during the period of dewatering and subsequent recovery of the groundwater level.

The actual location of the shaft is more likely to be within the indicative footprint of the WWTP site, shown on Figure A.5. This would be at a distance of 750 m to 800 m from the location of the Sunclose Farm borehole. As a result, the drawdown impact is calculated to be in the range 18 to 22 m. However, as the existing rest water level at the time of testing of the borehole was about 6 m below datum (presumably just less than 6 mbgl), the effect of dewatering could still be to draw the groundwater level down into the aquifer. The geological log for the borehole indicates that the Lower Greensand is located between 24.5 and 32.5 mbgl at Sunclose Farm.

Adopting an increased value for storage coefficient of 5 x 10^{-4} reduces the drawdown at Sunclose Farm to between 9 and 13 m with dewatering occurring within the footprint of the site infrastructure. Nonetheless, the impact would still be substantial. As testing at Sunclose Farm indicated a drawdown to a depth of about 18 mbgl, the additional drawdown due to the dewatering at Site 1 or 2 could lower the pumping water level to within the depth interval of the aquifer.

The conclusion from this assessment is that dewatering of an open shaft excavation at Sites 1 or 2 is likely to make the Sunclose Farm borehole unusable during the period of dewatering and subsequent recovery. A database of abstraction licences in the study area (Environment Agency, 2020c) includes no licence details for the borehole site. However, there is a groundwater licence for Sunclose Farm, for spray irrigation, which is given at a grid reference for a reservoir in relative proximity to the borehole location. The licence at the reservoir site may therefore relate to the borehole.

Other groundwater abstractions

A substantial number of additional groundwater abstractions, within a distance of up to about 5km from Sites 1 and 2, have been identified as dependent, or potentially dependent, on the Lower Greensand aquifer as the source of supply. Dependence on the Lower Greensand aquifer has been established, where possible, from geological logs held by BGS. The sources include the following groundwater abstractions listed either in the water abstractions database for licensed and deregulated abstractions (Environment Agency, 2020c) or in local council environmental health officers records for private abstractions:

- Three licensed groundwater abstraction sources in the Impington and Girton area, to the west of Site 2, at a distance of about 2 to 5 km from Sites 1 and 2;
- Up to three unlicensed groundwater abstractions within Cambridge, at a distance of 500 m to 3 km from Sites 1 and 2;
- Up to six unlicensed groundwater abstractions north of Histon, at a distance of about 2 to 5 km from Sites 1 and 2;
- Up to eight unlicensed groundwater abstractions in and south east of Cottenham, at a distance of about 2 to 5 km from Sites 1 and 2;
- Up to six unlicensed groundwater abstractions along or to the east of the River Cam, between the A14 and Clayhithe, at a distance of about 2 to 4 km from Sites 1 and 2; and,
- At least 10 unlicensed groundwater abstractions in the area in and around Landbeach and Waterbeach, at a distance of about 1.2 to 4 km from Sites 1 and 2.

Depending on which combination of dewatering rate and transmissivity is applied, the calculated impacts of dewatering from an open shaft over a period of four months (120 days) at these additional groundwater abstractions ranges from:

- More than 20 m additional drawdown at the minimum distance of 500 m from the sites; and,
- About 1 to 4 m additional drawdown at the maximum distance of 5 km.

The calculations assume a storage coefficient of 1 x 10^{-4}. If a storage coefficient of 5 x 10^{-4} is used in the calculations, the estimated drawdown reduces to between about 14 and 18 m at 500 m, and 0.25 m or less at 5 km.

The conclusion from this assessment is that dewatering of an open shaft excavation at Sites 1 or 2 would probably have a substantial temporary impact on abstraction at a number of additional groundwater sources dependent on the Lower Greensand in the area. Some sources could become temporarily unusable. Groundwater sources at a distance of more than 5 km from the sites might also be affected. For example, Environment Agency and council records indicate there may be several unlicensed water supplies using groundwater from the Lower Greensand, located to the north and east of Waterbeach at a distance of up to 7.5 km from Site 1.

Lower Greensand outcrop

Calculations using the Theis equation, together with estimated aquifer parameters, indicate that, in theory, the impact of dewatering might be to reduce the groundwater level close to the outcrop area by several metres over a period of four months. The calculated impact is substantial because the aquifer is confined below the Gault Formation between Sites 1 and 2 and the outcrop. As a result, piezometric changes are transmitted rapidly through the aquifer. Adopting an increased value for storage coefficient of 5×10^{-4} reduces the theoretical drawdown considerably. Nonetheless, the calculated impact could still give rise to a significant impact.

Dewatering of an unlined shaft excavation could therefore, in theory, have a significant temporary impact on any spring flows and groundwater seepages in the outcrop area, together with any local watercourses which are dependent on groundwater for baseflow. Some springs and watercourses are indicated in OS mapping on or close to the outcrop around Oakington.

Lower Greensand outcrop near Cottenham Moat

Cottenham Moat CWS is located at least 4 km from Sites 1 and 2. The impact of dewatering might also be to lower the groundwater level by several metres in the Lower Greensand close to the outcrop area in the vicinity of Cottenham Moat. This theoretical impact reduces to less than a metre, and possibly to a few centimetres depending on the dewatering rate, assuming a storage coefficient of 5×10^{-4}.

Cottenham Moat is shown by BGS to be located on Kimmeridge Clay. However, the moat is adjacent to the contact with the Lower Greensand outcrop which overlies the Kimmeridge Clay. It is possible, therefore, that a connection might exist between the moat and the Lower Greensand aquifer. Therefore, potential impacts of temporary dewatering in the Lower Greensand aquifer during shaft excavation need to be considered at Cottenham Moat. The site supports a population of great crested newts.

Grey Chalk at Site 3

The validity of applying the Theis equation to determine drawdown for the unconfined Grey Chalk aquifer at Site 3 is questionable. The equation could not be used to predict accurately short term drawdown impacts as a result of dewatering in unconfined conditions. However, as the assessment is made for pumping over a period of up to four months, at substantial distances from the shaft, the calculations should give an approximate indication of the potential maximum extent of drawdown due to dewatering.

The following dewatering rates were used in the calculations:

- a best indicator dewatering rate of 0.45 l/s for the Grey Chalk, comprising about 60% of the total inflow of 0.76 l/s (rounded to 0.8 l/s in Table 4.2) from the Grey Chalk and the Gault Formation, as discussed in Section 4.2.4; and
- an approximate upper bound rate of 2.5 l/s, comprising the component for the Grey Chalk from the total upper bound inflow of 3 l/s in Table 4.2. The remaining component of the total inflow is estimated to result from the dewatering of the Gault Formation.

The transmissivities relating to these dewatering rates (0.45 and 4.5 m^2/d respectively) were determined as follows:

- The transmissivity of 0.45 m^2/d was based on the permeability value of 1.0 x 10^{-6} m/s considered *more likely to be representative* in the dewatering analysis. An aquifer thickness of 5.2 m was also assumed, the difference between the base of the Grey Chalk at BH01 (10.9 m depth) and the water level in BH01 at the time of permeability testing in October 2020 (5.7 m depth).
- The transmissivity of 4.5 m^2/d was based on the upper bound permeability value of 1.0 x 10^{-5} m/s and the same calculated aquifer thickness.

A specific yield (unconfined storage coefficient) of 2% was assumed for the Grey Chalk, at the lower end of the range of values used in the Cam Bedford Ouse groundwater model (2 to 3%).

The potential impact of dewatering from an open excavation for the shaft in the Grey Chalk was estimated to extend a theoretical distance of about 270 m from the shaft at the best indicator rate of dewatering (0.45 l/s). Hence, if the best indicator rate was applicable, the temporary drawdown due to open shaft dewatering in the Grey Chalk would be unlikely to extend very much, if at all, beyond the limits of Site 3. The theoretical extent of calculated drawdown increased to about 860 m at the upper bound rate (2.5 l/s).

Allicky Farm Pond CWS is located about 580 m from Site 3 at the closest point. In theory therefore, applying the upper bound rate for dewatering, drawdown resulting from dewatering of an open excavation in the shaft would extend beyond Allicky Farm Pond. However, the extent of drawdown in the direction of Allicky Farm Pond would also depend on the actual location of the shaft within Site 3. The calculated drawdown would not extend as far as the Quy Fen SSSI which is about 1.1 km from Site 3 at the closest point.

If a reduced specific yield value of 1.5% is used for the calculations, together with the upper bound dewatering rate, the theoretical extent of drawdown increases further to just under 1 km from the shaft excavation. However, this value for specific yield is lower than the range of 2 to 3% used in the Cam Bedford Ouse groundwater model. Increasing the specific yield to 3% reduces the theoretical extent of drawdown to about 700 m.

4.3 Construction of shafts

4.3.1 Construction options

Two options have been considered for the construction of the shaft to accommodate the terminal pumping station for the waste water received at the WWTP site (the WWTP intake shaft):

Use of secant piles

- A ring of 1.2 m diameter piles drilled and installed to, or a few metres below, the required design depth for the shaft. The piles would be installed at adjacent locations around the perimeter of the shaft to form, in theory, a continuous barrier to groundwater flow. The shaft would then be excavated once the piles were in place. Following excavation and installation of the shaft base, concrete would be cast in-situ to line the shaft within the secant piles.
- Positional accuracy does decrease with depth of piling and in varying ground conditions. In the worst cases the piles may diverge with depth leaving a gap and, theoretically, a small degree of leakage is possible. However, modern construction techniques are likely to be quite reliable and, if significant leakage is encountered, this could be rectified by remedial techniques such as localised grouting.

Underpinning

- Following the construction of the concrete collar to a depth of about 4 m, the shaft would then be excavated in sections. Precast 0.5 m thick concrete lining segments would be installed each time a 1 m depth of shaft had been excavated. Grout would be emplaced behind each ring of concrete segments before shaft excavation continued. The process of excavation, lining and grouting would be repeated over 1 m depth intervals to the full depth of the shaft.

Secant piles would take a period of about two months to install. The shaft excavation would then take place over several weeks before installation of the concrete reinforced base.

Underpinning would be expected to progress with a metre depth of excavation occurring every two to three days, therefore taking a total of three to four months to complete. Underpinning would be followed by installation of the concrete reinforced base.

In addition to these options, the shallower, additional shafts referred to in Section 4.1 might be constructed by a caisson method. For this option, the shaft lining would first be lowered into the top of the initial shallow excavation. As excavation continued, further lining sections would be added at the surface. The lower part of the caisson would gradually sink under the weight of the sections added at the surface as the excavation proceeded. This is likely to be the preferred method of shaft construction if:

- pipe jacking is undertaken along the route of the tunnel to Site 3 and intermediate shafts are required; and,
- the effluent discharge is transferred to the River Cam via a tunnel and a pumping station shaft is required near the river.

As the caisson lining sinks gradually following the excavation, the formation in the wall of the excavation would be sealed off from the shaft. Therefore, dewatering would only be required in the shaft for the material actually being excavated within the lining, together with any seepages at the joints between caisson sections and upwelling through the base of the excavation when excavation is taking place in the superficial deposits and Grey Chalk. No further assessment is therefore included for the impact of dewatering for shafts constructed by the caisson method.

4.3.2 Requirements for dewatering

The requirements for dewatering with the proposed alternative construction methods for the WWTP intake shaft are described separately for Sites 1 and 2 and for Site 3. The potential impacts of dewatering on groundwater and surface water features, taking into account these methods, are then evaluated in the following sections 4.4 and 4.5.

Sites 1 and 2

The impacts on groundwater in the Lower Greensand aquifer would be dependent on which method of construction was chosen. The methods are, therefore, discussed separately.

If secant piles were used, the piles would be installed initially to full depth. Based on a total shaft depth of 39 m, the piles would be installed several metres into the Kimmeridge Clay underlying the Lower Greensand at both sites. The geological log for BH02 at Site 2 indicates that the upper boundary of the Kimmeridge Clay was at 31.3 m depth; in BH03 at Site 1, the upper boundary was at 34.5 m.

The use of secant piles would mean that dewatering would only be required in the shaft for the material actually being excavated within the piles, together with any minor seepages between the piles. As the piles would be installed into the Kimmeridge Clay underlying the Lower

Greensand, there would be no opportunity for upward flow to occur into the excavation within the Lower Greensand. The small amount of dewatering required could be carried out using sump pumps in the excavation, with the pumps being lowered as the excavation progressed.

If the thickness of the base and, hence, the total depth of the shaft were reduced, the base of the excavation might be located within the Lower Greensand rather than the Kimmeridge Clay. If the secant piles were also installed just to the base of the excavation within the Lower Greensand, then some dewatering of the aquifer would be needed as follows:

- Dewatering could be required when excavation was taking place in the lower part of the Gault Formation overlying the Lower Greensand. Without dewatering, the pressure of the confined groundwater in the Lower Greensand might cause a failure by uplift or heave in the base of the excavation, as excavation progressed towards the base of the Gault Formation.
- There would also be potential for upward leakage through the base of the excavation in the Lower Greensand.

Overall, the total amount of dewatering required should be less than the total amount required if the excavation was unlined. However, the requirement to provide any substantial dewatering from the Lower Greensand could complicate the excavation as a dewatering system would need to be installed and then operated during excavation.

If the minimum thickness for the base slab is assumed to be 5 m, the minimum total depth of the shaft would be reduced to just over 31 m. The base of the shaft would then be about 3 m above the base of the Lower Greensand at Site 1, and at, or very close to, the base of the Lower Greensand at Site 2. It seems quite likely, therefore, that the simplest option for shaft construction could be to continue with secant piling into the top of the Kimmeridge Clay, thus avoiding the need for any substantial dewatering of the Lower Greensand during shaft construction.

If the shaft was constructed by underpinning, dewatering could also be required in the lower part of the Gault Formation to prevent uplift or heave in the base of excavation. Below the Gault Formation, exposure of the base of the excavation in the Lower Greensand during underpinning would lead to the potential for upward leakage of groundwater through the base. In addition, up to one metre depth of excavation in the Lower Greensand would be exposed over a period of a few days prior to each ring of concrete lining being installed and grouted. Overall, the total volume of dewatering required should be less than the dewatering requirement indicated by the calculations for the unlined shaft excavation. However, assuming that the dewatering was undertaken by pumping from a ring of well points surrounding the excavation, the quantities are still likely to comprise a substantial portion of the dewatering for a totally unlined shaft. This is because low water levels/pressures would still need to be maintained in the surrounding aquifer to keep the excavation dry.

Alternative methods of shaft excavation, such as use of localised jet grouting in the Lower Greensand to prevent or significantly reduce the need for dewatering in the aquifer, may be possible but have not been considered in detail at this stage.

Site 3

Both methods of construction would reduce substantially the requirements for dewatering when compared to excavating an unlined shaft. The use of secant piles would mean that dewatering would only be required for the material actually being excavated within the piles, together with any minor seepages between the piles. As the piles would be installed a substantial depth into the Gault Formation underlying the Grey Chalk, there would be no opportunity for upward flow to occur into the excavation within the Chalk.

The small amount of dewatering required would be carried out using sump pumps in the excavation, with the pumps being lowered as the excavation progressed.

Exposure of the base of the excavation in the Grey Chalk during underpinning would lead to the potential for upward leakage through the Chalk base, up to the time at which the underlying Gault Formation was encountered. In addition, up to one metre depth of excavation in the Chalk would be exposed over a period of a few days prior to each ring of concrete lining being installed and grouted. Overall, however, the total volume of dewatering required would be a small proportion, perhaps of the order of 10 to 25%, of the dewatering requirement indicated by the calculations for the unlined shaft excavation. This assessment is based on reasonable hydrogeological and geotechnical judgement.

The Lower Greensand aquifer underlying the Gault Formation should not be encountered during excavation of the WWTP intake shaft at Site 3. However, a concern for excavation of the shaft is that the upward pressure of the confined groundwater in the Lower Greensand might cause a failure by uplift or heave in the base of the excavation at depth in the Gault Formation. As indicated by the section in Figure A.11, the depth to the Lower Greensand is expected to range from about 50 to 51 mbgl at Site 3. If the shaft is excavated to a depth of 39 mbgl, then the base of the excavation would be approximately 11 to 12 m above the top of the Lower Greensand. However, as indicated in Section 4.1, the thickness of the plug in the base of the excavation, assigned a nominal, initial depth of 12.7 m, is likely to be reduced substantially once the design is progressed. Hence the total depth of the excavation could be reduced significantly.

The precise engineering methods which may need to be applied to mitigate for the possibility of a failure in the base of the excavation will be determined as part of further design. One of several potential solutions might, in theory, be to undertake some dewatering in the Lower Greensand during excavation to relieve the pressure on the base of the excavation. However, from a hydrogeological viewpoint, if other solutions are available then these would be far preferable to dewatering of the Lower Greensand. In addition, it is not a solution which is likely to be considered in the design. This hydrogeological impact assessment has therefore been prepared assuming that dewatering of the Lower Greensand would not be undertaken during excavation for the shaft at Site 3.

As indicated in Section 4.1, the shaft connecting to the effluent discharge pipeline at Site 3 would also be excavated through the lowermost section of the Grey Chalk and into the Gault Formation. Assuming construction of the shaft was undertaken either by secant piling or underpinning, then the minor amount of dewatering required should be similar to the amount of dewatering required in the Grey Chalk during construction of the WWTP intake shaft. As the shaft is shallower than the WWTP intake shaft, there could be no potential impact on the Lower Greensand aquifer.

As also indicated in Section 4.1, two of the intermediate shafts required if pipe jacking is undertaken along the route of the tunnel to Site 3, would also be excavated through the lowermost section of the Grey Chalk. However, as indicated in Section 4.3.1, these shafts would be constructed by the caisson method. As a result, the amount of dewatering required should be minimal.

4.4 Potential groundwater impacts

4.4.1 Introduction

The potential groundwater impacts during excavation and dewatering of the shaft include the following:

- Temporary depletion of groundwater resources or impacts on licensed or unlicensed groundwater abstractions as a result of dewatering.
- Temporary impacts on groundwater dependent nature conservation sites.

The potential impacts of dewatering on groundwater features, taking into account the construction methods, are evaluated separately for the Lower Greensand at Sites 1 and 2, and the Grey Chalk at Site 3. The construction methods of secant piling and underpinning are also discussed separately for each aquifer.

4.4.2 Lower Greensand at Sites 1 and 2

Use of secant piles

For this hydrogeological impact assessment, it is assumed that, whatever the total depth of the shaft excavation, secant piles, if used, would be driven into the upper part of the Kimmeridge Clay underlying the Lower Greensand when constructing a shaft at Sites 1 or 2. As a result, dewatering would only be required for:

- The material actually being excavated within the piles; and,
- Any minor seepages between the piles.

As a result, the impact on groundwater in the Lower Greensand aquifer, and any abstractions, nature conservation sites or surface water features dependent on or supported by groundwater would be negligible.

Underpinning

The calculations of the impacts of dewatering of an unlined shaft provide an indication of the potential drawdown at various locations, as indicated in Section 4.2. They do not provide an accurate assessment. In addition, it is not possible to provide an accurate assessment of the impact of underpinning on the dewatering quantities and therefore, again, only an indication of the potential drawdown is possible.

The potential impact of dewatering during construction of the shaft by underpinning was assessed assuming the following possible conditions:

- The rate of dewatering applicable for an open excavation has to be maintained for a maximum continuous period of 30 days during underpinning; and,
- The average rate of dewatering over a four month period during underpinning is 50% of the rate of dewatering for an open excavation.

Both conditions still indicate a severe impact at the borehole at Sunclose Farm, with potential drawdown of the order of 10 to 15 m. Taking into account the drawdown during testing of the borehole, the additional drawdown due to dewatering might be sufficient to cause localised, partial dewatering of the aquifer during abstraction at Sunclose Farm. Hence, the conclusion from this assessment is that dewatering during underpinning at Sites 1 or 2 is likely to make the Sunclose Farm borehole unusable during the period of dewatering.

The impacts during construction by underpinning at other groundwater abstractions within 5 km of Sites 1 and 2 would be less severe than for construction involving the open excavation of the shaft. However, the drawdown impact could still range between about 10 and 20 m for any groundwater sources at a distance of 500 m from either site, and 0 to 2 m for groundwater sources 5 km from the site. This could therefore have a substantial impact on the yield of some groundwater abstraction sources; sources located closer to Site 1 or 2 could be temporarily unusable.

The calculations also indicate that the impact of dewatering during underpinning might cause a reduction of a few metres in the groundwater level close to the outcrop area. Again, adopting an increased value for storage coefficient of 5×10^{-4} reduces the drawdown substantially, possibly to less than a metre. Nonetheless, the calculated impact could have a significant temporary impact on any spring flows or groundwater seepages to local watercourses in the outcrop area.

The impact of dewatering might still be to lower the groundwater level in the Lower Greensand close to the outcrop area in the vicinity of Cottenham Moat by a few metres. This reduces to somewhere between a few centimetres and tens of centimetres assuming a storage coefficient of 5×10^{-4}.

4.4.3 Grey Chalk at Site 3

Use of secant piles

As already indicated, the use of secant piles in constructing a shaft at Site 3 would mean that dewatering would only be required for the material actually being excavated within the piles, together with any minor seepages between the piles. As a result, the impact on groundwater in the Grey Chalk aquifer, and any abstractions and nature conservation sites dependent on or supported by groundwater, would be negligible.

Underpinning

As indicated in Section 4.3, the total volume of dewatering required would be a small proportion, perhaps of the order of 10 to 25%, of the dewatering requirement indicated by the calculations for the unlined shaft excavation. The radius of influence of shaft dewatering should be substantially reduced from the distances of 270 m and 860 m indicated in Section 4.2.5, and may not extend beyond the site boundary. For example, the theoretical extent of drawdown would reduce to about 135 m and 425 m respectively assuming that:

- continuous dewatering would be required at rates which are 20% of the best indicator and upper bound rates; and
- the dewatering would take place over a period of 30 days during which excavation and underpinning was taking place in the Grey Chalk. Once underpinning was completed to the top of the Gault Formation, the pathway for direct inflow from the Chalk to the excavation should be cut off.

As a result, the theoretical drawdown for either the best indicator or upper bound dewatering rates does not extend as far as the Allicky Pond Farm CWS. In addition, the impact on any groundwater abstractions dependent on Chalk groundwater, should be negligible. This would include a negligible impact on a private groundwater source identified from council records at a location to the east of the Site 3 boundary, in the event that this abstraction is dependent on the Grey Chalk for supply.

Potential impacts on nature conservation sites

Taking into account these assessments for both methods of construction, there should be no impact on groundwater in the vicinity of either the Quy Fen SSSI or the Allicky Pond CWS. Both these conservation sites would be located a substantial distance outside the area of influence of any shaft dewatering at Site 3.

4.5 Potential surface water impacts

4.5.1 Introduction

The potential impacts during construction include the following:

- Temporary impacts on flows and water levels in surface water features which are dependent on groundwater resources.
- Impacts on surface water quality as a result of discharge from shaft dewatering.

4.5.2 Sites 1 and 2

As indicated in Section 4.4.2, if secant piles were used in the construction of the WWTP intake shaft at Site 1 or Site 2, the impact on groundwater in the Lower Greensand aquifer would be negligible. As a result, the impact on any surface water features dependent on or supported by groundwater, would also be negligible. This would include:

- Springs and seepages near the outcrop of the Lower Greensand;
- Surface watercourses on or close to the outcrop, supported by spring discharges and seepages; and,
- Any groundwater supply to Cottenham Moat and, hence, the water level in the moat.

If underpinning was used as the method of construction of the shaft there could be a significant temporary impact on any spring flows or groundwater seepages to local watercourses in the outcrop area and, hence, the flows in the watercourses. The impact of dewatering might also be to reduce the groundwater level in the Lower Greensand close to the outcrop area in the vicinity of Cottenham Moat. If a connection exists between Cottenham Moat and the aquifer, the reduction in groundwater level could reduce the supply of groundwater to the moat and, as a result, reduce the water levels in the moat. However, any such connection has not been investigated at the site and is unproven.

Neither method of construction would have any impact on surface water in the vicinity of Sites 1 and 2. The Lower Greensand aquifer, from which the dewatering would occur, is separated from the surface by the Gault Formation. The Gault Formation acts as a confining layer for the aquifer. As a result, there is no significant hydraulic connection between the aquifer and surface water or shallow groundwater features in the vicinity of the sites. Therefore, there would be no impact on these surface water or shallow groundwater features due to the drawdown in the aquifer.

There can be a high dissolved iron content in the Lower Greensand groundwater in some parts of the aquifer in eastern England. When the groundwater discharge is in contact with air, for example during shaft dewatering, the iron in the groundwater is likely to oxidise to a less soluble (ferric) state and could precipitate out of solution. Precipitation of iron compounds can give rise to an orange discolouration of the discharge which may then affect the quality/appearance of a receiving surface water body. The discharge from the dewatering wells, or from dewatering within the shaft, may also contain some silt. If untreated, the silt could be deposited in ditches receiving the discharge.

4.5.3 Site 3

As with impacts on groundwater abstractions, the impact on flows and water levels in any surface water features, potentially dependent on or supported by Chalk groundwater and located down-gradient of Site 3, such as the Black Ditch, should be negligible.

The dewatering discharge from the shafts is likely to contain silt, with a milky or grey colouration from the Grey Chalk and possibly the Gault Formation. The silt and colouration of the discharge could be conveyed or deposited in field boundary ditches discharging to the main watercourse in the area, the Black Ditch.

4.6 Mitigation measures

4.6.1 Sites 1 and 2

If secant piles were used in the construction of the WWTP intake shaft at Site 1 or Site 2, the impact on groundwater, and surface water features dependent on or supported by groundwater, would be negligible. Hence, no mitigation would be required in relation to shaft dewatering.

If the shaft was constructed by underpinning, there could be a significant impact on the following groundwater and surface water features:

- Potentially a minimum of 36 licensed and unlicensed groundwater abstractions located within about 5 km of the sites, and possibly additional groundwater abstractions at a greater distance;
- Springs near the outcrop of the Lower Greensand;
- Surface watercourses on or close to the outcrop, including any surface watercourses supported by spring discharges; and,
- Any groundwater supply to Cottenham Moat from the Lower Greensand and, hence, the water level in the moat.

A water features survey would be undertaken to identify all groundwater and surface water features which might be affected by the dewatering. This could include a survey of all licensed and unlicensed groundwater sources in the Lower Greensand within a minimum 5 km radius of the proposed site.

Mitigation measures would be described in detail in the Environmental Statement. However, the following points provide an indication of the practicalities of mitigation if construction of the shaft was undertaken by underpinning:

- The potential impacts of dewatering on licensed and unlicensed abstraction boreholes in the Lower Greensand would need to be assessed by test pumping of one or more boreholes at the proposed shaft site during further ground investigations. If the potential impacts of dewatering indicate that operation of abstraction boreholes may not be possible at the licensed rate (or the maximum abstraction rate required by the licensee, if less than the licensed rate), or at the current established rate for an unlicensed source, then actions will be taken to ensure the required supply can be maintained. Such actions could include:
  - lowering the pump in the borehole, or combining this action with provision of a replacement pump, taking into account the higher head requirements;
  - providing an alternative water supply for the duration of dewatering and the period of subsequent groundwater level recovery; or,
  - possibly constructing a replacement borehole to allow for increased drawdown. However, this is unlikely to be a preferred option taking into account the relatively short duration of dewatering. In addition, some existing boreholes may already extend through the Lower Greensand aquifer, for example at Sunclose Farm. Therefore, any temporary increase in drawdown could not be compensated for by deepening the borehole.
- Providing mitigation for loss or reduction of spring discharges and flows in watercourses in the Lower Greensand outcrop is unlikely to be practicable in the event that test pumping

indicates that dewatering could affect these features. The impact would, however, be temporary, for up to four months during dewatering, plus a subsequent period of aquifer recovery.

- Mitigation, if practicable, may need to be considered in the event that a significant temporary reduction in water levels is predicted in the Cottenham Moat CWS as a result of dewatering in the Lower Greensand.

Discharge during shaft dewatering

Discharge of groundwater from the Lower Greensand aquifer during shaft dewatering would involve careful planning and management to avoid the impacts of unacceptable discolouration of any surface water body, due to the possible precipitation of iron in the form of ferric compounds. The management of discharge would be based on an assessment of groundwater quality data obtained during test pumping of boreholes at Sites 1 or 2 as part of further site investigations. Depending on the groundwater quality, mitigation might include treatment prior to discharge from the site. A discharge consent would be required for this operation which would specify any treatment required.

It will be important to establish accurately the overall chemical condition of the groundwater at the point of discharge during test pumping. Accurate analyses, which are representative of the groundwater at source, will also be required for dissolved iron and any other determinands, for example manganese, which may precipitate and produce discolouration on contact with air. In addition, permission should be requested to carry out on-site analyses and take samples of the discharge from the borehole at Sunclose Farm. The on-site results, and analyses of the samples, could provide useful additional information regarding potential variations in Lower Greensand water quality in the area.

The Environment Agency has suggested that any groundwater removed during temporary dewatering for shaft construction, particularly from shafts in the Lower Greensand, should be recharged to the aquifer. However, this would be a complex undertaking, potentially requiring pressurised re-injection to the Lower Greensand aquifer. The hydro-chemistry of the groundwater might also change as a result of pumping and, potentially, exposure to the atmosphere. As with discharge to surface water, depending on overall groundwater conditions and the concentrations of iron or other metals, precipitation might occur between abstraction and reinjection.

In addition, reinjection might not provide the support or maintain precisely the established long term groundwater conditions at existing abstractions during temporary dewatering. Furthermore, it is unlikely to be practicable to reinject the groundwater at suitable locations to ensure no temporary impact on any groundwater seepages at the Lower Greensand outcrop.

It should also be stressed that the effects of dewatering in the Lower Greensand aquifer would be temporary only. The aquifer should return to the established long term condition in reasonable time, following the completion of dewatering.

Mitigation measures for removing silt from the discharge are described for Site 3 in Section 4.6.2, to be included in the code of construction practice (CoCP). These measures could also be applied at Sites 1 and 2 for silt removal.

4.6.2 Site 3

The impact of dewatering during shaft construction using secant piling or underpinning should be negligible for:

- Chalk groundwater and any abstractions from the Cha k; and,

- flows and water levels in any surface water features potentially dependent on or supported by Chalk groundwater and located down-gradient of Site 3.

As a result, no mitigation should be required in relation to groundwater and surface water features as a result of shaft dewatering. However, a water features survey would be undertaken in an area extending at least 1 km from Site 3 and monitoring considered for some features. This could include the unlicensed groundwater source shown in council records as located to the east of the Site 3 boundary, in the event that this abstraction is dependent on the Grey Chalk for supply.

Discharge during shaft dewatering

As indicated in Section 4.5, the dewatering discharge from the shafts is likely to contain silt, with a milky or grey coloration from the Grey Chalk and possibly the Gault Formation. If untreated, the silt and colouration of the discharge could be conveyed or deposited in field boundary ditches discharging to the main watercourse in the area, the Black Ditch. This impact will, however, be avoided by application of mitigation measures included in the code of construction practice (CoCP). The measures are likely to include retention of the dewatering discharge behind semi-permeable barriers in the upper reaches of ditches inside the site boundary. These temporary structures would allow the sediment to settle or filter out before discharge to the ditch system down-gradient.

As the dewatering quantities with either construction method should be very small, and the dewatering would take place over a limited period of, at most, a few months from the Grey Chalk, there appears to be little, if any, value in attempting to recycle this groundwater back to the aquifer. The disturbance to ground conditions required to implement a short-term infiltration scheme is likely to outweigh any value obtained from such a scheme.

4.7 Residual impacts and monitoring strategy

4.7.1 Sites 1 and 2

There would only be negligible impacts if the WWTP intake shaft was constructed using secant piling. Hence there would be no requirement for mitigation and no residual impacts.

There would be the following residual impacts in the event that shaft construction was undertaken by underpinning:

- Potential loss or reduction of spring discharges, seepages and flows in watercourses in the Lower Greensand outcrop. The impact would, however, be temporary, for up to four months during dewatering, plus a subsequent period of aquifer recovery.
- Reduction in water level in the Cottenham Moat CWS, in the event that a reduction in water level actually occurs as a result of a connection with the Lower Greensand aquifer, and mitigation is not practicable.

As previously indicated, however, the impact would be temporary, for up to four months during dewatering, plus a subsequent period of aquifer recovery.

As already indicated, test pumping would need to be carried out if the proposed method of shaft construction was by underpinning. Test pumping would involve the construction of test boreholes at the chosen site, and possibly one or more observation boreholes in the area. In addition, monitoring would be undertaken at existing licensed and unlicensed groundwater sources in the Lower Greensand and, possibly, at some springs and surface water features. The testing programme, and requirements for monitoring as part of this programme, would be agreed with the Environment Agency.

Monitoring of groundwater levels and surface water levels and flows would be undertaken prior to and during dewatering. The monitoring would be carried out at groundwater abstraction sources, and in spring and surface water features identified in the water features survey which could potentially be affected by dewatering. The programme of monitoring for the period of dewatering and subsequent recovery would be agreed with the Environment Agency following:

- the water features survey; and
- test pumping, monitoring and analysis during ground investigations.

4.7.2 Site 3

There would be negligible impacts on groundwater and surface water features if the shafts were constructed using either secant piling or underpinning. Hence, there would be no requirement for mitigation and no residual impacts.

Borehole construction with test pumping would be needed at the sites for the shafts, particularly if the proposed method of shaft construction was by underpinning. The testing and analysis of results would be used to confirm the aquifer properties of the Grey Chalk at the proposed locations. Test pumping would involve the construction of at least one test borehole and possibly one or more observation boreholes in the area near each shaft location. In addition, monitoring would be undertaken at any existing unlicensed groundwater sources in the West Melbury Marly Chalk and overlying superficial deposits and at some surface water features. This could include:

- groundwater levels for the unlicensed groundwater source shown in council records as located to the east of the Site 3 boundary, in the event that this abstraction is dependent on the Grey Chalk for supply; and,
- water levels at Quy Fen SSSI and Allicky Farm pond CWS.

However, if as expected, the yield from boreholes is very low, no impact is anticipated at any off-site features. The testing programme, and requirements for monitoring as part of this programme, would be agreed with the Environment Agency.

Monitoring of groundwater levels and surface water levels and flows would be undertaken prior to and during dewatering. The monitoring would be carried out at groundwater abstraction sources and surface water features identified in the water features survey which could potentially be affected by dewatering. The programme of monitoring for the period of dewatering and subsequent recovery would be agreed with the Environment Agency following:

- the water features survey; and
- test pumping, monitoring and analysis during ground investigations.

5 Assessment of other potential impacts

5.1 Introduction

This section covers the assessment of impacts to groundwater resulting from the following scheme components:

- The tunnel transferring waste water from the existing Cambridge WWTP to the new WWTP;
- The pipeline transferring waste water from the existing Waterbeach WWTP to the new WWTP;
- A pipeline diversion to the waste water transfer network, intercepting waste water in an existing pipeline close to the A14 and diverting the flow to either Site 1 or Site 2;
- Discharge pipelines, or a tunnel, transferring the treated effluent from the new WWTP to the outfall on the River Cam; and
- Permanent foundations and below-ground structures at the new WWTP site.

The impacts are considered for schemes in which the new WWTP is located at Sites 1 or 2, or at Site 3. The assessment takes into account the hydrogeological conditions at, and along proposed tunnel and pipeline routes to, each of the three sites.

5.2 Waste water transfer tunnel

5.2.1 Construction method

A tunnel is included in the design of the scheme transferring waste water from the existing Cambridge WWTP to the new WWTP. A minimum finished internal diameter of 2.5 m has been assumed for the tunnel. However, to accommodate a secondary lining, the primary lining diameter would need to be a minimum of 3.0 m. The secondary lining would be required for tunnel sections passing through the Lower Greensand aquifer to:

- protect the aquifer from the risk of waste-water exfiltration; and,
- prevent significant groundwater infiltration to the tunnel.

All tunnelling by closed-face methods using a tunnel boring machine (TBM)[5] would involve the injection of fluids to maintain pressure on the formation and hold back any potential groundwater inflows. Tunnel lining follows immediately behind the closed-face excavation. Bentonite and various polymers are used as fluids, and would be of an acceptable quality standard for tunnelling works and agreed with the Environment Agency.

As indicated in Section 1.3, the construction of the waste water transfer tunnel to Site 3 might be completed using a pipe-jacking method. For pipe-jacking, intermediate shafts would be required at regular intervals of about 500 m along the tunnel route. Construction by pipe-jacking involves pushing a sleeve into position along the tunnel route, using hydraulic jacks, behind the TBM.

The wastewater tunnel depth is anticipated to be about 17.3 mbgl at the existing Cambridge WWTP, with a fall of about 1 in 800 over the tunnel length to any of the new sites. The geological model predicts that, in the case of Site 1, the tunnel would be located within the Gault Formation along its entire length between the existing Cambridge WWTP and the receiving WWTP. However, it is likely to pass very close to the top of the Lower Greensand in the vicinity

[5]This is consistent for both potential me hods of tunnelling i.e. bored segmental lining or pipe-jacking

of Site 1. The tunnel to Site 2 would pass approximately into the top 2 m of the Lower Greensand in the vicinity of the site.

In the case of Site 3, the geological model predicts that the tunnel would be located within the Gault Formation along its entire length between the existing Cambridge WWTP and the receiving WWTP.

The construction methods for all shafts associated with the waste water transfer tunnels are discussed in Section 4.3.

The route corridors for the waste water transfer tunnels are provided on Figure A.13.

5.2.2 Potential impacts

The potential impacts of tunnelling include the following:

- Tunnelling construction works resulting in turbidity in groundwater;
- Contamination with tunnelling fluids;
- Waste-water exfiltration or infiltration of groundwater in relation to the tunnel and all shafts; and,
- Impact on groundwater flows.

5.2.3 Mitigation methods

Tunnelling in the Lower Greensand would not be expected to lead to significant turbidity in the aquifer as a result of the absence of rapid fissure flow in the formation. The occurrence of fractures in some horizons within the Lower Greensand is indicated by the geological logs for the cored site investigation boreholes drilled at Sites 1 and 2 in 2020. However, permeability testing at the Sites, and records for testing at the Sunclose Farm borehole between Sites 1 and 2, indicates that the Lower Greensand has low permeability in this area. Hence the presence of some fracturing in the formation should not give rise to turbidity concerns in the aquifer during tunnelling.

It has been assumed that both a primary and secondary lining would be required for tunnel sections passing through the Lower Greensand. Given the low permeability of the Gault Formation, there is unlikely to be any significant exfiltration or infiltration for the sections of tunnel passing through the formation.

It has also been assumed that secondary linings would be required for all shafts that penetrate productive superficial deposits, the Lower Greensand and the Grey Chalk to mitigate for any potential exfiltration and infiltration.

The tunnel and shaft sections in the Lower Greensand would replace a minimal part of the overall aquifer. They would not be expected to have any significant impact on groundwater flow, with flow being maintained below any short section of tunnel in the Lower Greensand, and around the shaft section in the aquifer.

Use of tunnelling fluid will be kept to a minimum. Any fluids used will need to be approved by the Environment Agency prior to use.

5.2.4 Residual impacts

With implementation of the CoCP and the use of Environment Agency approved drilling fluids, it is unlikely that there will be any significant residual impacts. However, groundwater quality will be monitored in boreholes located close to the route of the tunnel where it passes through any bedrock aquifer. Borehole locations for monitoring, and the groundwater quality data to be collected, will be agreed with the Environment Agency.

5.3 Pipelines

5.3.1 Construction method

Pipeline from Waterbeach WWTP

A pipeline will be constructed to carry wastewater from the existing WWTP at Waterbeach to the new WWTP (pipeline corridors are shown in Figure A.15). The existing Waterbeach WWTP will be replaced with a pumping station. The construction method for this pipeline will be similar to that of the treated effluent pipeline discussed below, although with a smaller diameter pipe.

The pipe will be 350 mm diameter and, assuming 350 mm spacing either side as per typical trench design, it would result in a trench just over 1050 mm wide, also allowing for pipe wall thickness. HDPE pipework is likely to be used which would be welded above ground and installed in the trench in lengths of about 100 m. Each 100 m section of trench should be open for a maximum period of one week. Once the pipe is installed, the 100 m section of trench would be backfilled before the next 100m section is excavated. This process would then be repeated for the entire length of the pipeline.

The depth of cover for the pipeline would range from about 900 mm to 2.5 m. The trenches would penetrate superficial deposits and the uppermost section of the Gault Formation for the pipelines required at Sites 1 and 2. For Site 3, the route from Waterbeach would be through superficial deposits and Gault Formation. To the south of Clayhithe, however, BGS geological mapping indicates the presence of an isolated patch of West Melbury Marly Chalk Formation. West Melbury Marly Chalk Formation is also present between Horningsea and Site 3.

Where a crossing is required, the pipeline would be constructed beneath the river, road or railway. Most notably, the crossings will include the River Cam and the railway line to the east of the river for the route to Site 3 and several road crossings for routes to Sites 1 and 2. Construction of the crossings will involve pipe-jacking a 500 mm sleeve which is pushed into position (using hydraulic jacks) behind a shield in which excavation is taking place. Pipe jacking with five metre ground cover above the pipeline has been assumed for all crossings. A drill pit will be required on each side of the crossing.

All drill pits will pass through some superficial cover and penetrate into the Gault Formation more than 10 m above the Lower Greensand. Construction activities for the River Cam crossing to Site 3 would be undertaken either side of the river and away from the riverbanks.

Pipeline from the existing waste water transfer network

For Sites 1 and 2, a pipeline diversion is required to the existing waste water transfer network, intercepting waste water in an existing pipeline close to the A14 and diverting the flow to the new WWTP. The potential route corridors for the pipeline are shown on Figure A.1. The method of construction, and pipe and trench sizes, would be similar to the Waterbeach WWTP pipeline. The trenches would penetrate superficial deposits and the uppermost section of the Gault Formation (depending on the thickness of the superficial deposits and the site location). For Site 1, there would be one road crossing, on Butt Lane between Sites 1 and 2.

Treated effluent transfer pipeline

One of the options for the transfer of treated effluent from the new WWTP on all three sites to an outfall on the River Cam would be via a pipeline. The pipeline would run from the selected site area to an outfall structure constructed on the bank of the River Cam north of the A14.

Three 1500 mm pipes would be laid in an open cut trench for most of the route. The trench would be approximately 7 m wide and the excavations would be stepped[6], battered[7] or supported by pre-piled sheets installed prior to excavation. In order to achieve a gravity system for the pipeline, it is assumed that the depth of trench would be a maximum of 5 m.

Steel pipe is likely to be used for the treated effluent transfer pipeline. At the diameter required, the pipe would be supplied in sections with a maximum length of 18 m and lowered into the trench one section at a time to be welded in place. It is assumed that at any time:

- two pipe lengths could be exposed in the trench; and
- the maximum length of open trench would be 50 m.

As a worst case, it is also considered that each 50 m section of trench could be open for several weeks. Once the pipe is installed, the open section of trench would be backfilled before the next 50 m section is excavated. This process would then be repeated for the entire length of the pipeline.

For the route corridors from sites 1 and 2 to the north of Milton, as shown on Figure A.16, the pipelines would cross the A10, an area of allotments, Landbeach Road/High Street and the railway to the west of the River Cam. It is currently assumed that the route corridor to the south of Milton from Site 1 and 2 would be too constrained at the surface and, therefore, would not be considered for a pipeline option. For the route corridor from Site 3, the pipeline would cross Horningsea Road. These crossings are shown on Figure A.19.

It is assumed that the crossings would require pipe jacking of three 1800 mm sleeves, through which the three 1500 mm pipes would be laid. A drill pit will be required on each side of the crossing. It is assumed that these pits would be 20 m wide, 10 m long and 8 m deep. Each pit will be set back 10 m from the road (or railway) and have a concrete base slab with sheet piled sides.

There is likely to be a need for dewatering during the construction of the trenches and the drill pits. Estimates of the dewatering requirements for trench construction are provided in Section 5.3.2.

The trenches used for Sites 1 and 2 would penetrate varying depths of superficial deposits, typically between 1 and 5 metres. As a result of varying depths of superficial deposits in this area, the trench will pass into the top of the Gault Formation in some places along the route. Drill pits required for any of the crossings for Sites 1 and 2 will be excavated through superficial deposits and into the Gault Formation.

Geological modelling of the area indicates that, for Site 3, the eastern section of the trench would be in West Melbury Marly Chalk Formation as far as Horningsea Road. The West Melbury Marly Chalk thins to the west and, as a result, the trench should pass into Gault Formation in the area around Horningsea Road. The Chalk thins out completely in the area between Horningsea Road and Biggin Abbey Cottages. In this area, the trench is likely to be excavated in superficial deposits above the Gault Formation. The superficial deposits increase in thickness generally with increasing proximity to the River Cam. Borehole logs near the River Cam indicate a thickness of up to 8 m of superficial deposits close to the location of the proposed outfall structure. However, the logs indicate that the superficial deposits may be thicker on the western side of the river, on the opposite bank to the proposed outfall.

[6] In deep excavations the side of the trenches may be stepped at intervals in order to maintain the stability of the trench walls and reduce the risk of collapse

[7] Battering involves forming he side an excavation to an angle to reduce the risk of collapse

For the pipeline from Site 3, geological modelling near Horningsea Road indicates that the drill pits will pass through the West Melbury Marly Chalk and into the Gault Formation.

For all site areas and route options, the outfall structure on the River Cam would be constructed using a piled cofferdam to seal the working area from the river and any shallow groundwater. The working area would then be excavated and the structure installed using a combination of in-situ and precast concrete components. Some limited dewatering of the working area is likely be needed during excavation and installation of the structure.

5.3.2 Pipeline dewatering assessment

Dewatering rates have been calculated for indicative sections of both the treated effluent and Waterbeach transfer pipelines. The calculations provide an order of magnitude estimate of the potential rates of dewatering in a reasonable worst case scenario, without any mitigation in place. They should, therefore, be viewed as a very approximate indication of potential worst-case dewatering requirements.

5.3.2.1 Method of assessment

The method of assessment has been taken from standard equations presented in CIRIA report 750 (Preene, Roberts, & Powrie, 2016). Dewatering rates under steady state conditions have been calculated using analysis of full and partial penetration by a single row of well points, to represent trench sections in the following scenarios:

- unconfined aquifer in superficial deposits, representative of sections of the Waterbeach and treated effluent pipeline corridors for all three sites; and
- unconfined aquifer in the Grey Chalk, representative of sections of the Waterbeach and treated effluent pipeline corridors for site area 3.

In addition, it was necessary to calculate a 'distance of influence' for use in the calculation of the dewatering rates. This was done using the Sichardt equation with an empirical calibration factor of 1500. The calibration factor is considered to be appropriate for a row of well points, as discussed in Section 6.2.1 of CIRIA report 750 (Preene, Roberts, & Powrie, 2016). This analysis method is questionable when calculating lower discharge rates. However, the method should provide a reasonable order of magnitude estimate of the discharge rates for the scenarios described above.

The parameters for the scenarios are provided in Table 5.1 and are based on the construction methods described in Section 5.3.1. For these simplified scenarios it is assumed that ground elevation, water level and trench depth are constant along the length of the open trench.

The depth to the water table is set at ground level to provide a worst case for a single section of open trench in areas where the water table is likely to be high. These could include areas close to the River Cam and at the edge of the Grey Chalk outcrop to the west and north of Site 3. In reality, the depth to the water table will vary considerably along the pipeline corridors. It is likely to be below the base of the trench in some locations, for example in sections of the Grey Chalk to the north of site 3.

In addition, the thickness of the aquifer in the superficial deposits, assumed to be 5 m, is considered to represent a reasonable worst case given the variability of the thickness of the superficial deposits across the project area (see Section 2.2.2).

Dewatering would be carried out in each open section of trench, depending on the ground conditions encountered. Following pipe installation, the trench section would be backfilled and dewatering would cease. The next section of trench would then be excavated and dewatering recommenced (if necessary) in the new open trench section. This process would be repeated for the entire length of the pipeline.

Table 5.1: Dewatering scenario parameters

Parameters	Waterbeach transfer pipeline		Treated effluent transfer pipeline	
	Superficial deposits	Grey Chalk	Superficial deposits	Grey Chalk
Trench depth (m)	2.85		5.00	
Length of open trench (m)	100		50	
Thickness of aquifer (m)	5	5	5	5
Dewatering calculation used	Partial penetration	Partial penetration	Full penetration	Full penetration

At a later stage of the project a more detailed assessment of the superficial deposits associated with the pipeline corridors will be carried out following ground investigation for the selected site.

5.3.2.2 Permeability

The permeability of the Grey Chalk was established for the shaft dewatering assessment as discussed in Section 4.2.2.

A range of permeabilities for superficial deposits was estimated by comparing the composition of the superficial deposits recorded in BH04 and BH05 in the preliminary Ground Investigation (Particle Size Distributions provided in the factual report (A F Howland Associates, 2020)) with representative values of hydraulic conductivity for various unconsolidated sedimentary materials (Domenico, P.A. and F.W. Schwartz, 1990. Physical and Chemical Hydrogeology, John Wiley & Sons, New York, 824 p). The superficial deposits recorded in BH04 and BH05 (shown on Figure A.5) comprise river terrace and alluvial deposits respectively. They should therefore provide a reasonable representation of the superficial deposits which will be encountered during installation of the Waterbeach and treated effluent pipelines for all three sites.

The range of permeabilities selected for the superficial deposits and Grey Chalk are provided in Table 5.2.

Table 5.2: Permeability ranges

Stratum	Potential values					
	Lower Bound		Reasonable indicator		Upper Bound	
	m/s	m/d	m/s	m/d	m/s	m/d
Superficial deposits	2.0×10^{-5}	1.7	2.0×10^{-4}	17.3	5.0×10^{-4}	43.2
Grey Chalk	1.0×10^{-7}	8.6×10^{-3}	1.0×10^{-6}	0.09	1.0×10^{-5}	0.9

The reasoning behind the selection of the permeabilities is as follows:

- Lower bound of 2.0×10^{-5} m/s is at upper end of the range for silt and the lower end of the ranges for fine, medium and coarse sands.
- The reasonable indicator of 2.0×10^{-4} m/s is the maximum for fine sand and mid-range for medium and coarse sand.
- The upper bound of 5.0×10^{-4} m/s is the maximum for medium sand and at the lower end of the range for gravel.

- Permeability will tend to be limited by the finer materials present in the deposits. Therefore, these values are considered to be reasonable in deriving very approximately the dewatering requirements, based on the composition of the superficial deposits recorded in boreholes BH04 and BH05.

5.3.2.3 Estimated dewatering rates

Approximate estimates of the dewatering rates, using the range of parameters and permeabilities described above, are provided in Table 5.3.

Table 5.3: Estimated dewatering rates

			Dewatering discharge rate	
Pipeline	**Strata**	**Permeability**	**(m^3/d)**	**(l/s)**
Waterbeach transfer	Superficial deposits	Lower Bound	170	2
		Reasonable indicator	**520**	**6**
		Upper Bound	820	9
	Chalk	Lower Bound	10	0.1
		Reasonable indicator	**35**	**0.4**
		Upper Bound	120	1
Treated effluent transfer	Superficial deposits	Lower Bound	65	0.8
		Reasonable indicator	**210**	**2**
		Upper Bound	320	4
	Chalk	Lower Bound	5	0.1
		Reasonable indicator	**15**	**0.2**
		Upper Bound	45	0.5

Superficial deposits

The results show that there is considerable variation in dewatering rates for the superficial deposits depending on the permeability which is chosen. Dewatering quantities could be substantial in the reasonable indicator and upper bound scenarios for both the treated effluent and Waterbeach transfer pipelines. The rates are higher for the Waterbeach transfer pipeline than the treated effluent transfer pipeline due to the longer length of open trench.

It is considered that the reasonable indicator and upper bound scenarios are more likely to be representative of the pipeline corridors for sites 1 and 2, which all pass through areas of extensive river terrace deposits on the western flank of the River Cam valley. The lower bound may better represent the corridors for site area 3 as they pass through areas of alluvial deposits in proximity to the River Cam.

For the Waterbeach pipeline, it is considered that dewatering at the rates estimated for the reasonable indicator and upper bound scenarios is unlikely to be practical for pipeline installation in a 100 m section of relatively shallow trench. However, the rate of dewatering may be lower in reality and could be reduced further by shortening the length of open trench. Dewatering at the rate estimated for the lower bound scenario is not likely to present any significant practical issues.

For the treated effluent transfer pipeline, it is likely that dewatering at the range of estimated rates could be achievable, although potentially requiring more complex methods of dewatering.

With a large trench in unconsolidated material, however, it may be necessary to support the trench using pre-piled sheets installed prior to excavation. Use of pre-piled sheets would substantially reduce the inflow to the trench and, in turn, the amount of dewatering required.

Grey Chalk

The estimated dewatering rates for sections of the Waterbeach and treated effluent transfer pipelines in the Grey Chalk are all relatively low. It is considered that dewatering at all the estimated rates would be achievable with standard methods as described in Section 5.3.4. There are unlikely to be any significant impacts resulting from dewatering in the Grey Chalk during construction of the Waterbeach and treated effluent transfer pipelines.

The potential impacts of dewatering are discussed in the following section.

5.3.3 Potential impacts

The potential impacts during construction of the trenches, drill pits and pipeline crossings include the following:

- Pollution of groundwater, due to poor construction practices. This could include contamination by oils, fuels or chemicals used for site plant or in construction processes.
- Temporary depletion of groundwater resources or impacts on licensed or private groundwater abstractions as a result of dewatering undertaken during trench excavation and drill pit construction.
- Temporary impacts on surface water features or baseflows in watercourses which are dependent on groundwater resources, including potentially the River Cam County Wildlife Site, as a result of dewatering undertaken during trench excavation, drill pit construction and the river crossing on the pipeline route from Waterbeach to Site 3, although this short-term impact on river flow and river level should be negligible.
- Impacts on surface water quality, including water quality in the River Cam County Wildlife Site, as a result of discharge of silt-laden water from dewatering of pits or excavations.
- Changes to groundwater flow e.g. trenches acting as drains, pipeline interrupting flow.

As discussed in Section 5.3.2, temporary dewatering will be required for sections of trenches and drill pits located in superficial deposits, particularly where sands and gravels are encountered in river terrace or alluvial deposits. Dewatering of the superficial deposits could have a localised effect on water levels in the shallow aquifer and any surface water features in hydraulic continuity with the aquifer. However, any impacts would be temporary and no impact on any nature conservation site is anticipated, including Worts Meadow LNR as discussed in Section 2.6.3. Sections of pipeline trenches or drill pits penetrating the Gault Formation will require minimal dewatering due to the low hydraulic conductivity of the formation.

As discussed in Section 5.3.2 construction of sections of the treated effluent pipeline and Waterbeach transfer pipeline for Site 3 in the lowermost section of the Grey Chalk should require very limited dewatering, which would have only a localised impact. Groundwater yields or natural discharges from this section in the West Melbury Marly Chalk Formation are likely to be very low. The minor dewatering should have only a temporary impact on any seepages from the formation to overlying superficial deposits, or any groundwater abstraction, in the unlikely event that groundwater is abstracted from this lowermost section of the Grey Chalk.

As drill pits will be constructed away from the river bank, the construction of the River Cam crossing would not be expected to affect the shallow groundwater system that may be present within the superficial deposits in the river valley. Therefore, there should be no significant temporary adverse impact on the River Cam and associated CWS.

The outfall structures constructed at the River Cam for all sites could potentially have a localised impact on the flow of groundwater within the superficial deposits. However, the overall effect on the shallow groundwater system is not likely to be significant.

Gravel used in trench excavations as pipe bedding can act as a drain for shallow groundwater in any aquifers encountered along the pipeline route. The aquifers could include sands and gravels in river terrace or alluvial deposits, as well as the West Melbury Marly Chalk Formation along pipeline routes associated with Site 3. Drainage in the pipe bedding could have a permanent impact by affecting shallow groundwater levels and, possibly, reducing minor seepages from the West Melbury Marly Chalk Formation.

5.3.4 Mitigation methods

Appropriate standard mitigation methods to limit contamination or pollution of groundwater and surface watercourses will be taken into account in a detailed code of construction practice (CoCP) to be produced as a separate document and implemented on all construction sites, including sites for tunnels, pipelines and shafts. The CoCP will include measures to ensure that the sediment content of site runoff and dewatering from excavations is at an acceptably low level when discharged to watercourses.

Before construction starts, any existing land drainage will be modified to remove the risk of drainage flows discharging into the pipe trench both during and after construction. During construction, where gravel is used as pipe bedding and there is a risk of the trench acting as a land drain, impermeable partitions (clay stanks) will be installed at approximate 20-30 m spacings to prevent the transfer of water along the length of the trench. After construction is complete, the land drainage will be reinstated to operate as close to its previous condition as practicable.

The trench will be engineered to provide a low point where a pump will be used for dewatering. The drill pits for the pipe jacked crossings will also require dewatering. The discharge would usually be pumped to ground at a point sufficiently far away as not to impact the trench or drill pit. If the ground is too saturated to accept the dewatered flow, the dewatering discharge would be pumped to a watercourse with suitable water quality protection measures in place. Dewatering discharge would be subject to approval by the Environment Agency.

5.3.5 Residual impacts

There should be no significant residual, temporary or permanent impacts resulting from construction of the pipelines and associated trenches and drill pits providing:

- a CoCP is produced and adhered to during construction;
- land drainage is reinstated; and
- clay stanks are installed, as required, along pipe trenches in the superficial deposits, or in the West Melbury Marly Chalk Formation.

5.4 Treated effluent transfer tunnel

5.4.1 Construction method

A second option for the treated effluent transfer from the new WWTP at all three sites to an outfall on the River Cam would be via a tunnel.

The treated effluent tunnel would be constructed as described for the waste water transfer tunnel in Section 5.2.1, with the exception that it would be at a much shallow depth.

The depth of the treated effluent tunnel is anticipated to be about 8 mbgl at the new WWTP, with a fall of about 1 in 800 over the tunnel length to a shaft located in proximity to the outfall on the River Cam. The treated effluent would then be transferred from this shaft to the outfall on the river via a pipeline similar to that described in Section 5.3.

The geological model indicates that, in the case of Sites 1 and 2, the tunnel would be located within the Gault Formation along its entire length between the new WWTP and the shaft in proximity to the River Cam. In the case of Site 3, the geological model predicts that the tunnel would be located at the base of the Grey Chalk at the new WWTP. The tunnel would, however, be within the Gault Formation for much of its length, before transitioning into the superficial deposits near to the shaft in proximity to the River Cam.

The construction methods for all shafts associated with the treated effluent transfer tunnel are discussed in Section 4.3.

5.4.2 Potential impacts

The potential impacts during construction include the following:

- Tunnelling works resulting in turbidity in groundwater;
- Contamination with tunnelling fluids;
- Effluent exfiltration or infiltration of groundwater in relation to the tunnel and all shafts; and,
- Impact on groundwater flows.

Tunnelling in the superficial deposits would not be expected to lead to significant turbidity in the aquifer as a result of the predominance of intergranular flow in the sediments and the absence of rapid fissure flow. Tunnelling in the Grey Chalk should not give rise to significant turbidity due to the low permeability of the formation below Site 3.

The tunnel section for site 3 in the base of the Grey Chalk, and shafts in proximity to the River Cam for all three sites within the superficial deposits, could potentially have a localised impact on the flow of groundwater. However, the overall impacts on groundwater flow in the Grey Chalk and the shallow groundwater system in the superficial deposits are not likely to be significant.

5.4.3 Mitigation methods

It has been assumed that both a primary and secondary lining would be required for tunnel sections passing through the Grey Chalk or superficial deposits to mitigate for any potential exfiltration and infiltration. Although the Grey Chalk comprises a formation with low permeability, as confirmed by testing of BH01 at Site 3, a secondary lining would be included for tunnelling in the formation. Given the low permeability of the Gault Formation, there is unlikely to be any significant exfiltration or infiltration for the sections of tunnel passing through the formation.

It has also been assumed that secondary linings would be required for all shafts that penetrate productive superficial deposits and the Grey Chalk to mitigate for any potential exfiltration and infiltration.

Use of tunnelling fluid will be kept to a minimum. Any fluids used will need to be approved by the EA prior to use.

5.4.4 Residual impacts

With implementation of the CoCP and the use of EA approved drilling fluids, it is unlikely that there will be any significant residual impacts.

5.5 Foundations and below-ground structures

5.5.1 Construction methods

The depth of foundation required at each site is dependent on the height of the structure:

- The buildings/structures with a max height of 7 m will require shallow foundation of between 3-4 m.
- The buildings/structures with a height between 7-10 m will require deeper foundation (piles) to approximately 8-10 m.
- The buildings/structures higher than 10 m will require deep foundation (piles) to 10-25 m.

The initial concept design for the WWTP includes about 30 structures, more than half of which comprise tanks and other structures with heights less than or equal to 7 m.

The geological logs available from Sites 1 and 2 indicate that the top of the Lower Greensand was at 22.0 m and 26.8 mbgl at the borehole sites (BH03 and BH02) drilled during the site investigation in 2020. Only the most heavily loaded structures comprising the anaerobic digesters should have foundations, potentially to about 25 m, that might go deep enough to encounter the Lower Greensand, as shown in Figure A.17. At Site 3, the deepest foundations will pass through the Grey Chalk and into the underlying Gault Formation, to a depth about 25 m above the contact with the Lower Greensand, as shown in Figure A.18.

The deep foundations are most likely to be formed by continuous flight auger (CFA) piles. This process involves a hollow stemmed auger drilled into the ground below the site. When the design depth is reached, concrete is pumped through the hollow stem of the auger as the auger is gradually extracted. This method of piling limits the potential for cross-contamination between different strata and does not require the use of casing.

It is possible that some of the larger structures will have bases which are below ground level. Due to the potentially shallow groundwater table at the sites, there may be the need for minor dewatering during construction of any below ground components of the structures. The water level in the Grey Chalk at BH01 in Site 3 was measured at 5.7 m depth (approximately 4.6 mAOD) on 16th October 2020.The water level was higher in two measurements taken in November 2020, and was recorded at 3.9 m depth (approximately 6.4 mAOD) on 14th December 2020. BH01 was located close to the 10 mAOD contour which crosses the site. As the elevation of a substantial part of the site is below 10 mAOD, the groundwater level is likely to be closer to ground level in these lower areas.

As indicated in Section 3.2.2, depending on the time of year, and the hydrological conditions in any particular year, groundwater levels might be encountered close to ground level in lower areas of Site 3 during construction. However, regardless of these potential variations in groundwater conditions, if dewatering is required in the Grey Chalk at Site 3, the rates of dewatering should be substantially less than the rates for dewatering of an unlined shaft indicated in Section 4.

5.5.2 Potential impacts

The potential impacts of foundations include the following:

- Introducing vertical flow paths around piles;
- Blocking pre-existing horizontal groundwater flow paths in aquifers as a result of piling; and
- Altering the direction of shallow groundwater flow with the installation of below-ground structures.

For Sites 1 and 2, the presence of deep foundations may affect local flow paths within the upper part of the Lower Greensand. However, taking into account the overall areal extent of the Lower Greensand, the impact on the aquifer and groundwater flow would be negligible. Furthermore, the continuous flight auger method of piling is efficient at sealing any potential vertical flow paths created during piling.

For Site 3, all piles will be constructed in or through the Grey Chalk. Taking into account the expected low permeability of the West Melbury Marly Chalk Formation, the impact on the aquifer and groundwater flow should be negligible.

5.5.3 Mitigation methods

If any dewatering is required during construction, this would be pumped to ground at a point sufficiently far away from the excavation to prevent the immediate recycling of the groundwater. If the ground is too saturated or insufficiently permeable to accept the dewatered flow, the dewatering discharge would be pumped to a water course with suitable water quality protection measures in place. Dewatering discharge would be subject to approval by the Environment Agency.

As with all construction sites, the CoCP will be adhered to, ensuring construction activities are maintained at the site such that there will be negligible impacts to the water environment.

Some structures might be installed to a depth below the water table at Site 3. Drains would be installed to ensure that the groundwater flow is re-directed around the structure in the event that the structure could either:

- give rise to a permanent localised change in the direction of groundwater flow in the top of the Grey Chalk; or
- cause the groundwater to rise to the surface in the vicinity of the structure.

One option for the drains might comprise the installation of gravel-filled trenches around the structure.

5.5.4 Residual impacts

With implementation of the CoCP, the application of CFA methodology for piling, and installation of drainage measures, if required, there should be no residual impacts.

6 Contaminant transport assessment

6.1 Sites 1 and 2

Due to the presence of Milton Landfill, located on the east side of Site 2, concerns have been raised by the Environment Agency that construction activities could have an impact on mobilising contaminants originating from the landfill. The contaminants could then pollute groundwater away from the landfill, potentially including the dewatering discharge resulting from dewatering of the shafts. This would be of greater concern if contaminants are known to have leached out from the landfill into the surrounding Gault Formation, although the likelihood of significant migration within the Gault Formation is considered to be very low, owing to the unproductive nature of the formation. Additionally, the landfill incorporates leachate capture as part of the landfill management. The concern is principally in relation to Site 2, due to the proximity of the landfill to the proposed works, however a potential risk may also apply with temporary shaft dewatering for Site 1, which is located around 300 m from Milton Landfill.

The modelling therefore aims to understand the potential for migration from the landfill through the Gault Formation, and if dewatering would present an increased risk to new infrastructure (the tunnel and shafts), or to the Lower Greensand.

6.1.1 Background and methodology

A detailed assessment of the potential migration of contaminants from the landfill has been undertaken to better understand the likely risks associated with the works. To do this, ConSim[8] models were applied using its probabilistic functionality to simulate the fate and transport of dissolved contaminants in the subsurface, with the aim of estimating concentrations at defined compliance points.

6.1.1.1 Lateral migration

On the basis of the conceptual model (discussed in Section 3), a ConSim model was created with consideration of the following factors:

- A level 3a (independent groundwater transport) scenario was run, to simulate saturated transport of contaminants through the Gault Formation to a defined compliance point. This provides estimations of time for contaminants to reach the compliance point as well as relative concentrations that may be expected at the compliance point when considering the attenuating effects of degradation, retardation and dispersion (included as active processes). Note that the compliance point of 50 m was selected to represent the minimum distance to any construction but will also be protective of the point at which groundwater could be drawn down into the Lower Greensand as a result of dewatering for the proposed shafts.
- The 50 m stand-off zone from the landfill will be maintained during construction, which is the defined compliance point. The 50 m compliance point is a conservative limit for Site 2 and is therefore protective of receptors (the tunnel and shaft sites, and the Lower Greensand), even under conditions associated with dewatering for shaft construction. It should be noted that Site 2 is up-hydraulic gradient of the landfill under natural flow conditions and therefore although the closest distance to the landfill, it is highly unlikely that contaminants would ever move towards this site.

[8] Golder Associates. ConSim version 2.5

- Concentrations of determinands in local groundwater (both in the River Terrace Deposits and Lower Greensand) are known from available monitoring data provided by the landfill operator[9]. Determinands with exceedances against water quality standards (Environmental Quality Standards and Drinking Water Standards[10]), or considered to be highly mobile in the environment, have been modelled, including; cadmium, copper, mercury, nickel, potassium, zinc, ammoniacal nitrogen, nitrate, naphthalene and TPH fractions (aliphatics C5-C6, aliphatics C6-C8, aromatics EC5-7 and aromatics EC7-8).
- The model has been run for 1001 iterations to increase the confidence level (or percentile) in the results. Results focus on steady state conditions (assumed to be 10,000 years).
- Generic input concentrations for the determinands were all set to 0.001 mg/l to ensure contaminants did not reach saturation limits, and consequently skew the outputs. The model was set up to determine reasonable site specific target levels (SSTLs) above which any monitored concentrations shown to exceed the SSTL could be a potential issue. In this respect, the starting concentration is irrelevant as the SSTL calculation is based on the ratio between the initial and receptor concentrations (allowing for dilution, dispersion and retardation).

The following assumptions were also made:

- Although the majority of the landfill is believed to be lined, historical cells of the landfill are thought to be unlined, with waste placed directly into the excavation in the Gault Formation, however a thickness of >10 m of clay exists between the deepest excavation for the landfill and the upper boundary of the Lower Greensand. The tunnel is also to be located only within the Gault Formation in the immediate vicinity of the landfill, although the tunnel to Site 2 will transition into the Lower Greensand as it passes the north west corner of the landfill (which the 50 m compliance point is still protective of). The model has therefore assumed a single strata of clay through which migration of contaminants would occur between the 'source' and the compliance point.
- Groundwater flow in the Gault Formation is assumed to be very low, if connected water in the clay is present at all. For the model, and as an overconservative estimate, flow direction and gradient have been calculated from data collected during borehole monitoring of the Lower Greensand at Milton Landfill as this is likely to influence any flow within the Gault Formation. With dewatering of the shafts, it is assumed flow in the Lower Greensand will change, although it's effect on the flow regime in the Gault Formation within the 50 m distance from the landfill boundary is unlikely to be significant.
- The centre point of the model has been specified with a bias towards the older, unlined cells in the south-east of the landfill, as it is assumed that this is where the main mass of the contamination would be in direct contact with the Gault Formation. The compliance point has been set to be 50 m beyond the landfill boundary in the direction of groundwater flow from this point. Irrespective of this, the model output could equally apply to any 50 m point away from the landfill boundary assuming groundwater flow is always in the direction of the 50 m compliance boundary. To better define the results we have focussed the centre point and compliance point immediately down-gradient of this.
- Vertical migration of contaminants to the underlying Lower Greensand is unlikely as there is at least 10 m of Gault formation below the landfill and the landfill is lined in the newer sections / cells. The chemistry data for the River Terrace Deposits and the Lower Greensand

[9] FCC Environment, Milton Landfill groundwater quality monitoring results, June 2019

[10] Environmental Quality Standards (Water Framework Directive (Standards & Classification) Directions (England and Wales) 2015 Freshwater Annual Average) and UK Drinking Water Standards (The Water Supply (Water Quality) Regulations (England and Wales) 2018) have been used as the limiting criteria due to potential impacts to aquifers in the area as well as the proposed discharge of waters to the River Cam following treatment at the new works.

do not specifically demonstrate any contamination has migrated from the landfill site. Some analytes have been identified in both aquifers (NH_4 and SO_4 for example). However, their concentrations are low and are recorded across the monitoring area both up and down-hydraulic gradient. This suggests that the chemical data are more indicative of natural chemistry or catchment wide impacts from other sources.

Parameter values were assigned from site specific data and a variety of literature sources where site specific data were unavailable. A full list of the physical and chemical parameters used for the modelling are presented in Table 6.1 and Table 6.2 respectively.

Table 6.1: ConSim lateral model Site 1 and 2 physical input parameters

Parameter	Distribution	Value	Justification
Analysis level	**3a**		
Source			
Area (m^2)	Single	400,000	Area calculated from drawings of landfill cells
Aquifer – Gault Formation			
Thickness (m)	Uniform	10, 19.9	Assumed >10m thickness underlying landfill, increasing to 19.9m thickness Gault Formation at Site 2, estimated from Leapfrog model
Dry bulk density (g/cm^3)	Uniform	1.45, 1.58	BGS (1995) Engineering geology of British rocks and soils - Gault Clay. Page 23 (http://nora.nerc.ac.uk/id/eprint/20421/1/WN94031.pdf)
Fraction of organic carbon (%)	Uniform	0.048, 0.051	Environment Agency (2002) - Jurassic aquitards
Saturated hydraulic conductivity (m/s)	Uniform	1e-8, 1e-7	Professional judgement based on conditions encountered in borehole logs and knowledge of the area
Effective porosity (fraction)	Triangular	0.01, 0.06, 0.18	McWorter and Sunada (1977) Table 1: representative porosity values (Clay)
Hydraulic gradient (m/m)	Uniform	0.00125, 0.0016	Calculated from borehole monitoring of the Lower Greensand at Milton Landfill (winter and summer range)
Groundwater flow direction (º)	Single	50	Calculated from borehole monitoring of the Lower Greensand at Milton Landfill (winter and summer range)
Longitudinal dispersivity (m)	Uniform	5	10% distance of flow (50 m CP)
Lateral dispersivity (m)	Uniform	0.5	1% distance of flow (50 m CP)
Infiltration (mm/year)	Normal	300, 100	Normal distribution based around CIEH CHESS Met data for Cambridge (2012) (precipitation – evapotranspiration)
Receptor			
Compliance point (m)	Single	50	>50 m to tunnel or shaft sites

Table 6.2: ConSim model chemical input parameters

Contaminant	Koc/Kd (ml/g)	Henry's Law constant	Max solubility (mg/l)	Half life (years) (water)
Cadmium	100[1]	0	10000000	10000000
Copper	35[6]	0	10000000	10000000
Mercury	500[1]	0	10000000	10000000
Nickel	500[1]	0	10000000	10000000
Potassium	5.5[6]	0	10000000	10000000
Zinc	38[1]	0	10000000	10000000
Aliphatic C5-C6	794[2]	33[2]	36[2]	1.95[2]
Aliphatic C6-C8	3981[2]	50[2]	5.4[2]	1.95[2]
Aromatics EC5-7	79[2]	0.23[2]	1800[2]	2[3]
Aromatics EC7-8	251[2]	0.27[2]	520[2]	0.55[3]
Naphthalene	1288[4]	0.0174[4]	31[4]	0.71[3]
Ammoniacal nitrogen	0.65-6.5[5]	0.000658[6]	10000000	10000000 [5]

[1] Nathanail et al 2015: "The LQM / CIEH S4ULs for Human Health Risk Assessment ", Copyright Land Quality Management Limited reproduced with permission: Publication No. S4UL3389

[2] Total Petroleum Hydrocarbon Criteria Working Group Series (TPHCWG), 1999. Human Health Risk-Based Evaluation of Petroleum Release Sites: Implementing the Working Group Approach, Volume 5, Table 1.

[3] Howard et al. 1991. Environmental Degradation Rates. Max values.

[4] Environment Agency/Atkins, 2003. Review of the Fate and Transport of Selected Contaminants in the Soil Environment. Tables 2.4, 3.2 & 4.3.
[5] Buss et al., 2004. A Review of Ammonium Attenuation in Soil and Groundwater. QJEGH v37. Kd values chosen for clay. Half life is maximum for strata with mean pore size of >1um assuming anaerobic conditions
[6] RAIS database (Risk Assessment Information System, http://rais.ornl.gov/tools/)

6.1.2 Results

A summary of the modelling results are presented in Table 6.3, with full results provided in Table C.1 in Appendix C. An image of the modelled area is shown in Figure C.20.

Table 6.3: Site 1 and 2 ConSim lateral model results summary

Contaminant	SSTL (mg/l)	Unretarded travel time to CP (years)	Retarded travel time to CP (years)
Cadmium	N/A - No breakthrough occurred to receptor	2780	4.97e6
Copper	6.80e16		1.74e6
Mercury	N/A - No breakthrough occurred to receptor		3.75e7
Nickel	N/A - No breakthrough occurred to receptor		2.48e7
Potassium	1.97e12		2.76e5
Zinc	6.77e18		1.89e6
Aliphatic C5-C6	N/A - No breakthrough occurred to receptor		2.26e4
Aliphatic C6-C8	N/A - No breakthrough occurred to receptor		1.01e5
Aromatics EC5-7	N/A - No breakthrough occurred to receptor		4.65e3
Aromatics EC7-8	N/A - No breakthrough occurred to receptor		8.93e3
Naphthalene	N/A - No breakthrough occurred to receptor		3.46e4
Ammoniacal nitrogen	2.86e5		1.80e5

These results indicate that the migration of any groundwater within the Gault Formation is very limited, with travel times to the 50 m compliance point taking thousands of years, even when unretarded. Environment Agency guidance in the Remedial Targets Methodology[11] indicates that retarded travel times of over 1000 years are generally indicative of breakthrough that would not be considered significant, owing to other changes that could occur in the environment and the increased uncertainty of results above this time-scale.

For those determinands that reached the receptor within the given steady state timeframe (10,000 years), SSTL were derived that further support the idea of very limited migration within the Gault Formation, as all SSTL are extremely high (mostly above determinand solubility limits) and are several orders of magnitude higher than observed concentrations, such as NH_4 which has been monitored with a maximum concentration of 2 mg/l historically at one location.

The model is very conservative and protective of the defined compliance point (50 m boundary around the landfill) based on the input parameters used but predominantly as the closest works to the landfill will be at least 50 m from the landfill and for much of the construction it will be more. In addition, the closest construction works to the landfill boundary would be up-hydraulic gradient of the natural groundwater flow, assumed in the Gault Formation from the landfill at Site 2. Even if the hydraulic gradient was reversed as a result of dewatering, the works would not extend closer than 50 m. Down-hydraulic gradient of the natural flow of any works at Site 1 are expected to be in the region of 300 m from the landfill, and therefore any impacts from the landfill would not be realised at this site well beyond the operational lifetime.

[11] Environment Agency (2006) Remedial Targets Methodology: Hydrogeological Risk Assessment for Land Contamination.

Based on the model results, the Gault Formation acts as a highly effective barrier to contaminant migration, both from the landfill to any works, but also from the works in the Gault Formation to the surrounding area.

6.1.2.1 Vertical migration

Additional consideration has been given to the vertical migration of potential contamination from the landfill to the top of the Lower Greensand as a result of dewatering. The dewatering may result in drawdown of the Lower Greensand aquifer which could enhance the hydraulic drive for contaminant migration from the base of the landfill to the underlying aquifer, as well as imposing a hydraulic gradient within the aquifer towards the construction site, including at Site 2 which is located up-gradient of the landfill under natural flow conditions.

To consider the vertical pathway, the above model was re-run with the compliance point set to 10 m (and associated updated dispersivity), to represent saturated vertical migration through the Gault Formation. This model indicated that the unretarded travel time was 1315 years, and the retarded travel times varied from 2.32e3 to 1.25e7 years.

As it is not known if the Gault is fully saturated, a further model was run at analysis level 2 to represent the concentrations that could reach the base of the Gault Formation in unsaturated conditions, with an assumed Gault thickness of 10m. On the basis that there is likely limited water within the Gault, application of the model to consider vertical migration through the unsaturated zone is still likely to be representative of vertical migration through the Gault. Input parameters are listed in Table 6.4.

Table 6.4: ConSim vertical model Site 1 and 2 physical input parameters

Parameter	Distribution	Value	Justification
Analysis level	**2**		
Source			
Area (m^2)	Single	400,000	Area calculated from drawings of landfill cells
Thickness (m)	Single	27	Estimated from design drawings in unlined cells
Aquifer – Gault Formation			
Thickness (m)	Single	10	Assumed >10m thickness underlying landfill
Dry bulk density (g/cm^3)	Uniform	1.45, 1.58	BGS (1995) Engineering geology of British rocks and soils - Gault Clay. Page 23 (http://nora.nerc.ac.uk/id/eprint/20421/1/WN94031.pdf)
Fraction of organic carbon (%)	Uniform	0.048, 0.051	Environment Agency (2002) - Jurassic aquitards
Hydraulic conductivity (m/s)	Uniform	1e-8, 1e-7	Professional judgement based on conditions encountered in borehole logs and knowledge of the area
Water filled porosity (fraction)	Single	0.47	Environment Agency (2008) SR3, Table 4.4 - clay
Vertical dispersivity (m)	Uniform	1	10% distance of flow (10m)
Infiltration (mm/year)	Normal	300, 100	Normal distribution based around CIEH CHESS Met data for Cambridge (2012) (precipitation – evapotranspiration)
Receptor			
Base of Gault Formation			

Results of the modelling are presented in Appendix C.1 and summarised in Table 6.5.

Table 6.5: Site 1 and 2 ConSim vertical model results summary

Contaminant	SSTL (mg/l)	Unretarded travel time to base USZ (years)	Retarded travel time to base USZ (years)
Cadmium	5.0e-3 (DWS value)	1.41e1	4.65e3
Copper	2.0e0 (DWS value)		1.64e3
Mercury	N/A - No breakthrough occurred to receptor		2.32e4
Nickel	N/A - No breakthrough occurred to receptor		2.32e4
Potassium	1.2e1 (DWS value)		2.69e2
Zinc	5.0e0 (DWS value)		1.78e3
Aliphatic C5-C6	1.62e-1		3.24e1
Aliphatic C6-C8	1.62e-1		1.05e2
Aromatics EC5-7	1.03e14		1.61e1
Aromatics EC7-8	5.88e10		2.01e1
Naphthalene	2.53e5		4.37e1
Ammoniacal nitrogen	3.80e-1 (DWS value)		1.78e2

Modelled steady-state concentrations of the mobile hydrocarbons indicated that degradation and sorption are likely to occur during migration to the base of the unsaturated zone, which would occur even with increased drawdown. Where breakthrough occurred in the metals and ammoniacal nitrogen, little retardation was modelled to occur and therefore steady state concentrations remained at the input concentration, with a corresponding SSTL as the water quality standard (in this case the DWS).

When considering the modelled diluted concentration (based on arbitrary but highly conservative input concentrations of 100 mg/l, as no data are currently available on landfill leachate concentrations), no breakthrough was recorded, indicating that concentrations of the contaminants would be insignificant (less than 1xe^{-30}).

Results of the modelling indicate that without retardation, travel times through the 10 m of Gault clay underlying the landfill would be in the region of 14 years. However, from the results of the monitoring in the Lower Greensand, there is no indication that contamination from the landfill has occurred, despite it having been constructed since 1982[12]. The retarded travel times range between 16 and 105 years for the modelled hydrocarbons, and over 1000 years for the metals, with the exception of potassium at 269 years.

Although dewatering in the Lower Greensand could enhance flows vertically through the Gault Formation, it is important to note that dewatering is anticipated to last for a maximum of four months, after which enhanced flow in the Gault will be reversed.

Based on the vertical migration modelling, there is not likely to be a significant increase in the contaminant concentrations reaching the Lower Greensand or being pulled towards the dewatering sites and any impacts from contamination are likely to be insignificant, even under a worst case (unsaturated) scenario. In reality, there is likely to be some unsaturated and some saturated migration resulting in travel times between those seen in the two models.

It can therefore be concluded that the shaft construction works, including dewatering, would not have a significant impact on mobilising contaminants originating from the landfill.

[12] From historical mapping.

6.1.2.2 Sensitivity test

Sensitivity testing was undertaken on the lateral flow model to assess the impacts of variable input parameters, particularly those with the greatest uncertainty. The parameters for the sensitivity test were chosen due to the wide range and variability in values used in the model, as a result of the lack of site specific data. The parameters tested are:

- Fraction of organic carbon;
- Hydraulic conductivity; and
- Effective porosity.

A summary of the findings against one inorganic (potassium) and one organic (ammoniacal nitrogen) contaminant are provided in Table 6.6. These contaminants were chosen due to the greatest concentrations recorded during the modelling, as presented in Table 6.3.

Table 6.6: Site 1 and 2 ConSim sensitivity test results

Modified parameter	Input value	SSTL (mg/l)	Unretarded travel time to CP (years)	Retarded travel time to CP (years)
Potassium				
Fraction of organic carbon	0.005	1.96e12	2730	2.77e5
	0.048-0.051	**1.97e12**	**2780**	**2.76e5**
Hydraulic conductivity	1e-6 m/s	1.91e-4	146	1.50e4
	1e-7 - 1e-8 m/s	**1.97e12**	**2780**	**2.76e5**
	1e-9 m/s	No breakthrough	1.40e5	1.51e7
Effective porosity	0.01	1.98e12	342	2.83e5
	0.06	**1.97e12**	**2780**	**2.76e5**
	0.18	1.98e12	5720	2.74e5
Ammoniacal nitrogen				
Fraction of organic carbon	0.005	3.45e5	2730	1.76e5
	0.048-0.051	**2.86e5**	**2780**	**1.80e5**
Hydraulic conductivity	1e-6 m/s	0.38	146	9.31e3
	1e-7 - 1e-8 m/s	**2.86e5**	**2780**	**1.80e5**
	1e-9 m/s	No breakthrough	1.40e5	9.87e6
Effective porosity	0.01	4.23e5	342	1.80e5
	0.06	**2.86e5**	**2780**	**1.80e5**
	0.18	8.62e6	5720	1.80e5

Note: Results from the main model run are in bold for comparison.

The results of the sensitivity tests indicate that the controlling parameter on the migration of contaminants through the Gault Formation is the hydraulic conductivity, with the other parameters showing little impact on the steady state concentrations or travel times.

From investigation and analysis of samples collected in the Cambridge area, as discussed in Section 4.2.2, it is known that the formation comprises a stiff to very stiff clay and the derived permeabilities of the clay are 1.1 x 10^{-8} m/s to 4.5 x 10^{-8} m/s, with an average of 3.0 x 10^{-8} m/s. The values of 1 x 10^{-8} to 1 x 10^{-7} used within the model are therefore considered to be representative and conservative, however if values were marginally greater than this, the model results still indicate that migration of any contaminants from the landfill would be limited since it was constructed in the 1980s, with some management and protective measures in place, resulting in very low levels/no breakthrough to have occurred to date.

From the test, it can be concluded that the model results provide a conservative assessment of the potential contaminant migration that could occur through the Gault Formation at the site, and that even if on-site conditions vary slightly from those parameters used within the model, insignificant concentrations of contamination from the landfill would reach the proposed works.

6.1.3 Conclusions and recommendations

The model has indicated that contaminant migration within the Gault Formation is very slow, owing predominantly to the low hydraulic conductivity of the clay. This is presumably a reason why the landfill was situated within the borrow pit initially, as leachates generated would be effectively contained within the confines of the pit and need to pass through over 10 m of Gault Formation to reach the underlying aquifer in the Lower Greensand.

Modelling of vertical migration of contaminants through the Gault Formation was also undertaken to assess the impacts of contaminants reaching the Lower Greensand aquifer with consideration of potential enhanced vertical migration as a result of dewatering. The results indicated that degradation of the mobile organic contaminants reduced concentrations reaching the aquifer, whilst metals and ammoniacal nitrogen were retarded during migration, and therefore had travel times generally over 1,000 years. On the basis that the dewatering would occur for no longer than four months, the impacts of this on the vertical migration of contaminants through the clay is likely to be insignificant, and it is unlikely to result in the generation of more leachate from the landfill. When combined with existing monitoring data, reporting that there is no significant contamination of the aquifer to date, it is unlikely that the drawdown resulting from dewatering in the Lower Greensand will result in significant vertical flow of contaminants.

This characteristic also then applies to the proposed works for the relocation of the WWTP by limiting water, and therefore contaminant migration, to and from the construction activities and new infrastructure, especially the transfer tunnel which is to be wholly or primarily located in the Gault Formation. Despite this, a 50 m standoff zone surrounding the landfill is to be used, which means that the modelling presents the worst case scenario in terms of distance that the contaminants from the landfill would travel to impact the infrastructure, even if the hydraulic gradient was reversed to encourage migration towards Site 2. The model therefore also accounts for the tunnel transition into the Lower Greensand near the north west of the landfill as at least a 50 m buffer will be included between the landfill boundary at the tunnel and the geology would remain Gault Formation in the 50 m buffer zone. Based on the conservatism build into the model through the selection of input parameters, and the fact that the nearest assumed down-gradient work is anticipated to be approximately 300 m away associated with Site 1, it can be concluded that the landfill will not impact the tunnel or any other proposed infrastructure. Additionally, the methodology proposed to construct the tunnel (see Section 5.2.1) is not anticipated to increase mobilisation of any contamination or allow the tunnel to act as a flow pathway.

Through the implementation of the 50 m standoff zone, the project CoCP, and the use of Environment Agency approved drilling fluids, it is considered unlikely that there will be any impacts associated with contaminant transport from the landfill as a result of the proposed works at Sites 1 or 2.

6.2 Site 3

During the construction and operation of Site 3, the potential exists for contamination from the site to migrate in shallow groundwater through the West Melbury Marly Chalk Formation to the Black Ditch watercourse.

Modelling of this migration has been undertaken to better understand the risks from Site 3 to the nearby environmental receptors connected to the Black Ditch i.e. Stow Cum Quy Fen SSSI and Allicky Farm Pond CWS.

6.2.1 Background and methodology

A further ConSim model was run to simulate the fate and transport of dissolved contaminants in groundwater, sourced from Site 3, with the aim of estimating concentrations that would reach the Black Ditch. The model assumes that any contaminants would result from normal operation of the site and not an incident resulting in significant contamination. It should be noted that the design and construction of the WWTP will include mitigation against major pollution incidents and will include mitigation to limit any contamination during normal operating procedures. The model is therefore a hypothetical model to provide reassurance only.

On the basis of the conceptual model (discussed in Section 3), a ConSim model was created with consideration of the following factors:

- A level 3a (independent groundwater transport) scenario was run to simulate saturated transport of contaminants through the West Melbury Marly Chalk Formation to the compliance point at the Black Ditch. This assumes that any release of contaminants directly enters the groundwater, such as though preferential pathways caused by piling or works, or from the base of a tank during operation, creating a more conservative model than the inclusion of unsaturated zone transport.
- The West Melbury Marly Chalk Formation is underlain by >10 m of Gault Formation, which from the modelling in Section 6.1, is known to have very low permeability and transmissivity. Therefore, the saturated pathway is modelled to exist only in the Chalk.
- Groundwater flow direction and gradient have been calculated based on topographical levels towards the Black Ditch as monitoring data are unavailable.
- The same determinands have been modelled as in Section 6.1, which are known to exceed water quality standards in the area or are considered to be highly mobile in the environment. SSTL have been developed for these determinands through a calculation of the relative difference of the input concentration to the concentration at the receptor, and the applicable water quality standard at the receptor that should not be exceeded.
- The model was run for 1001 iterations to increase the confidence level (or percentile) in the results. Results focus on steady state conditions (assumed to be 10,000 years).
- Input concentrations of determinands were all set to 0.001 mg/l to ensure contaminants did not reach saturation limits, and consequently skew the outputs. The input concentrations are irrelevant when calculating the SSTL.

The following assumptions were also made:

- Groundwater flow in the West Melbury Marly Chalk Formation is assumed to be low in the matrix, due to the low permeability of the formation based on the results of testing of BH01 at the site. However, preferential flow in small channels/cracks is likely based on logs from the borehole.
- Nitrate has not been included in this model, although it is a potential contaminant to be sourced from a water treatment works. This is due to the complexities and interactions of

denitrifying bacteria on the process, which are unknown and modelling nitrate could therefore be unrepresentative. Ammoniacal nitrogen and other determinands have been included.

Parameter values were assigned from site specific data and a variety of literature sources where site specific data were unavailable. A full list of the physical parameters used for the modelling are presented in Table 6.7. Chemical parameters are the same as those presented in Table 6.2.

Table 6.7: ConSim model Site 3 physical input parameters

Parameter	Distribution	Value	Justification
Analysis level	**3a**		Assumes Site 3 as source and migration to off-site receptors.
Source			
Area (m^2)	Single	1,270,000	Area calculated from drawings of Site 3
Aquifer – West Melbury Marly Chalk			
Saturated thickness (m)	Single	5.2	Estimated from Leapfrog model - the difference between the base of the Grey Chalk (10.9 mbgl) at BH01 and the water level (5.7 mbgl).
Dry bulk density (g/cm^3)	Triangular	1.88, 1.89, 1.9	Reported values from BH01 during site investigation
Fraction of organic carbon	Single	0.0004	Reported values from BH01 during site investigation
Saturated hydraulic conductivity (m/s)	Triangular	6.99e-8, 1e-6, 1e-5	Min value estimated from permeability testing carried by a rising head test on BH01 located on Site 3. Likely and max values based on dewatering assessment for Site 3 (see Section 4.2.5)
Effective porosity (fraction)	Uniform	0.01, 0.3	PSD of samples from BH01 returned clayey silt. Estimated effective porosity from McWorter and Sunada (1977)
Hydraulic gradient (m/m)	Single	0.005	Based on topographic gradient, towards Black Ditch
Groundwater flow direction (º)	Single	45	Based on topographic gradient, towards Black Ditch
Longitudinal dispersivity 1 (m)	Uniform	5	10% distance to Black Ditch
Lateral dispersivity 1 (m)	Uniform	0.5	1% distance to Black Ditch
Longitudinal dispersivity 2 (m)	Uniform	40	10% distance to standard aquifer compliance point
Lateral dispersivity 2 (m)	Uniform	4	1% distance to standard aquifer compliance point
Infiltration (mm/year)	Normal	300, 100	Normal distribution based around CIEH CHESS Met data for Cambridge (2012) (precipitation – evapotranspiration)
Receptors			
Compliance point 1 (m)	Single	50	Standard aquifer compliance point
Compliance point 2 (m)	Single	400	Distance downgradient to Black Ditch

6.2.2 Results

A summary of the modelling results are presented in Table 6.8, with full results provided in Table C.3 in Appendix C, and an image of the model setup in Figure C.21. SSTL have been calculated to understand the concentration of determinand that could leave the site, without exceeding water quality standards at the receptor.

Table 6.8: Site 3 ConSim model results summary

Contaminant	SSTL for 50m CP (mg/l)	Retarded travel time to 50m CP (years)	SSTL for Black Ditch (mg/l)	Retarded travel time to Black Ditch (years)
Cadmium	2.09e4	>1000	9.79e6	>1000
Copper	0.01	>1000	1506	>1000
Mercury	1.19e7	>1000	3.82e11	>1000
Nickel	9.50e8	>1000	3.05e13	>1000
Potassium	12	>1000	15.5	>1000
Zinc	0.13	>1000	8.07e4	>1000
Aliphatic C5-C6	2.48	172	336	303
Aliphatic C6-C8	2.48	203	336	353
Aromatics EC5-7	2.18	166	275	291
Aromatics EC7-8	1.70e5	168	5.52e8	295
Naphthalene	1399	178	2.72e6	310
Ammoniacal nitrogen	0.38	>1000	0.38	>1000

The results from this modelling indicate that once contaminants are in the groundwater in the West Melbury Marly Chalk Formation, migration downgradient will occur, likely in the small preferential flow channels in the strata. It is not known what the exact transmissivity of these channels is, however, a conservative judgement has been made from borehole logs and permeability values from the dewatering analysis, which has been modelled alongside site collected permeability test data carried by a rising head test on BH01. It should be noted that the site collected data are up to three orders of magnitude below the estimated flow in the channels, which is considered to provide sufficient conservatism within the model.

Unretarded travel time to the 50 m compliance point was modelled to be 165 years, and to the Black Ditch was modelled to be 290 years, which are based on the assumption that preferential flow channels are present in the Chalk in the direction of the receptors. However, these numbers are not representative of real conditions as subsurface geochemical and biological processes will cause the contaminants to sorb, degrade or change redox state, which will slow the travel as well as reduce concentrations reaching the receptors.

Overall, the results indicate that for most determinands, the retarded travel time to the receptors is significant (>1,000 years) for the inorganics. For the determinands with a retarded travel time <1000 years, the organics (hydrocarbons), further consideration has been given to the SSTL relative to the solubility limit of the determinands. The results of this assessment are provided in Table 6.9.

Table 6.9: Solubility limits of key determinands

Contaminant	SSTL for 50m CP (mg/l)	SSTL for Black Ditch (mg/l)	Solubility limit[13] (mg/l)
Aliphatic C5-C6	2.48	336	36
Aliphatic C6-C8	2.48	336	5.4
Aromatic EC5-7	2.18	275	1800
Aromatic EC7-8	1.70e5	5.52e8	590
Naphthalene	1399	2.72e6	25

[13] Taken from Table 5.1 in CL:AIRE (2017) Petroleum Hydrocarbons in Groundwater: Guidance on assessing petroleum hydrocarbons using existing hydrogeological risk assessment methodologies. ISBN 978-1-905046-31-7

Where the SSTL exceeds the solubility limit, the release of the determinands at this concentration would produce free-phase contamination. Contamination in free-phase would limit the migration of the contamination through the West Melbury Marly Chalk Formation which can be monitored by groundwater monitoring and remediated accordingly. It should be noted that any fuel spills would be contained on site by the containment system designed for the site.

Aromatic EC5-7 (benzene) is therefore the main contaminant of concern at the site based on the travel time and solubility limit, however the predominance of this contaminant at the site would likely be limited as a result of site design, management systems and suitable operation procedures to minimise any release of fuels. In addition, the ConSim model assumes an infinite source, however any contamination spill incident would be an isolated event or mobilisation following rainfall, rather than an ongoing source. Dilution would occur associated with rainfall events, which would be a further factor, especially if the leak was a result of a failure of the drainage system[14]. Degradation would also occur prior to the entry of the contaminant to the groundwater, which would act to reduce concentrations prior to their mobilisation in groundwater. It is therefore considered that the risk of this, or any of the other determinands reaching the receptors at significant concentrations and causing impacts are low.

Despite the modelling results and conclusions, it should be remembered that the design and construction of the WWTP will include mitigation against major pollution incidents and will include mitigation to minimise the generation or mobilisation of contamination. The likelihood of contamination exceeding the SSTL being released into the groundwater at the site is therefore considered very low to negligible.

6.2.3 Conclusions and recommendations

Although migration of contaminants through the West Melbury Marly Chalk Formation could occur, the retarded travel time exceeds 1,000 years for inorganics and is considered insignificant, as indicated in the EA Remedial Targets Methodology. Concentrations of the contaminants would be reduced through physical, geochemical and biological processes prior to reaching compliance points, which has been represented in the retarded travel times. The hydrocarbons have lower retarded travel times but the modelling indicates concentrations would have to be indicative of free phase before concentrations could impact Black Ditch. Site containment, management and operational systems will capture any fuel spills on the site and minimise leakages from drainage.

Should any unforeseen contamination reach the Black Ditch, it would enter the watercourse, which was estimated very approximately to have a flow rate of the order of 50 l/s during a site reconnaissance visit in December 2020. This would result in instant and significant dilution of any contaminants within the groundwater prior to the ditch water entering Stow Cum Quy Fen SSSI or any other water bodies or ecological sites (such as Allicky Farm Pond CWS) associated with Black Ditch. In lower flow conditions, greater retardation and degradation of contaminants (especially organics) will occur during migration in Black Ditch, which will further limit the potential concentrations at the CWS and Stow Cum Quy Fen.

Construction and operation of the works would be managed under appropriate and robust management plans. These would include a foundation works risk assessment, code of construction practice and operational environmental management plan, which would assess the risks from potential on-site activities and assign appropriate mitigation, in order to reduce the

[14] Assuming a site area of 1,270,000 m^2 and an averaged daily precipitation rate of 2.2 mm/day, the site drainage would accept in the region of 2,800 m^3 runoff per day.

likelihood and impacts of any pollution events to the environment. Through these, the potential for the release of contaminants into the groundwater is limited.

In conclusion, with appropriate construction design, management and operational management, including mitigation features, it is unlikely that significant concentrations of potential contaminants will reach Black Ditch within 1,000 years. For some of the hydrocarbons, the retarded travel time has been modelled within 1,000 years, although solubility limits and dilution greatly reduce any impacts that would occur from the release of these determinands. As such, it is unlikely that there will be an adverse impact on Stow Cum Quy Fen SSSI or Allicky Farm Pond CWS.

7 Conclusions

7.1 Construction of shafts

7.1.1 Requirements for dewatering

Sites 1 and 2

The impacts on groundwater in the Lower Greensand aquifer would be dependent on which method of construction was chosen.

The use of secant piles would mean that dewatering would only be required in the shaft for the material actually being excavated, together with any minor seepages between the piles.

If the shaft was constructed by underpinning, dewatering could be required in the lower part of the Gault Formation to prevent uplift or heave in the base of excavation. Underpinning would also lead to the potential for upward leakage through the base in the Lower Greensand. In addition, up to one metre depth of excavation in the Lower Greensand would be exposed over a period of a few days prior to each ring of concrete lining being installed and grouted. Assuming that the dewatering was undertaken by pumping from a ring of well points surrounding the excavation, the quantities are likely to comprise a substantial portion of the very significant dewatering required for a totally unlined shaft.

Alternative methods of shaft excavation, such as use of localised jet grouting in the Lower Greensand to prevent or significantly reduce the need for dewatering in the aquifer, may be possible but have not been considered in detail at this stage.

Site 3

Both methods of construction, using secant piles and underpinning, would reduce the minor amount of dewatering which should be required for excavating an unlined shaft. The use of secant piles would mean that dewatering would only be required for the material actually being excavated, together with any minor seepages between the piles.

Underpinning would lead to the potential for upward leakage through the base in the Grey Chalk. In addition, up to one metre depth of excavation in the Grey Chalk would be exposed over a period of a few days prior to each ring of concrete lining being installed and grouted. Overall, however, the total volume of dewatering required should be a small proportion of the minor dewatering requirement indicated by the calculations for the unlined shaft excavation.

The shaft connecting to the effluent discharge pipeline or tunnel at Site 3 would also be excavated through the lowermost section of the Grey Chalk. Assuming construction of the shaft was undertaken either by secant piling or underpinning, then the small amount of dewatering required should be similar to the amount of dewatering required in the Grey Chalk during construction of the WWTP terminal pumping station shaft.

Other shafts

Any other shafts required for tunnelling works could be constructed by a caisson method. The amount of dewatering required should be minimal. As the caisson lining sinks gradually following the excavation, the formation in the wall of the excavation would be sealed off from the shaft. Therefore, dewatering would only be required in the shaft for the material actually being excavated within the lining, together with any seepages at the joints between caisson sections and upwelling through the base of the excavation when excavation is taking place in the superficial deposits and Grey Chalk.

7.1.2 Potential groundwater impacts

Lower Greensand at Sites 1 and 2

As dewatering requirements would be minimal for construction using secant piles, the impact on groundwater in the Lower Greensand aquifer, and any abstractions, nature conservation sites or surface water features dependent on or supported by groundwater would be negligible.

Dewatering required with underpinning at Sites 1 or 2 could have a severe impact at the borehole at Sunclose Farm. As a result, it would probably make the Sunclose Farm borehole unusable for a limited period during and following dewatering. The drawdown due to dewatering might also have a substantial impact on the yield of some groundwater abstraction sources located in the Lower Greensand at distances possibly up to about 5 km from Site 1 or 2.

The drawdown due to dewatering as a result of underpinning could also have a significant temporary impact on any spring flows, seepages and local watercourses dependent on groundwater in the Lower Greensand outcrop area. Dewatering might also reduce the groundwater level in the Lower Greensand in the outcrop area near the Cottenham Moat CWS, although it is not known whether there is a connection between the aquifer and Cottenham Moat.

Mitigation measures would be described in detail in the Environmental Statement. However, the following points provide an indication of the practicalities of mitigation if construction of the shaft was undertaken by underpinning:

- If the potential impacts of dewatering indicate that abstraction at an existing groundwater source may not be possible during dewatering then actions will be taken to ensure the required supply can be maintained.
- Providing mitigation for loss or reduction of spring discharges, seepages and flows in watercourses in the Lower Greensand outcrop is unlikely to be practicable in the event that test pumping indicates that dewatering could affect these features. The impact would, however, be temporary, for up to four months during dewatering, plus a subsequent period of aquifer recovery.
- Mitigation, if practicable, may need to be considered in the event that a significant temporary reduction in water levels is predicted in the Cottenham Moat CWS as a result of dewatering in the Lower Greensand.

Discharge of groundwater from the Lower Greensand aquifer during shaft dewatering would involve careful planning and management to avoid the impacts of unacceptable discolouration of any surface water body due to the possible precipitation of iron compounds. Depending on the groundwater quality, mitigation might include treatment prior to discharge from the site.

The Environment Agency has suggested that any groundwater removed during temporary dewatering for shaft construction, particularly from shafts in the Lower Greensand, should be recharged to the aquifer. However, this would be a complex undertaking potentially requiring pressurised re-injection to the Lower Greensand aquifer. The hydro-chemistry of the groundwater might also change as a result of pumping and, potentially, exposure to the atmosphere. In addition, reinjection might not maintain precisely the established long-term groundwater conditions at existing abstractions or at the Lower Greensand outcrop.

It should also be stressed that the effects of dewatering in the Lower Greensand aquifer would be temporary only. The aquifer should return to the established long term condition in reasonable time, following the completion of dewatering.

Grey Chalk at Site 3

As dewatering requirements would be minimal for construction using secant piles, the impact on groundwater in the Grey Chalk aquifer, and any abstractions dependent on or supported by Chalk groundwater, would be negligible.

The impact of shaft dewatering during underpinning should also be small and unlikely to extend beyond the site boundary. As a result, the impact on groundwater in the Grey Chalk aquifer, and any abstractions dependent on or supported by Chalk groundwater, would also be negligible.

Taking into account the assessments for both methods of construction, there should be no impact on groundwater in the vicinity of either the Quy Fen SSSI or the Allicky Pond CWS. Both these nature conservation sites are likely to be located a substantial distance outside the area of influence of any shaft dewatering at Site 3. The impact on baseflows and water levels in any surface water features, potentially dependent on or supported by Chalk groundwater and located down-gradient of Site 3, such as the Black Ditch, would be negligible. No mitigation should therefore be required in relation to groundwater and flows or water levels in surface water features as a result of shaft dewatering.

The dewatering discharge from the shafts is likely to contain silt which could be conveyed or deposited in field boundary ditches discharging to the main watercourse in the area, the Black Ditch. This impact will, however, be avoided by application of mitigation measures included in the code of construction practice (CoCP) for site work. The measures are likely to include retention of the dewatering discharge behind semi-permeable barriers in the upper reaches of ditches inside the site boundary. These temporary structures would allow the sediment to settle or filter out before discharge to the ditch system down-gradient.

As the dewatering quantities with either construction method should be very small, and the dewatering would take place over a limited period of, at most, a few months from the Grey Chalk, there appears to be little, if any, value in attempting to recycle this groundwater back to the aquifer. The disturbance to ground conditions required to implement a short-term infiltration scheme is likely to outweigh any value obtained from such a scheme.

7.2 Potential impacts of tunnels, pipelines and foundations

7.2.1 Tunnelling

Tunnelling would not be expected to lead to significant turbidity in aquifers. With implementation of the CoCP and the use of drilling fluids approved by the Environment Agency, it is unlikely there will be any significant residual impacts on aquifers. However, groundwater quality would be monitored in boreholes located close to the route of the tunnel where it passes through any bedrock aquifer. Monitoring borehole locations, and the groundwater quality data to be collected, will be agreed with the Environment Agency.

Significant leakage would not be expected for any tunnel section in the Grey Chalk during operation owing to the low permeability of the formation. However, it has been assumed that both a primary and secondary lining would be required for tunnel sections passing through both Grey Chalk and Lower Greensand aquifers to mitigate for any potential exfiltration and infiltration.

It has also been assumed that secondary linings would be required for all shafts that penetrate productive superficial deposits, Lower Greensand or Grey Chalk to mitigate for any potential exfiltration and infiltration

The tunnel sections and shafts in the Lower Greensand or the Grey Chalk would replace a minimal part of the overall aquifer. They would not be expected to have any significant impact on groundwater flow.

Construction of a treated effluent transfer tunnel in the superficial deposits or Grey Chalk would not be expected to lead to significant turbidity.

7.2.2 Pipelines

The potential impacts due to construction of the trenches, drill pits and crossings for the pipelines include the following:

- Pollution of surface waterbodies due to poor construction practices.
- Discharge of silt-laden water during dewatering of excavations affecting surface water quality.
- Temporary depletion of groundwater resources or impacts on licensed or private groundwater abstractions as a result of dewatering of trenches and drill pits.
- Temporary impacts on surface water features or baseflows in watercourses which are dependent on groundwater resource, as a result of dewatering undertaken during trench excavation, drill pit construction and the river crossing on the pipeline route from Waterbeach to Site 3.
- Permanent changes to groundwater flow for example due to trenches acting as drains and pipelines interrupting flow.

Dewatering will be required for sections located in superficial deposits, particularly where sands and gravels are encountered in river terrace or alluvial deposits. Dewatering of the superficial deposits could have a localised effect on water levels in the shallow aquifer and any surface water features in hydraulic continuity with the aquifer. However, any impacts would be temporary and no impact on any nature conservation site is anticipated.

Estimation of the potential dewatering rates indicate that construction of the treated effluent and Waterbeach transfer pipelines for Site 3 should require very limited dewatering in the Grey Chalk which would have only a localised and temporary impact on the aquifer and any seepages from the Grey Chalk.

Appropriate standard mitigation methods to limit contamination or pollution of surface watercourses and groundwater will be included in a detailed code of construction practice (CoCP). The CoCP will include measures to ensure that the sediment content of site runoff and dewatering from excavations is at an acceptably low level when discharged to watercourses. Any dewatering discharge from trenches or drill pits will usually be pumped to ground at a point sufficiently far away as not to impact the excavation. If the ground is too saturated to accept the dewatered flow, the dewatering discharge will be pumped to a watercourse with suitable water quality protection measures in place. Dewatering discharge would be subject to approval by the Environment Agency.

Before construction starts, any existing land drainage will be modified to remove the risk of discharge to the pipe trench during construction. After construction is complete, the land drainage will be reinstated to operate as close to its previous condition as practicable. Where gravel is used as pipe bedding, and there is a risk of the trench acting as a land drain, impermeable partitions (clay stanks) will be installed to prevent the transfer of water along the length of the trench.

7.2.3 Foundations and below-ground structures at the new WWTP

The maximum depth of pile foundations for structures at the new WWTP would be in the range 10 to 25 m. The method of piling most likely to be used for foundations (continuous flight auger) limits the potential for cross-contamination between different strata. The presence of deep foundations may affect local flow paths within aquifers. However, taking into account the overall areal extent of the Lower Greensand and Grey Chalk, and the expected low permeability of the West Melbury Marly Chalk Formation in particular, the impact on groundwater flow would be negligible.

It is possible that some of the larger structures will have bases which are below the shallow groundwater table, requiring dewatering during construction and potentially affecting groundwater flow. If dewatering is required in the Grey Chalk at Site 3, the rates of dewatering should be minor. If needed, permanent drainage would be installed to ensure that the groundwater flow is re-directed around the structure.

The CoCP will be adhered to at the new WWTP site, ensuring there will be negligible impacts to the water environment during construction.

7.3 Contaminant transport assessment

Modelling was undertaken for Sites 1 and 2 to better understand the potential for contamination from the existing landfill near Site 2 to affect groundwater at the construction sites. The landfill is located in the Gault Formation.

The conclusions of the assessment indicate that it is very unlikely any contaminants will migrate away from the landfill. This is, in part, due to the existing leachate capture system at the landfill. In addition, the modelling indicates that:

- the low hydraulic conductivity of the Gault Formation will mitigate the migration of potential contaminants, such that they will not reach a 50 m buffer zone from the landfill boundary within 1,000 years; and
- modelled concentrations should not adversely impact groundwater at the construction sites (in either the Gault Formation or the Lower Greensand).

For Site 3, modelling was used to simulate:

- unexpected contamination occurring during construction or operation of the site; and
- the potential impact this could have at Black Ditch and thereby Stow Cum Quy Fen SSSI and Allicky Farm Pond CWS (if the CWS is connected to the watercourse).

In addition to the CoCP and protective design measures that will be in place at the site, should any contamination occur in the West Melbury Marly Chalk, it is unlikely to reach Black Ditch at significant concentrations, or within 1,000 years, as a result of the relatively low hydraulic conductivity of the aquifer at this location. Additionally, any low concentrations that do reach Black Ditch will be diluted further by any flows in the ditch and, possibly, by the addition of groundwater from other areas in connectivity with the ditch.

In conclusion, the modelling has provided supporting information to confirm that:

- Construction of the new WWTP near the landfill is very unlikely to affect groundwater quality; and

- In the unlikely event of any potential spills or leakages of contaminants associated with a WWTP at Site 3, they would not be expected to affect water quality at Quy Fen SSSI and Allicky Farm Pond CWS.

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Appendices

A. Figures

Figure A.1: Location plan

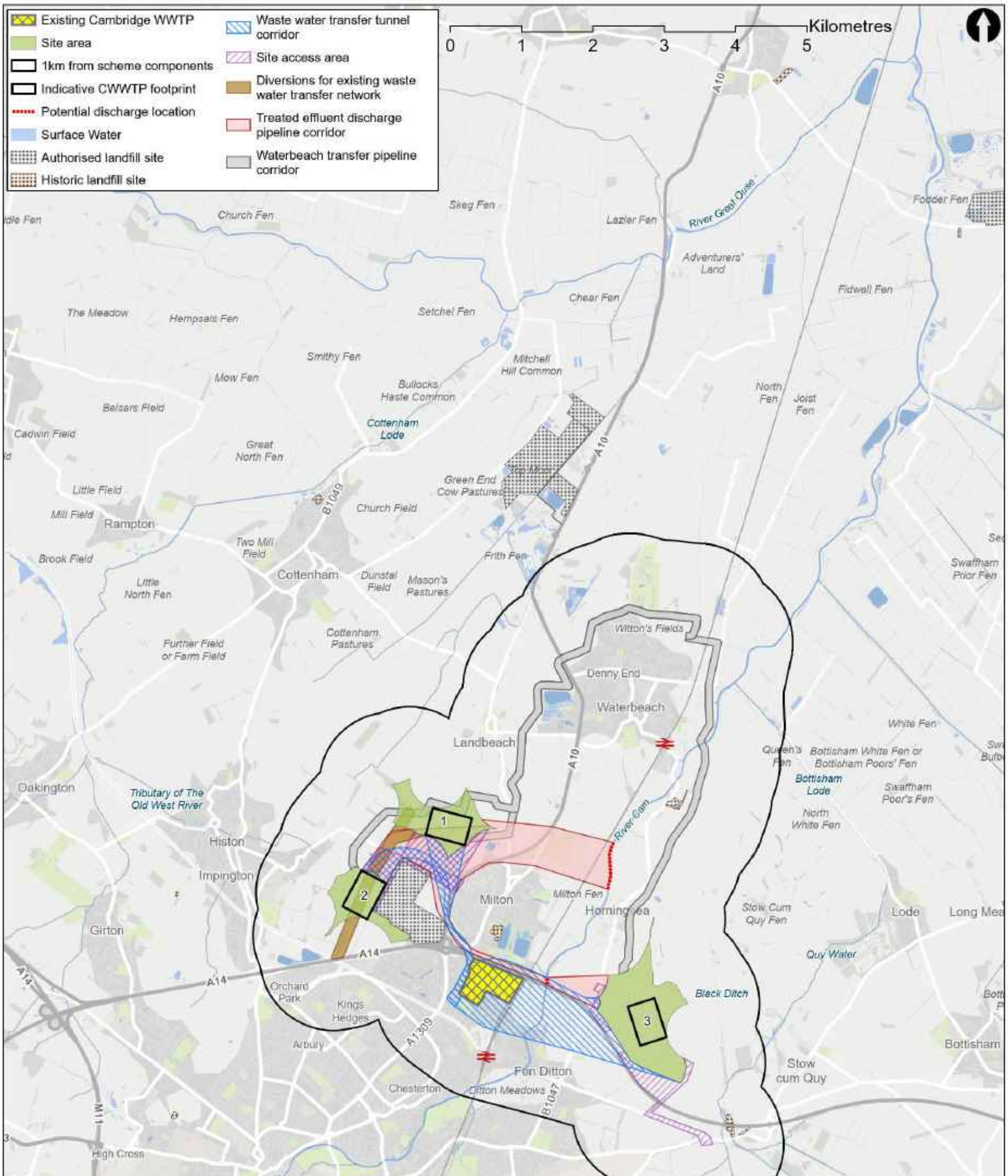


Figure A.2: Flood zones and nature conservation sites

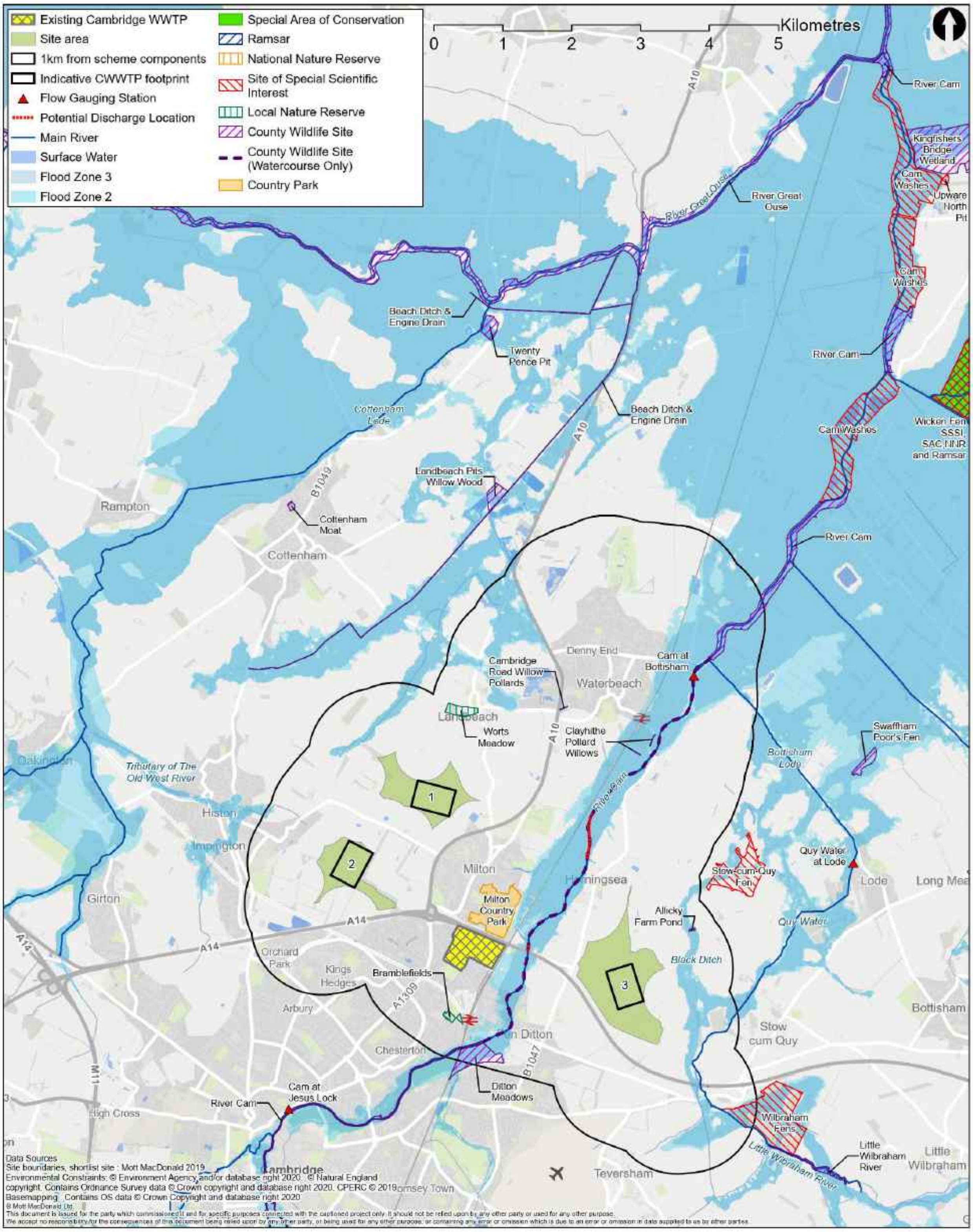


Figure A.3: Bedrock Geology

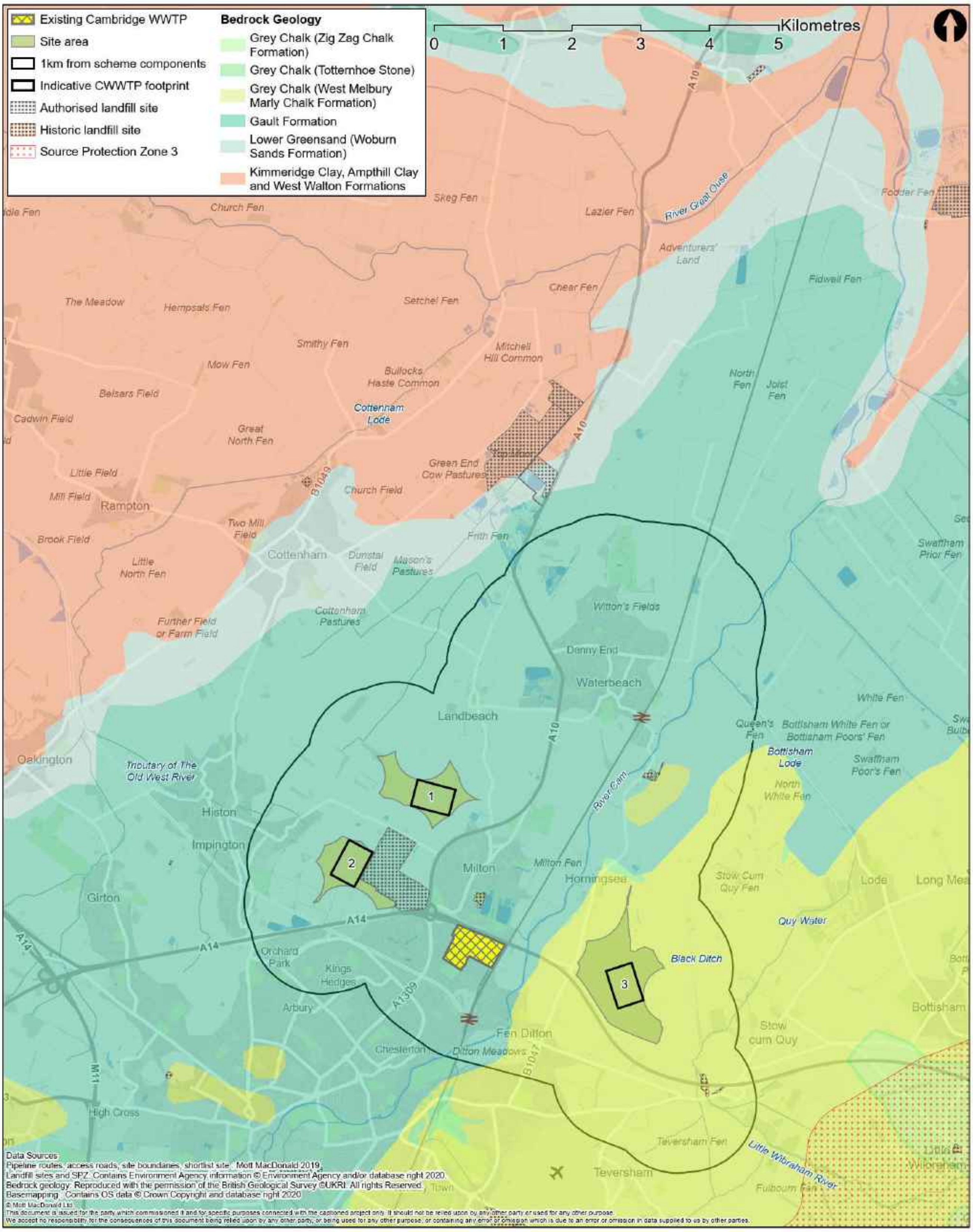


Figure A.4: Superficial Geology

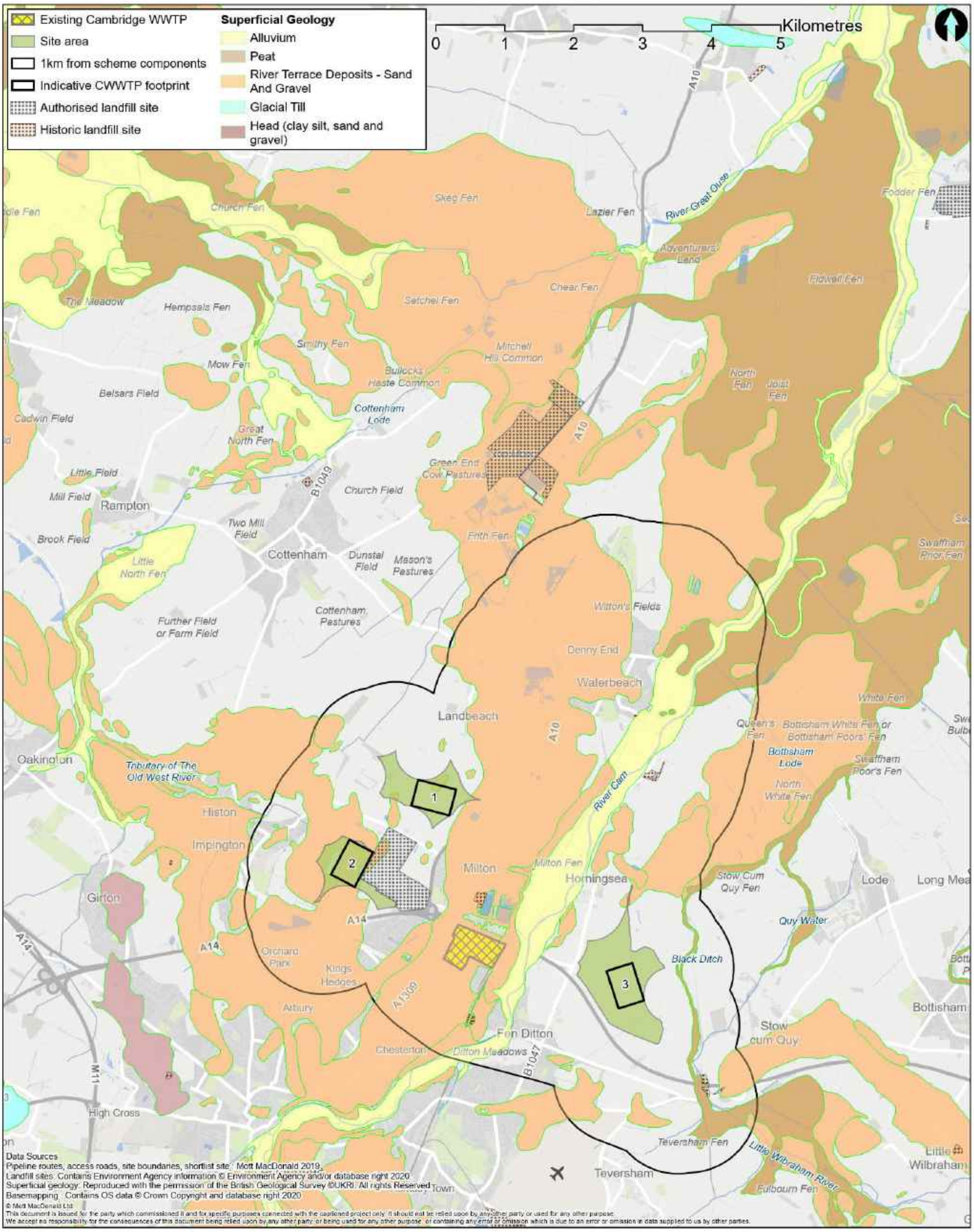
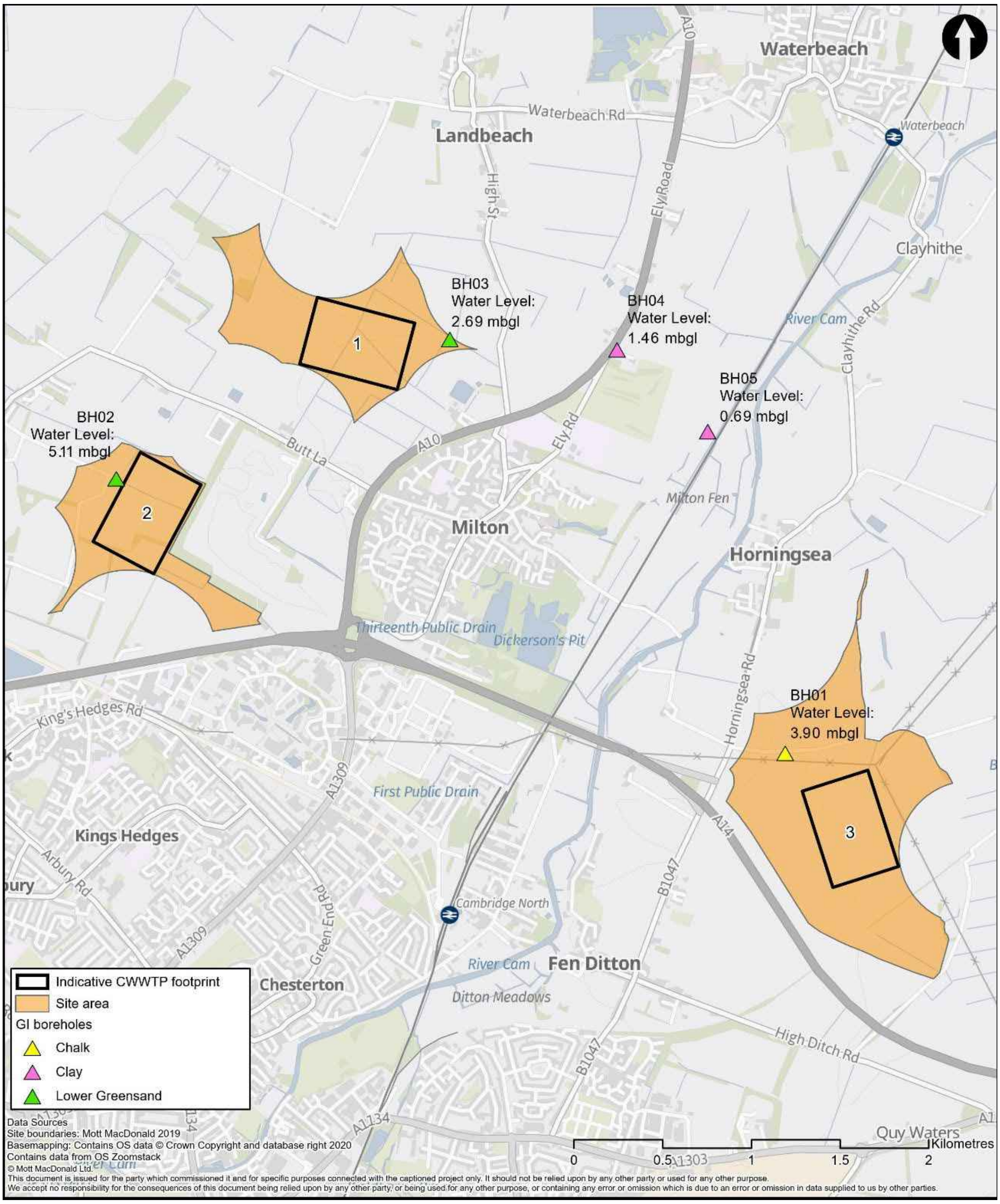


Figure A.5: Ground investigation boreholes



Note: The water levels shown were measured on 14th December 2021

Figure A.6: Stow cum Quy Fen SSSI

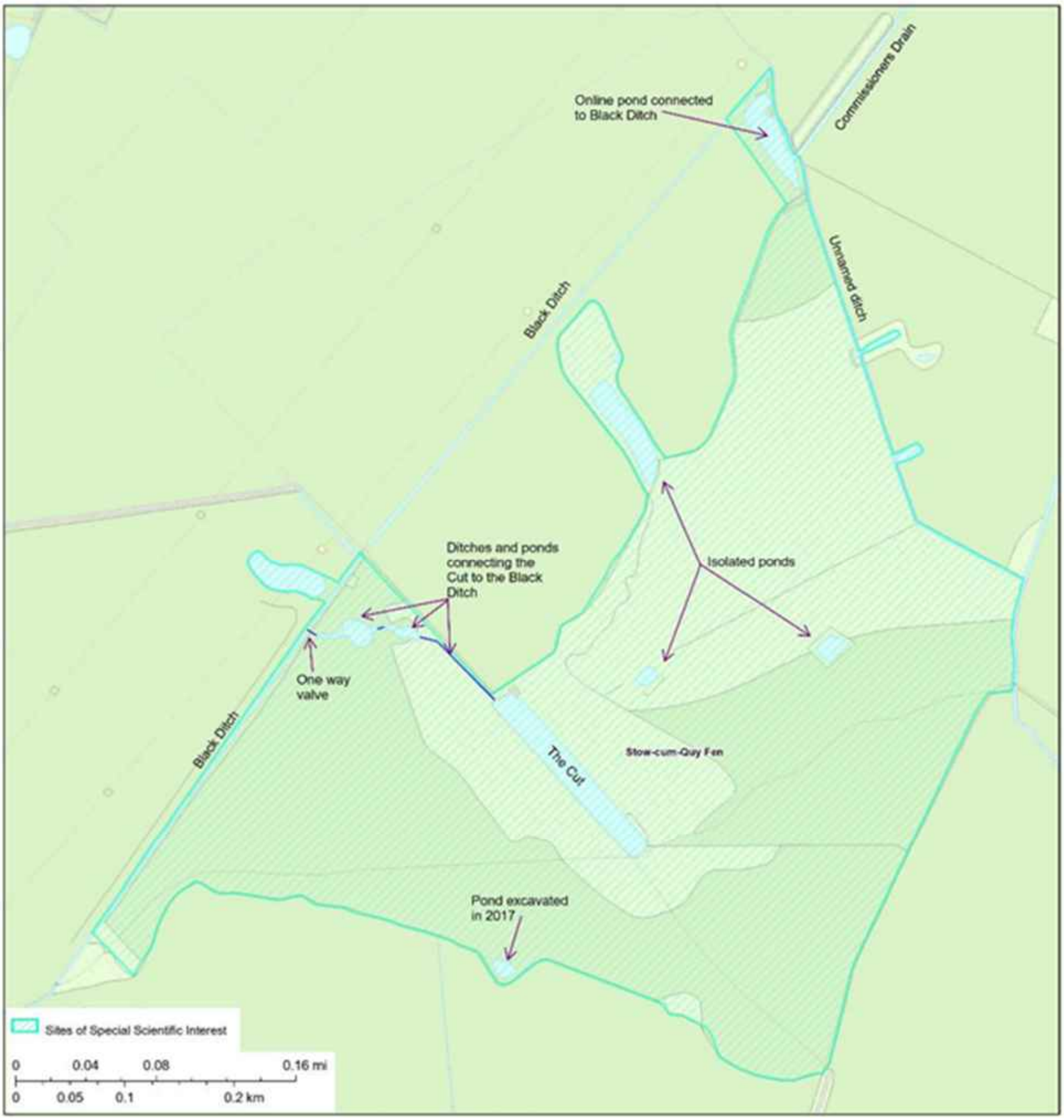


Figure A.7: Cross section locations

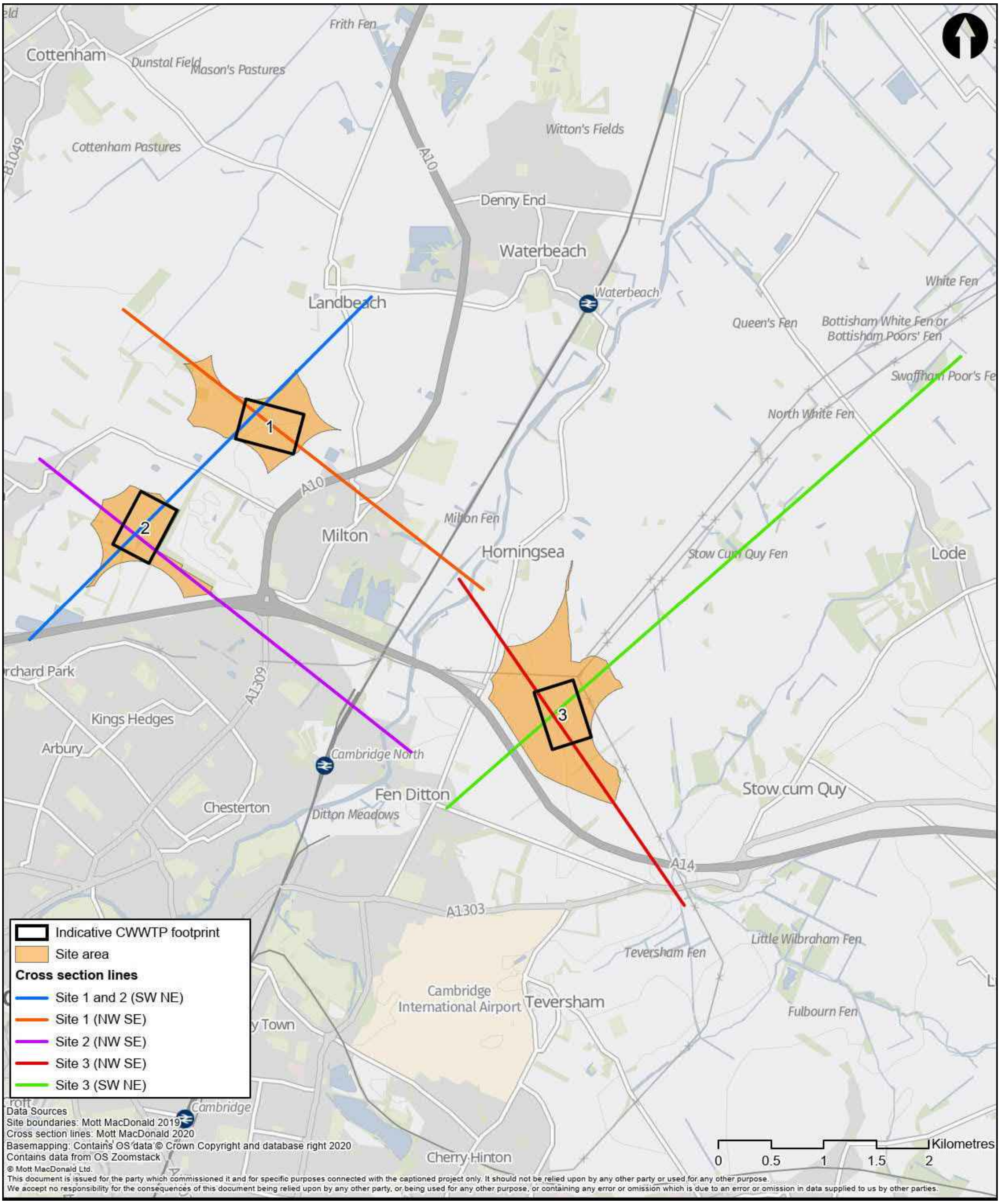


Figure A.8: NW/SE cross section beneath Site 1

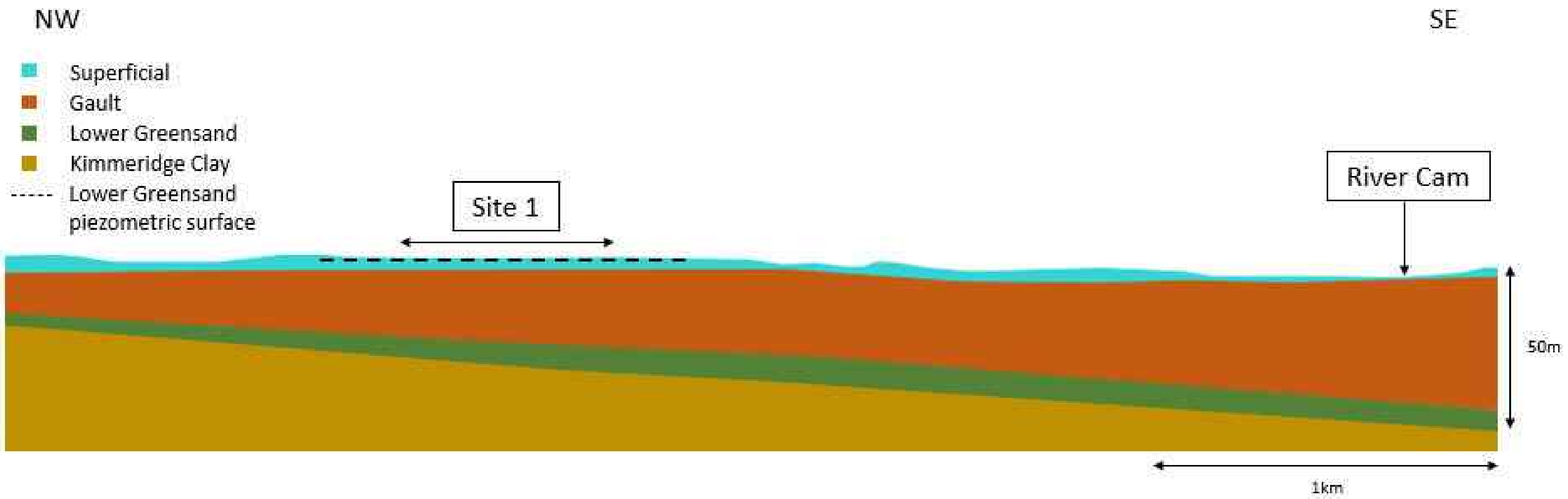


Figure A.9: NW/SE cross section beneath Site 2

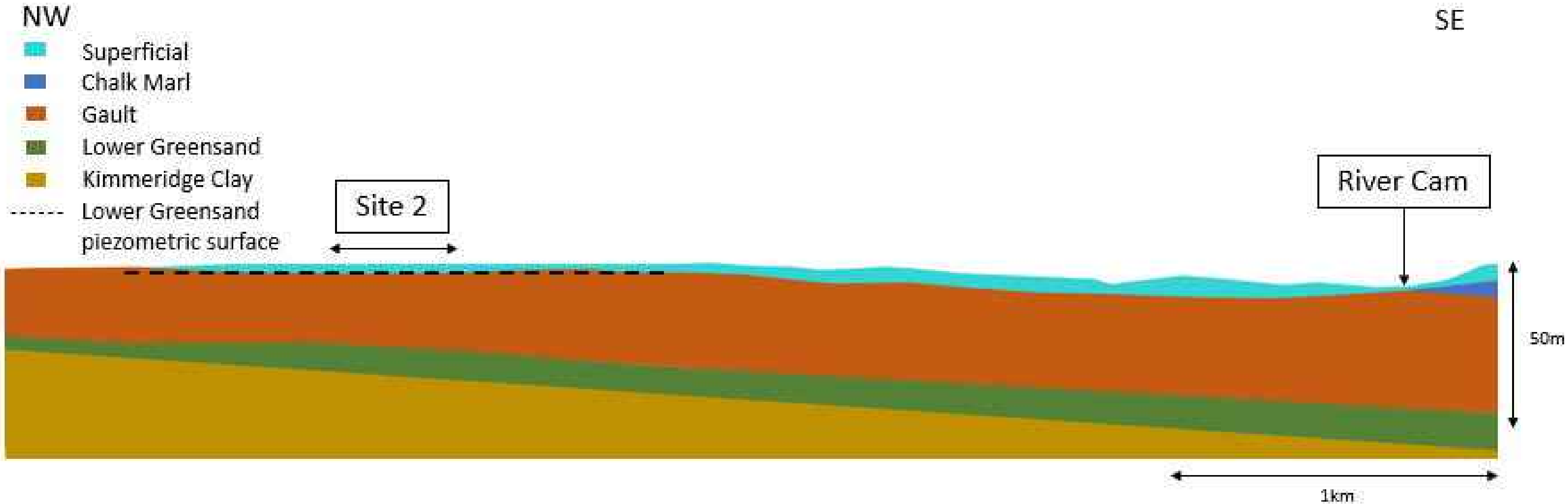


Figure A.10: SW/NE cross section beneath Sites 1 and 2

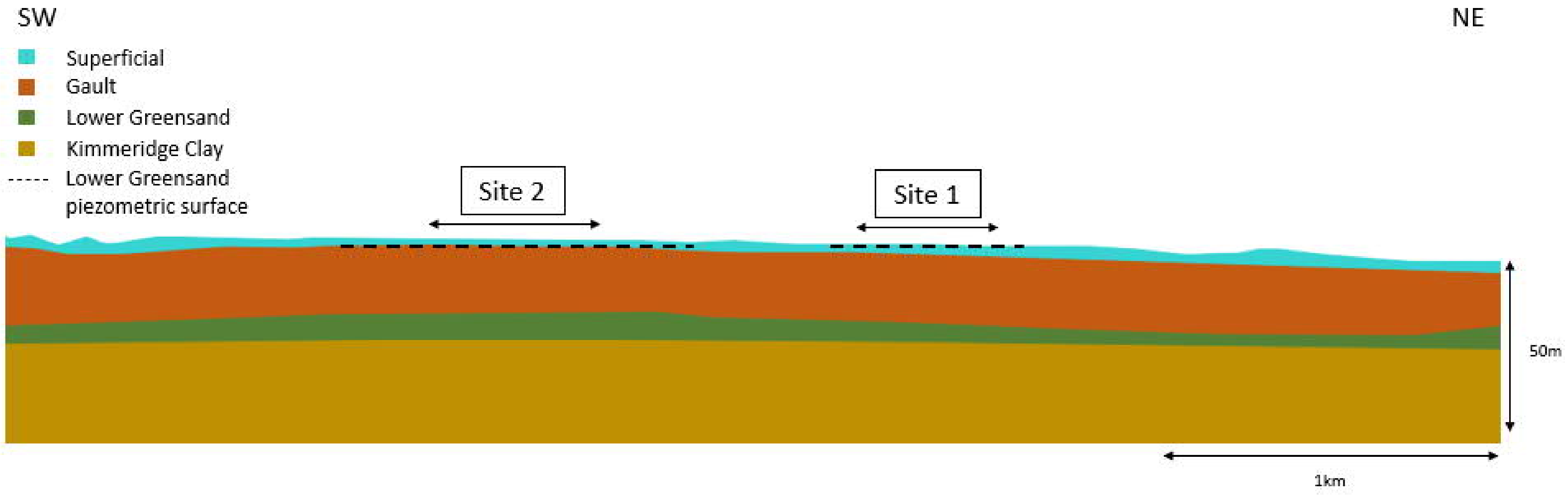


Figure A.11: NW/SE cross section beneath Site 3

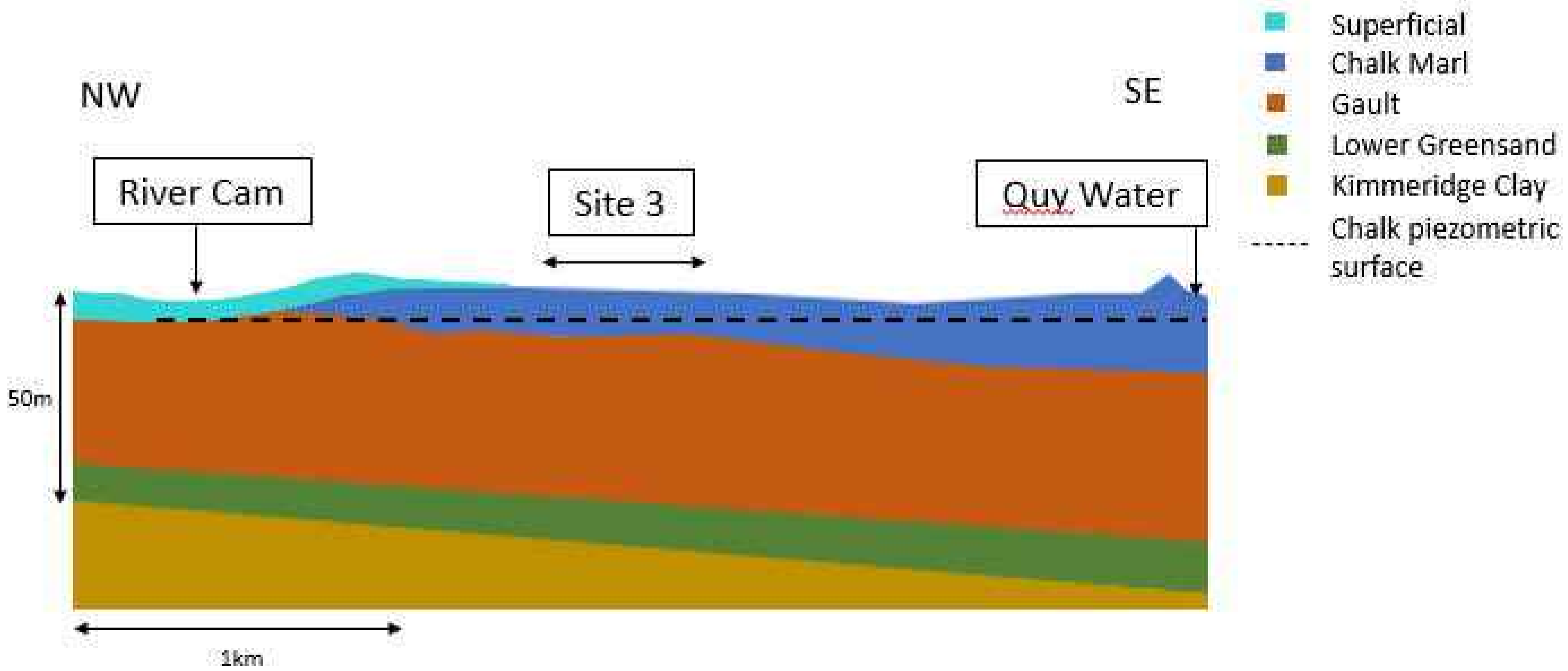


Figure A.12: SW/NE cross section beneath Site 3

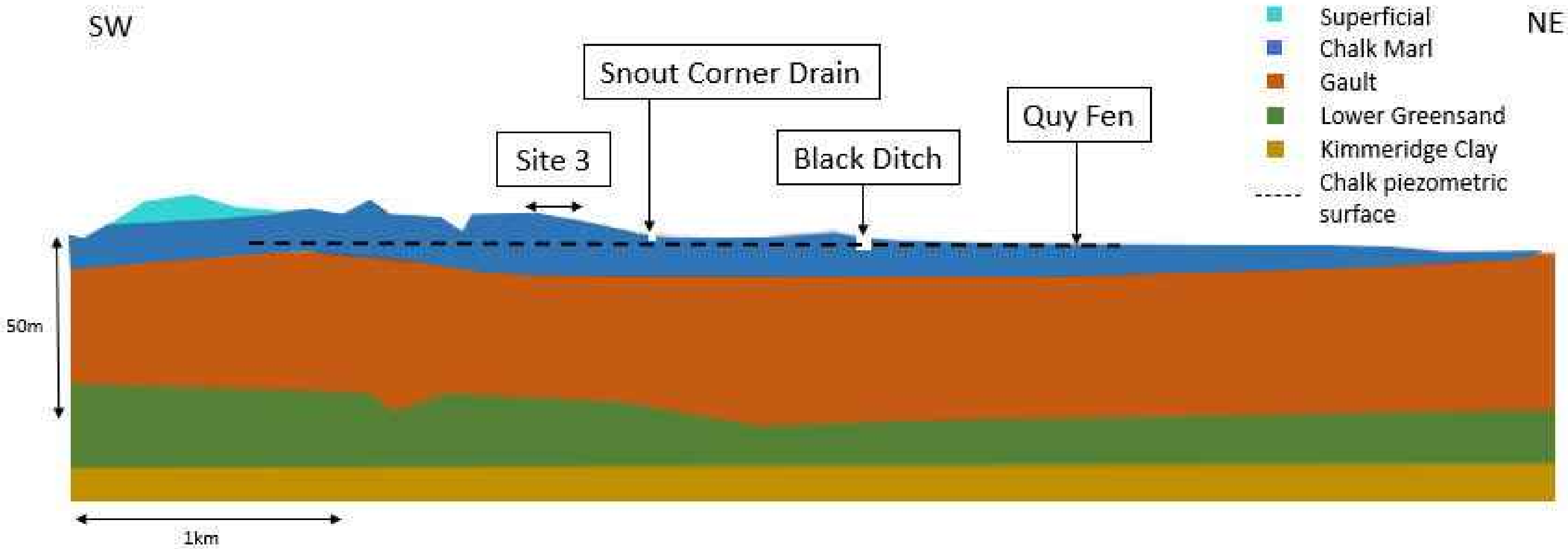


Figure A.13: Waste water transfer tunnel corridors

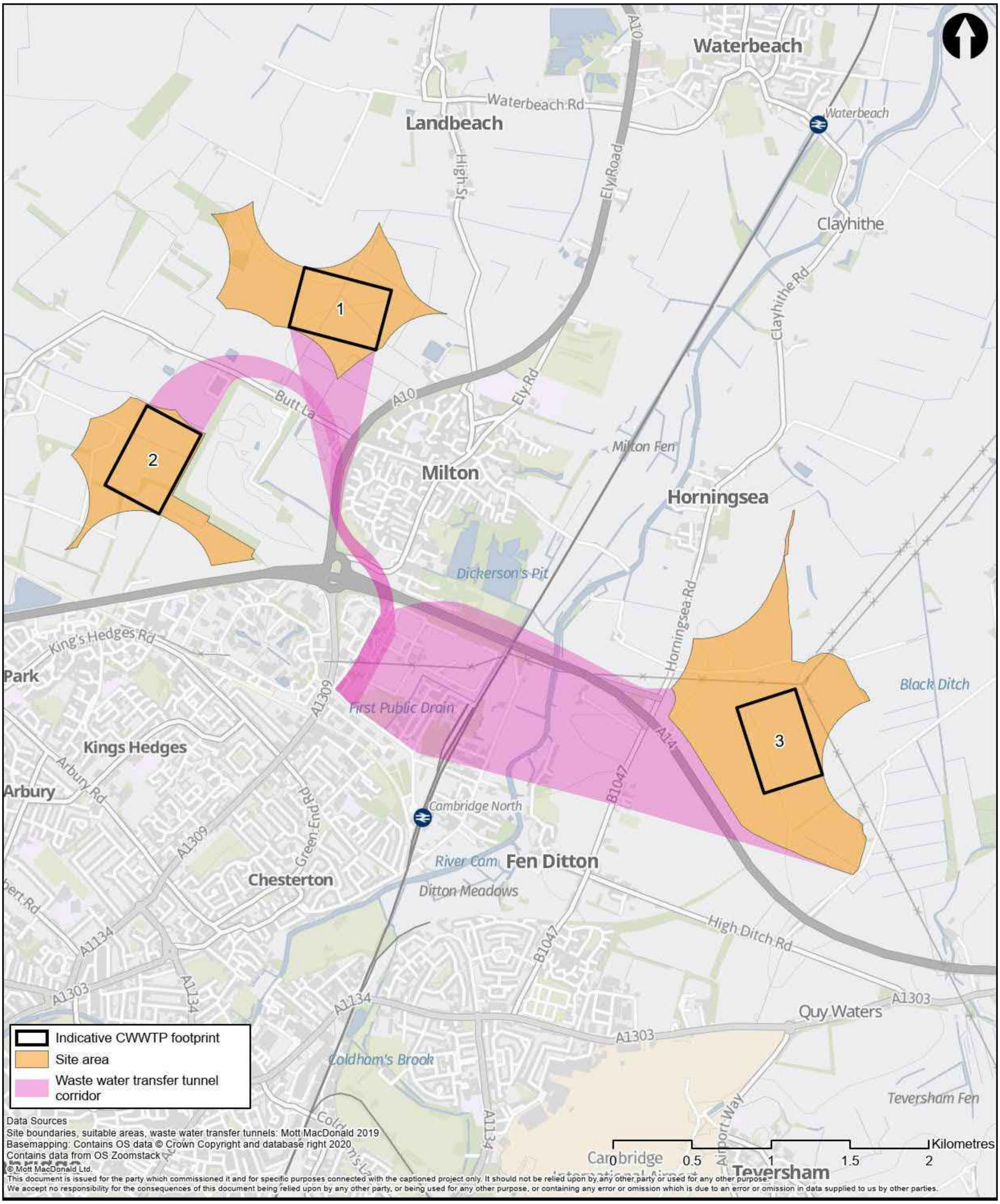


Figure A.14: Transfer tunnel at Site 2

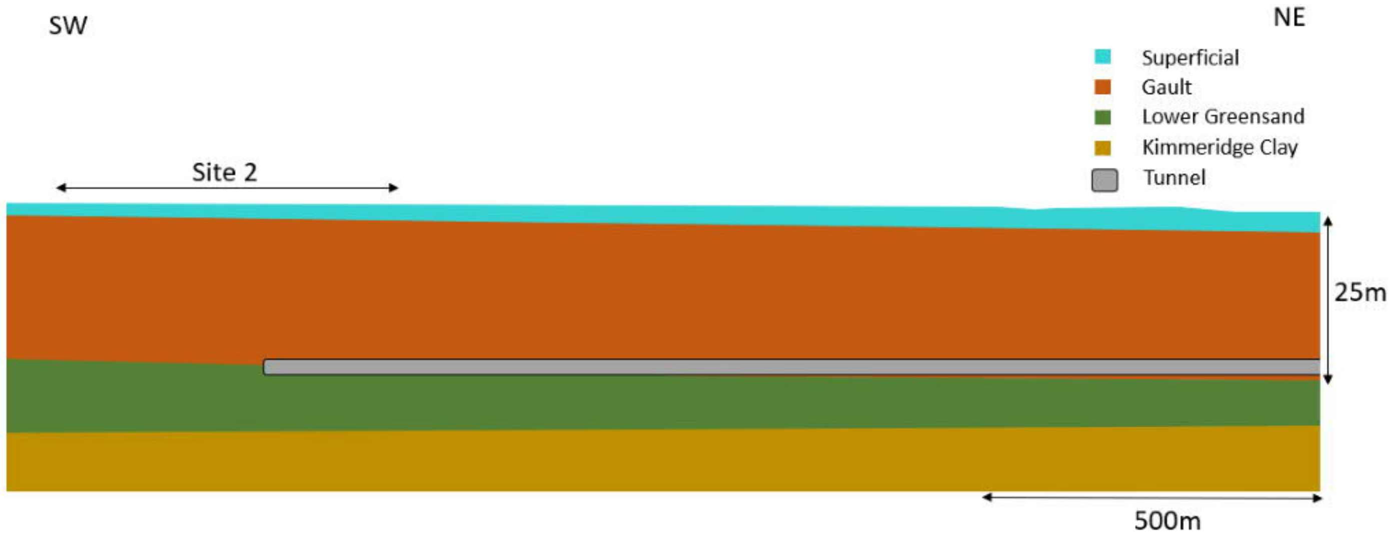


Figure A.15: Waterbeach pipeline corridors

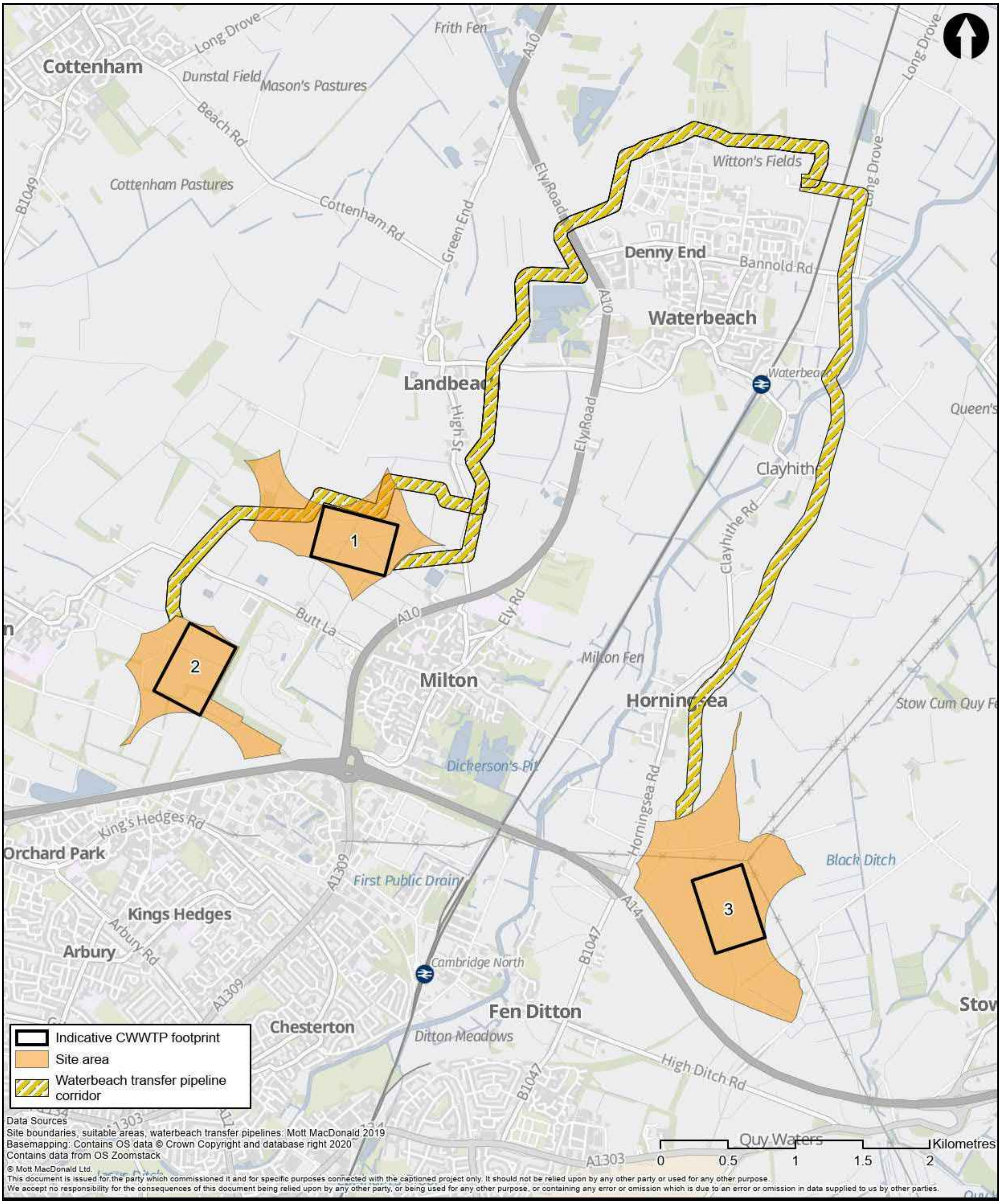


Figure A.16: Treated effluent transfer tunnel/pipeline corridors

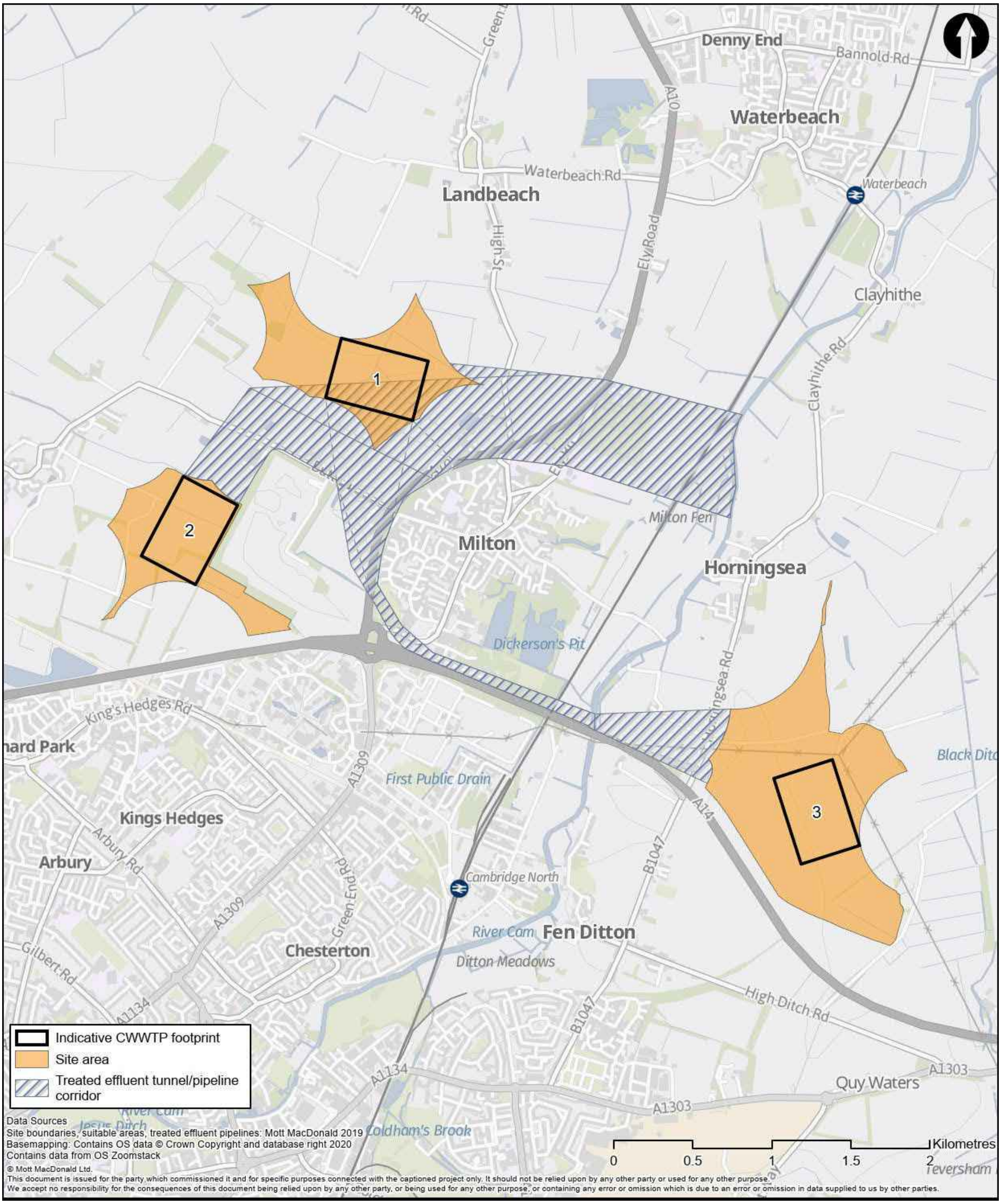


Figure A.17: Maximum foundation depth at Sites 1 and 2

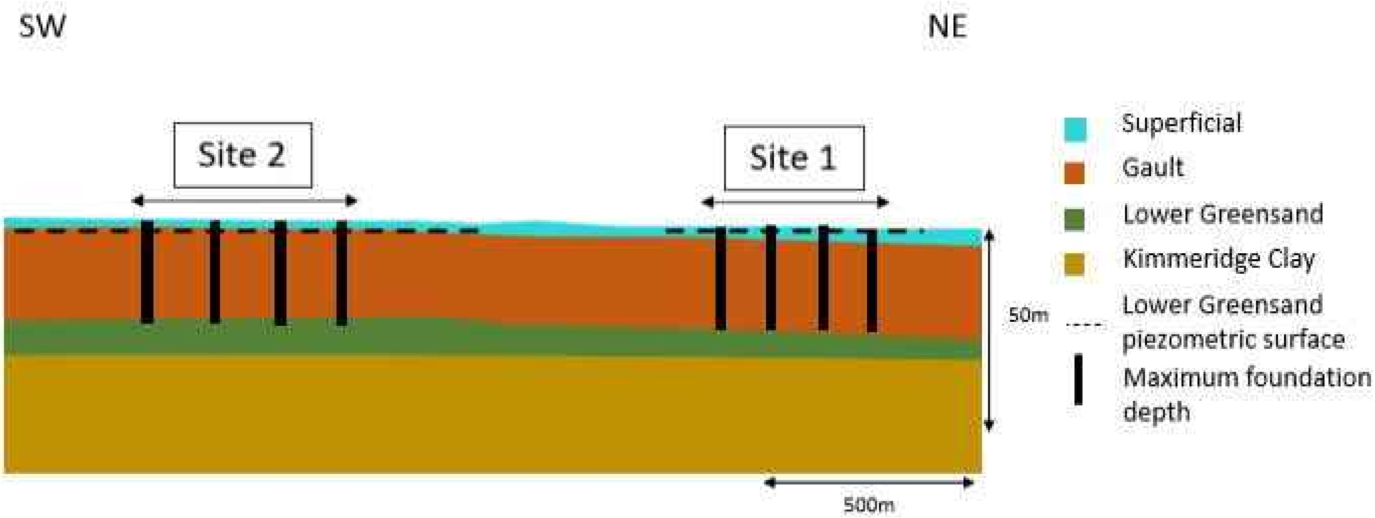


Figure A.18: Maximum foundation depth at Site 3

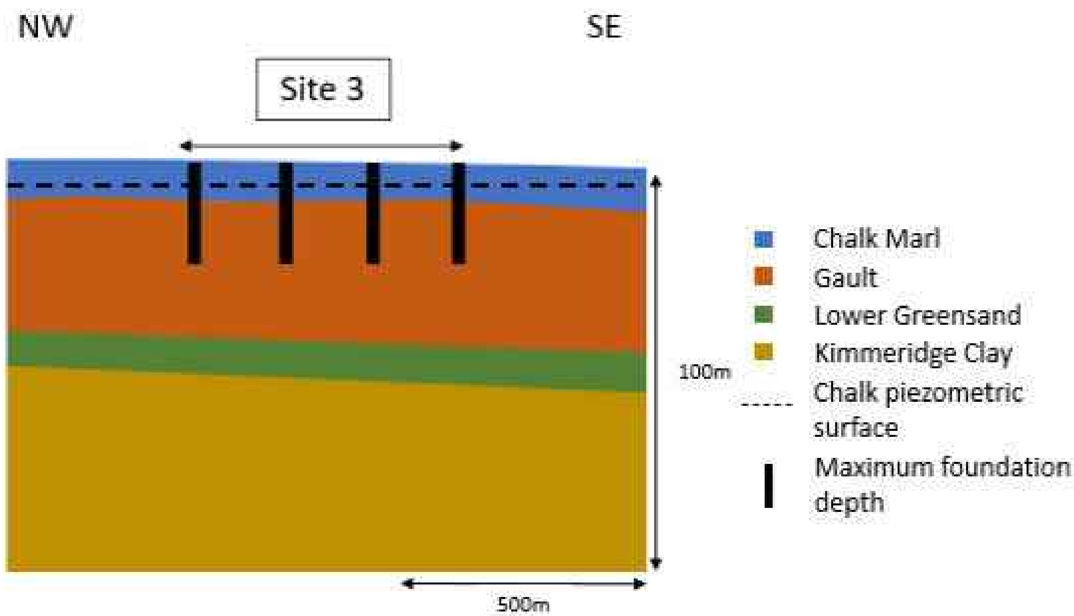
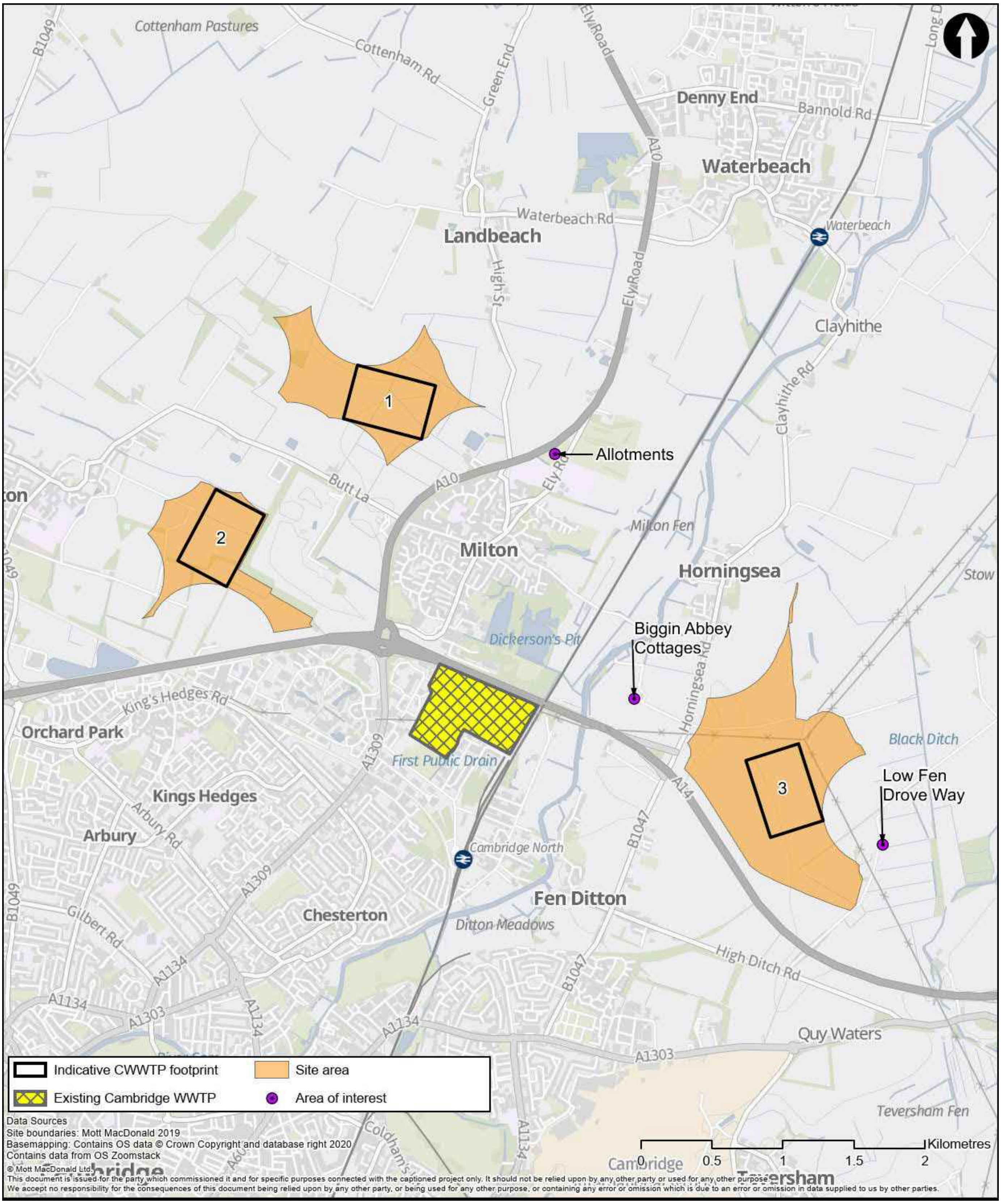


Figure A.19: Surface features of interest during construction



B. Stow Cum Quy Fen photographs

This appendix contains the photographs taken on the reconnaissance visits to Stow Cum Quy Fen on 7th December 2020 (Photos B.1-B.7) and 31st December 2020 (Photo B.8).

Photo B.1: View from south-east bank of the Cut

Photo B.2: South-east bank of the Cut indicating excavation in Grey Chalk

Photo B.3: View of pipe culvert and gauge board at western end of the Cut

Photo B.4: Drainage ditch connecting the Cut to ponds

Photo B.5: View east from bank of Black Ditch towards ponds connected to the Cut

Photo B.6: Control structure with one-way valve on Black Ditch

Photo B.7: View south along Black Ditch towards site 3

Photo B.8: Evidence of flooding on Fen adjacent to Black Ditch

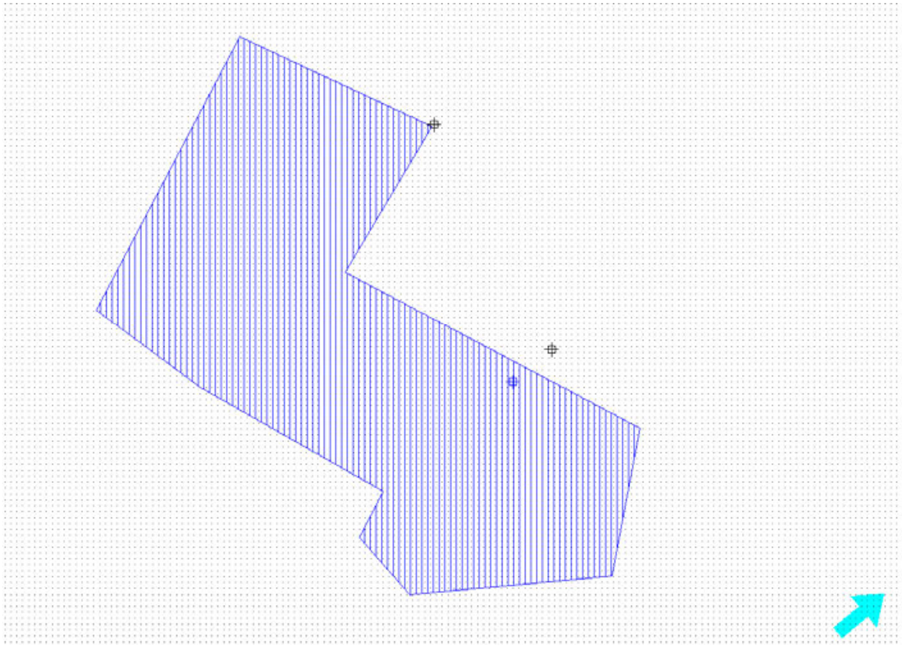
C. ConSim model results

This appendix contains the outputs of the ConSim modelling (discussed in Section 6) to provide a better understanding of the generation of the SSTL.

C.1 Sites 1 and 2

An image of the modelled area is shown in Figure C.20 including landfill boundary (blue), groundwater flow direction, set centre-point from which the model calculates the contaminant migration (blue cross-hair), and receptors (black cross-hairs)

Figure C.20: ConSim model for Sites 1 and 2



Results given as 95th percentile for concentration and 50th percentile for travel times:

Table C.1: Site 1 and 2 ConSim lateral model results

Constituent	DWS	EQS	WQS - Minimum	WQC in effluent used to establish SSTL	Concentrations at 50m CP, steady state	Retarded travel time to 50m CP	SSTL based on 50m CP
	mg/l	mg/l	mg/l	mg/l	mg/l	(years)	mg/l
Cadmium	0.005	0.00008	0.00008	0.001	No breakthrough	4.97E+06	N/A
Copper	2	0.001	0.001	0.001	1.47E-23	1.74E+06	68027210884353700
Mercury	0.001	0.00005	0.00005	0.001	No breakthrough	3.75E+07	N/A
Nickel	0.02	0.004	0.004	0.001	No breakthrough	2.48E+07	N/A
Potassium	12		12	0.001	6.08E-15	2.76E+05	1973684210526
Zinc	5	0.0109	0.0109	0.001	1.61E-24	1.89E+06	6770186335403730000
Aliphatic C5-C6	0.01		0.01	0.001	No breakthrough	2.26E+04	N/A
Aliphatic C6-C8	0.01		0.01	0.001	No breakthrough	1.01E+05	N/A
Aromatics EC5-7	0.01	0.01	0.01	0.001	No breakthrough	4.65E+03	N/A
Aromatics EC7-8	0.01	0.074	0.01	0.001	No breakthrough	8.93E+03	N/A
Naphthalene		0.002	0.002	0.001	No breakthrough	3.46E+04	N/A
Ammoniacal nitrogen	0.38	0.78	0.38	0.001	1.33E-09	1.80E+05	285714

Note: assumed steady state at 10,000 years.

Table C.2: Site 1 and 2 ConSim vertical model results

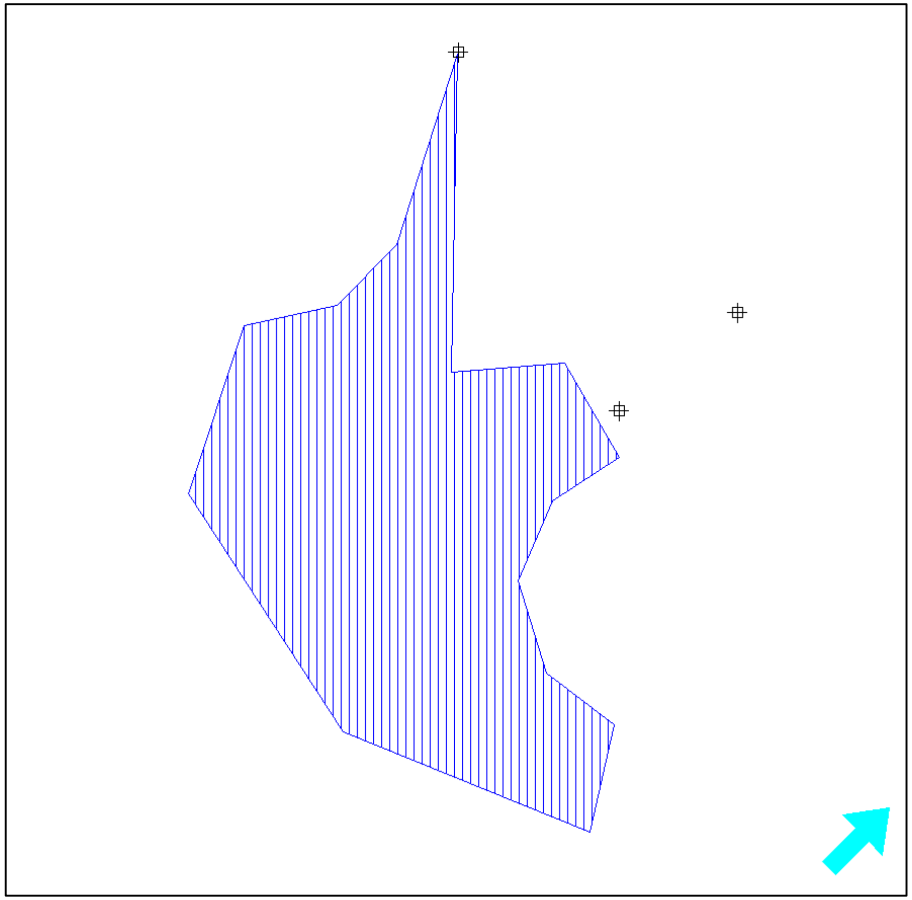
Constituent	DWS	EQS	WQS used	WQC in effluent used to establish SSTL	Concentrations at base USZ, steady state	Retarded travel time to base USZ	SSTL based on base USZ
	mg/l	mg/l	mg/l	mg/l	mg/l	(years)	mg/l
Cadmium	0.005	0.00008	0.005	0.001	0.000998	4.65E+03	0.00501
Copper	2	0.001	2	0.001	0.000999	1.64E+03	2.002
Mercury	0.001	0.00005	0.001	0.001	No breakthrough	2.32E+04	N/A
Nickel	0.02	0.004	0.02	0.001	No breakthrough	2.32E+04	N/A
Potassium	12		12	0.001	0.000999	2.69E+02	12
Zinc	5	0.0109	5	0.001	0.000999	1.78E+03	5.0050
Aliphatic C5-C6	0.01		0.01	0.001	6.16E-05	3.24E+01	0.16234
Aliphatic C6-C8	0.01		0.01	0.001	6.16E-05	1.05E+02	0.16234
Aromatics EC5-7	0.01	0.01	0.01	0.001	9.69E-20	1.61E+01	103199174406605
Aromatics EC7-8	0.01	0.074	0.01	0.001	1.70E-16	2.01E+01	58823529412
Naphthalene		0.002	0.002	0.001	7.90E-12	4.37E+01	253165
Ammoniacal nitrogen	0.38	0.78	0.38	0.001	0.000999	1.78E+02	0.380

Note: assumed steady state at 10,000 years.

C.2 Site 3

Model setup as shown in Figure C.21.

Figure C.21: ConSim model for Site 3



Results given as 95th percentile for concentration and 50th percentile for travel times:

Table C.3: Site 3 ConSim model results

Constituent	DWS	EQS	WQS - Minimum	WQC in effluent used to establish SSTL	Concentrations at Black Ditch, steady state	Concentrations at 50m CP, steady state	Retarded travel time to Black Ditch	Retarded travel time to 50m CP	SSTL based on Black Ditch	SSTL based on 50m CP
	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	(years)	(years)	mg/l	mg/l
Cadmium	0.005	0.00008	0.00008	0.001	8.17E-15	3.83E-12	3.72E+05	2.39E+05	9791922	20888
Copper	2	0.001	0.001	0.001	6.64E-10	1.58E-04	1.30E+05	8.36E+04	1506	0.01
Mercury	0.001	0.00005	0.00005	0.001	1.31E-19	4.21E-15	1.86E+06	1.19E+06	381679389313	11876485
Nickel	0.02	0.004	0.004	0.001	1.31E-19	4.21E-15	1.86E+06	1.19E+06	30534351145038	950118765
Potassium	12		12	0.001	7.75E-04	1.00E-03	2.09E+04	1.33E+04	15.5	12.0000
Zinc	5	0.0109	0.0109	0.001	1.35E-10	8.17E-05	1.42E+05	9.08E+04	80741	0.1334
Aliphatic C5-C6	0.01		0.01	0.001	2.98E-08	4.04E-06	3.03E+02	1.72E+02	336	2.4752
Aliphatic C6-C8	0.01		0.01	0.001	2.98E-08	4.04E-06	3.53E+02	2.03E+02	336	2.4752
Aromatics EC5-7	0.01	0.01	0.01	0.001	3.63E-08	4.59E-06	2.91E+02	1.66E+02	275	2.18
Aromatics EC7-8	0.01	0.074	0.01	0.001	1.81E-14	5.88E-11	2.95E+02	1.68E+02	552486188	170068
Naphthalene		0.002	0.002	0.001	7.34E-13	1.43E-09	3.10E+02	1.78E+02	2724796	1399
Ammoniacal nitrogen	0.38	0.78	0.38	0.001	1.00E-03	1.00E-03	1.37E+04	7.87E+03	0.38	0.38

Note: assumed steady state at 10,000 years.

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