OAKLANDS FARM SOLAR PARK

Applicant: Oaklands Farm Solar Ltd

Environmental Statement

Appendix 8.1 – Flood Risk Assessment and Outline Drainage Strategy

October 2024

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Oaklands Farm Solar Park: Flood Risk Assessment and Drainage Strategy

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Title

Oaklands Farm Solar Park: Flood Risk Assessment and Drainage Strategy

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1. Introduction

1.1. Instruction

Yellow Sub Geo Ltd (Yellow Sub) was instructed by BayWa R.E. UK Ltd (the Client) to provide a Flood Risk Assessment (FRA) and outline drainage strategy for a large parcel of land between Oaklands Farm and Park Farm (the Site).

1.2. Brief

The brief was to provide a suitable Flood Risk Assessment (FRA) and Outline Sustainable Drainage (SuDS) Strategy for the Site to support the application for a Development Consent Order and Environmental Impact Assessment (EIA) for a proposed solar farm.

1.3. Background

The Site is located in Swadlincote to the south of Burton-on-Trent. The proposed development involves the installation of a solar farm comprising ground mounted photovoltaic (PV) panels across 37No. agricultural fields with associated Battery Energy Storage System (BESS) and a connection established to the nearby former Drakelow Power Station.

1.4. Scope

This report presents the findings of an FRA and Outline SuDS Strategy for the Site that demonstrates that the proposed development meets the requirements of the National Planning Policy Framework (NPPF) and Planning Practice Guidance (PPG).

1.5. Limitations

This report is written strictly for the benefit of the Client and bound by the conditions presented in Appendix A.

2. Development description and location

2.1. The Site

The Site (Figure 2-1) lies within the administrative boundaries of South Derbyshire District Council (SDDC) and Derbyshire County Council (DCC), located approximately 0.25km west of the village of Rosliston and 0.7km south east of Walton-on-Trent and stretching from the former Drakelow Power Station, north of Walton Road, to the south of Coton Road. The Site occupies a total area of approximately 191 hectares (ha), although Oaklands Farm covers only 135ha of the Site.

The Site itself includes land within three farms, Park Farm in the north, Fairfields Farm in the centre of the Site and Oaklands Farm in the south. The Drakelow substation land, where the Proposed Development will connect to the grid, is north of Walton Road within the former Drakelow Power Station site.

The southern part of the Site (Oaklands Farm area) comprises a large area of agricultural land to the south of Rosliston Road and west of Catton Lane that wraps around the north and east of the farmstead at Oaklands Farm. A small part of the Site extends south of Coton Road.

A small section of the Cross Britain Way / National Forest Way long distance path (which runs between the villages of Walton Upon Trent and Rosliston), crosses the northern fields of the Oaklands Farm area and is partly enclosed by woodland associated with the Rosliston Forestry Centre to the north-east. The Site is located within the National Forest.

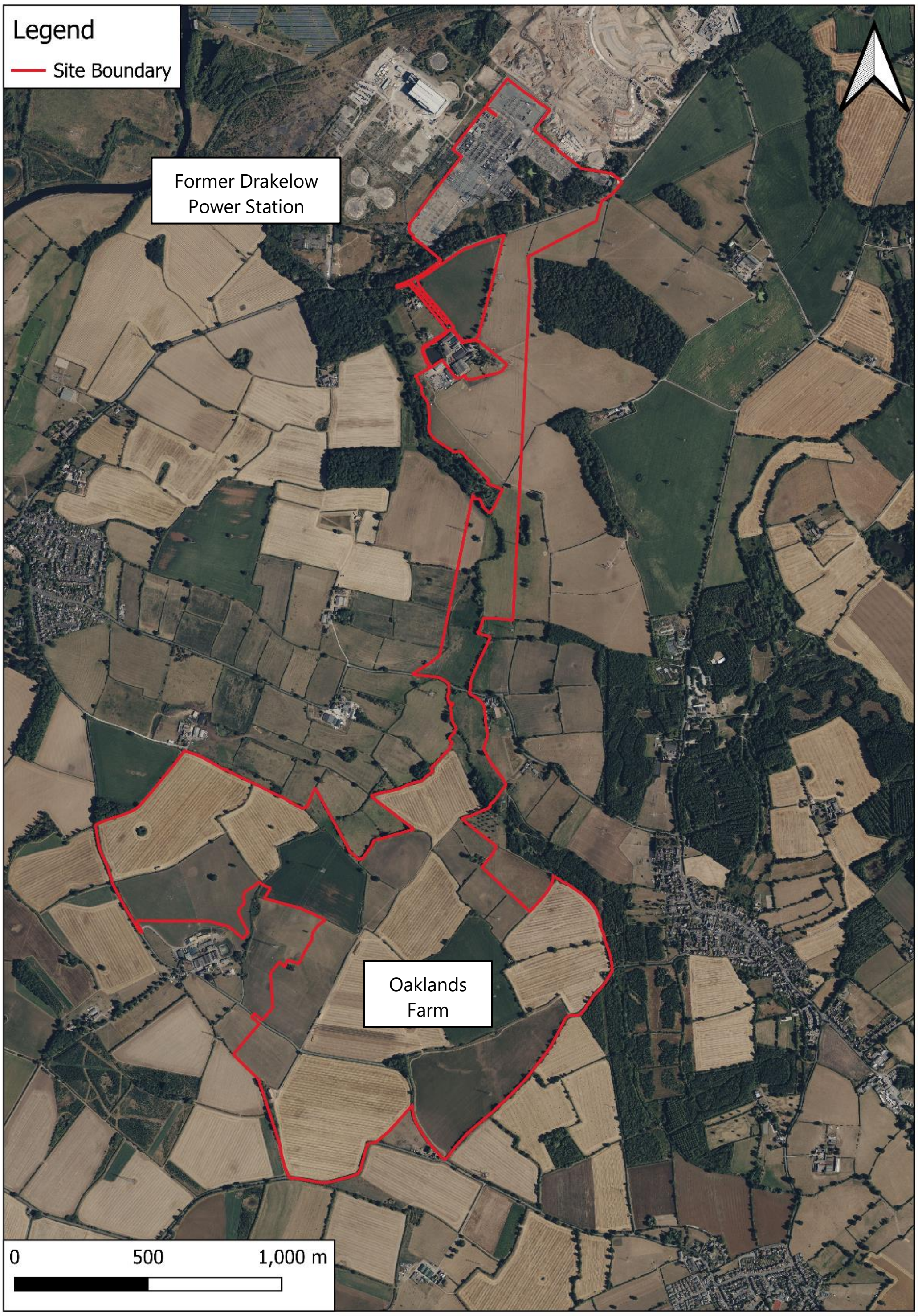
Immediately north of Rosliston Road is the land holding of Fairfields Farm and, further north, the Park Farm area up to Walton Road. Land use here comprises medium-large scale mixed arable and pastoral fields.

Two separate overhead electricity transmission lines run north to south through the Site, connecting into Drakelow substation. One 11kV overhead electricity distribution line also runs north into the Park Farm buildings.

Several adopted roads either border or run through the Site. These include:

- Coton Road, which connects Walton-on-Trent to Coton in the Elms and runs through the southern part of the Site.
- Catton Lane which links Rosliston to Lads Grave and borders the southeastern edge of the Site.
- Rosliston Road, which connects Walton-on-Trent to Rosliston and runs east-west through the Site.
- Walton Road, which connects Walton-on-Trent to the southwest with Stapenhill to the northeast, runs through the north of the Site along the southern boundary of the Drakelow Power Station area.

Figure 2-1 Site location



2.2. Topography

The Site is variable in elevation generally sloping down from an elevated high point of 92m above Ordnance Datum (m aOD) in the southern section of Site to around 64m aOD at the northern extent.

2.3. Proposed development

The Oaklands Farm Solar Park comprises a proposed solar farm with an associated battery energy storage facility ('the Proposed Development'). The Proposed Development would have a generating capacity of over 50MW and would be situated on 191 hectares of land at Oaklands Farm to the south-east of Walton-on-Trent and to the west of Rosliston in south Derbyshire.

The solar farm itself, comprising photovoltaic panel arrays, a central electricity substation and Battery Energy Storage System (BESS) together with access, landscaping and other works would be located on 135 hectares at Oaklands Farm currently in use for arable production and grazing. A high voltage underground electricity cable would then run through land at Fairfields Farm and Park Farm to the north to connect the solar farm to the national grid via an electricity substation located at the former Drakelow Power Station which sits south of Burton-upon-Trent.

As the Proposed Development would be an onshore generating station with a generating capacity of over 50MW an application for a Development Consent Order is being made under the Planning Act 2008 to the Planning Inspectorate, for determination by the Secretary of State for Energy Security and Net Zero.

2.4. Geology and hydrogeology

British Geological Survey (BGS) published geology indicates that the Site bedrock comprises the Edwalton Member (siltstone and very fine-grained sandstone). This is partly overlain by superficial deposits, comprising fluvioglacial diamicton in the south and some areas of alluvium in the north typically along watercourses through the Site. The soils close to the watercourse are described as slowly permeable, seasonally wet, with impeded drainage, whilst those away from the watercourse are described as "loamy and clayey soils with slightly impeded drainage".

The alluvium and glaciofluvial deposits beneath some areas of the Site are classified by the Environment Agency (EA) as a high vulnerability Secondary A Aquifers. These are defined by the EA as 'permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers'.

The Edwalton Member bedrock beneath the Site is classified as a Secondary B Aquifer. These are defined by the EA as 'predominantly lower permeability layers which may store and yield limited amounts of groundwater due to localised features such as fissures, thin permeable horizons and weathering'.

2.5. Hydrology

The vast majority of the Site is within the catchment of the River Trent with a very small area along the far southern edge of the southern-most parcel of the Site lies in the catchment of the River Mease, a tributary of the River Trent.

The majority of the Site drains to the River Trent via an unnamed tributary that flows through the Site. The unnamed tributary (an Ordinary Watercourse[1]) is shown on Ordnance Survey (OS) mapping to originate south of the village of Rosliston, and have its confluence with the Trent approximately 1.4km to the north-west of the Site).

A small tributary to the Ordinary Watercourse crosses the west of the Site from Oaklands Farm buildings to its confluence with the Ordinary Watercourse immediately upstream of Rosliston Road. The Ordinary Watercourse and its tributary are shown in Figure 2-2 along with LiDAR data of the Site.

2.5.1. Flood Defences

There are no formal flood defences throughout the area.

2.5.2. Greenfield Runoff

Greenfield Runoff has been calculated using the online Greenfield runoff rate estimation tool available on uksuds.com and the results are shown in Table 2-1 calculated for 1 ha in the centre of the Oaklands Farm parcel of land as a representative calculation. Further details are provided in Appendix B.

[1] Designation of 'main rivers': guidance to the Environment Agency, 2017. UK Gov. Available at: [Designation of 'main rivers': guidance to the Environment Agency - GOV.UK (www.gov.uk)](https://www.gov.uk)

Figure 2-2 Watercourses on Site and Lidar data

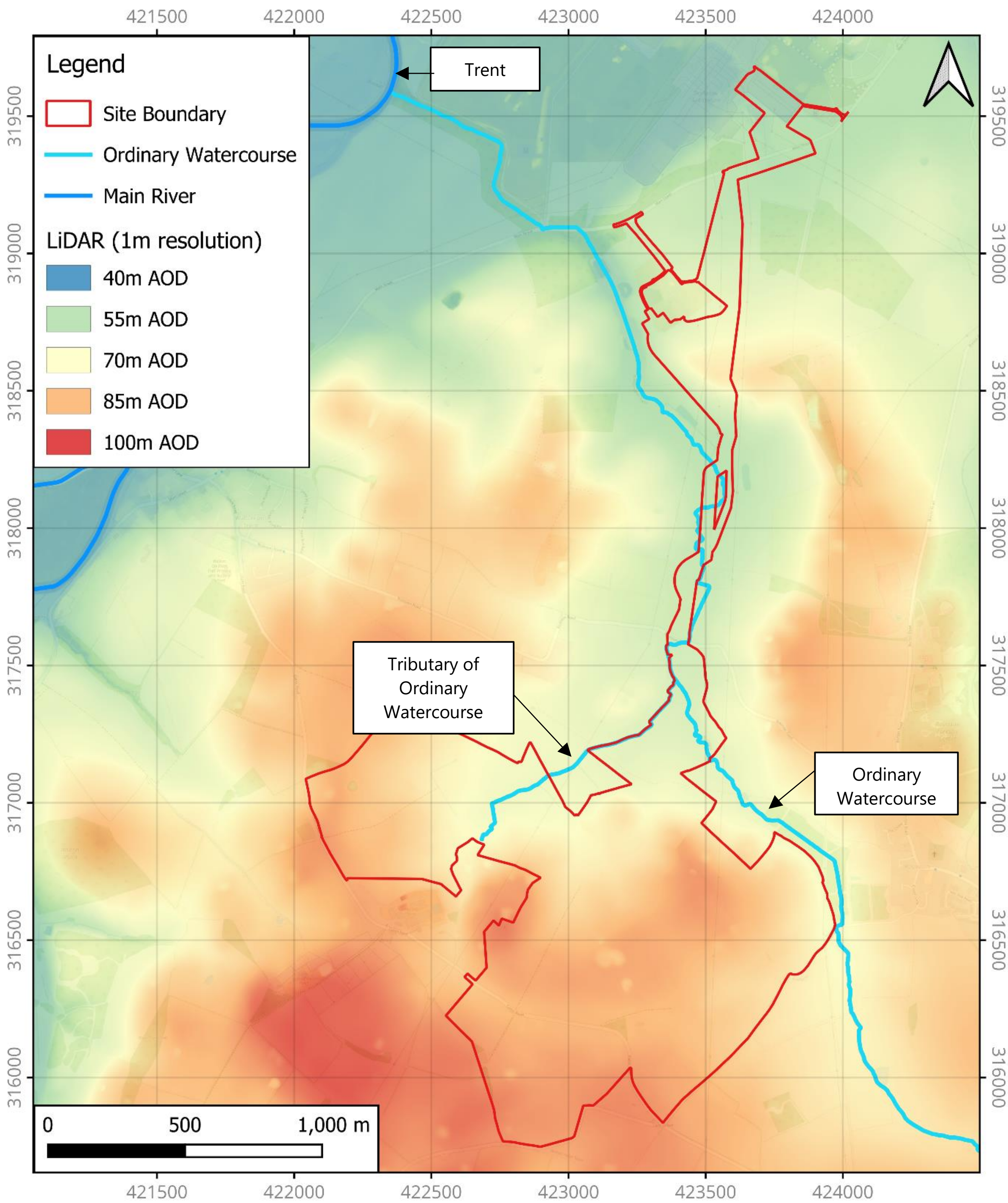


Table 2-1 Greenfield runoff rates per Hectare for the Site

Results using the IH124 method

Estimated site discharges

	My values
Qbar (l/s)	4.34
Greenfield runoff rates	
1 in 1 year (l/s)	3.6
1 in 30 years (l/s)	8.68
1 in 100 years (l/s)	11.15
1 in 200 years (l/s)	13.19

3. Planning Policy

3.1. National Flood Policy

National policy on planning and flood risk is provided by the National Planning Policy Framework (NPPF) and supplementary guidance. The acceptability of different types of development depends on its vulnerability to flooding and the flood zone in which the proposed development is to take place.

Flood risk has been mapped nationally by the EA to show the flood zones used in the NPPF.

3.1.1. Lead Local Flood Authority (LLFA) Flood Zones

Flood Zones 1, 2, 3a and 3b are defined by the LLFA in their Strategic Flood Risk Assessment (SFRA)[2] as:

- Flood Zone 1 refers to all areas that are considered to be at low risk of flooding and fall outside of Zones 2, 2a and 3b.
- Flood Zone 2 outlines an extreme flood of a 1 in 1,000-year flood event.
- Flood Zone 3a outlines a 1 in 100-year event and encompasses everything in Flood Zone 3 outside of Flood Zone 3b. Flood Zone 3a has been determined with an allowance for climate change adding a net increase of 20% over and above peak flows for a 1 in 100-year event. Where climate change modelling has not been undertaken, the Flood Zone 2 outline has been used as a proxy for Flood Zone 3a
- Flood Zone 3b outlines a 1 in 20-year floodplain or land within a Functional Floodplain (FFP) (defined by the 1 in 25-year outline where available, and if absent the 1 in 100-year outline).

It should be noted that national guidance has been updated since the SFRA was published in 2008 and Flood Zone 3b is now typically represented by the 1 in 30-year outline. In addition climate change, assessed per river basin, is not typically accounted for in the Flood Zone data. The EA is planning to publish an update to their 'Flood map for planning' in Spring 2025 which will incorporate future scenarios accounting for climate change.

3.2. Sequential / exceptions test

Solar farm developments are listed as essential infrastructure within Annex 3: Flood Risk vulnerability classification of the NPPF.

[2] South Derbyshire District Council Level 1 Strategic Flood Risk Assessment, 2008. Available at: https://www.southderbyshire.gov.uk/assets/attach/1788/level-1-strategic-flood-risk-assessment.pdf

Essential infrastructure, such as is proposed at the Site, is considered by the NPPF as acceptable in Flood Zones 1, 2 and 3a and 3b, but in 3a and 3b should be subject to an Exception Test as summarised in Figure 3-1.

The layout of the Proposed Development has been sequentially tested to steer infrastructure to areas of lowest flood risk within the Site, with all electrically sensitive infrastructure (solar panels, BESS and substation) within Flood Zone 1 and only buried cables and a short section of internal access track located in Flood Zones 2 and greater[3] (See Figure 4-1). The access track and underground cables within Flood Zone 2 and greater provides the most direct route to the grid connection at Drakelow Substation, minimising environmental impacts associated with construction.

Emergency access to the Site has also been provided along this same route, south off Rosliston Road towards Park Farm and Drakelow Substation as it provides the shortest route from the public highway. As the track will already be in place during construction, retaining this track would result in less impact than constructing a new emergency access route to the west of Site within Flood Zone 1. Therefore, development outside of Flood Zone 1, and most likely within Flood Zone 3a and 3b due to proximity of the watercourses is unavoidable to provide a cable connection and emergency access route for the Proposed Development.

The exception test for infrastructure within Flood Zone 3 (both 3a and 3b) requires that the infrastructure is designed and constructed to remain operational and safe for users in times of flood, result in no net loss of floodplain storage, not impede water flows and not increase flood risk elsewhere. The buried cables and short section of internal access track within Flood Zone 3 meet these requirements (with alternative access tracks within Flood Zone 1 useable during flood conditions) as no significant changes to land profiles are proposed.

[3] Due to the available Flood Zone data it has not been possible to distinguish between Flood Zone 2, 3a or 3b based on the publicly available data. This is further discussed in Section 4.2.1

Figure 3-1 Acceptability of development in Flood Zones

Flood Zones	Flood Risk Vulnerability Classification				
	Essential infrastructure	Highly vulnerable	More vulnerable	Less vulnerable	Water compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test required	✓	✓	✓
Zone 3a †	Exception Test required †	✗	Exception Test required	✓	✓
Zone 3b *	Exception Test required *	✗	✗	✗	✓ *

Key:

✓ Development is appropriate

✗ Development should not be permitted.

4. Definition of Flood Hazard

4.1. Historical records

There is no mapping of events for the Site in the EA historic flood dataset.

4.2. Sources of flooding

4.2.1. Fluvial and Tidal flooding

The flood risk arising from rivers and the sea is mapped nationally by the EA. The site is not subject to tidal flooding – therefore the risk of flooding from the sea has not been further assessed.

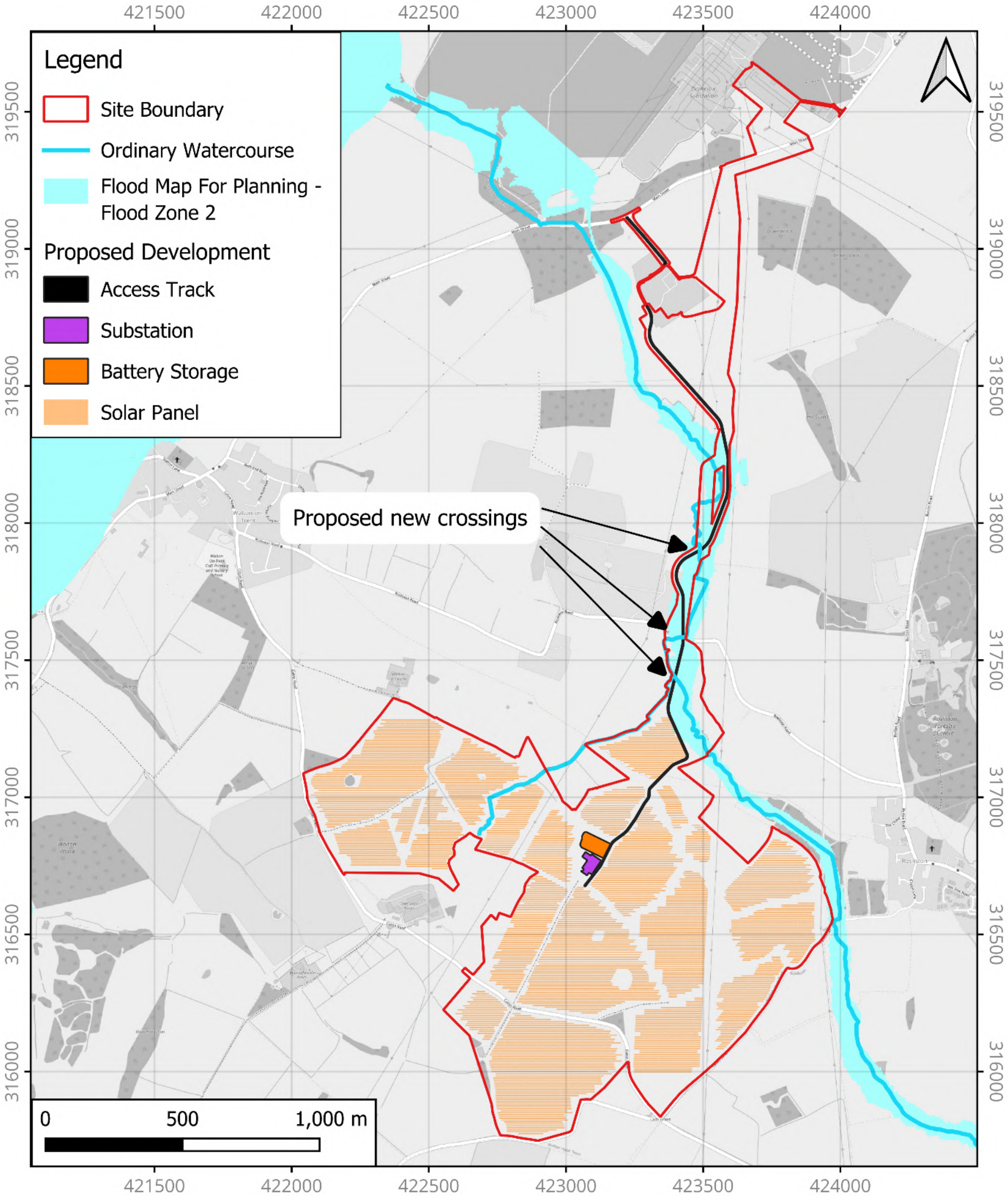
The only available flood modelling available from the Environment Agency are the Flood Zone extents which are based on coarse national modelling. The coarse national modelling has typically been undertaken for the 0.1% Annual Exceedance Probability (1 in 1000 year return period) and 1% AEP (1 in 100 year return period) events, without climate change. Based on the Flood Zone definition provided by the LLFA (Section 3.1.1):

- Flood Zone 2 has been based on the 0.1% AEP flood event (1 in 1000 year return period)
- Flood Zone 3a, in the absence of a modelled 1% AEP event (1 in 100 year return period) with climate change has been based on the 0.1% AEP flood event – hence the same as Flood Zone 2
- Flood Zone 3b, in the absence of a modelled 1 in 20 or 1 in 25 year return period has been based on the 1% AEP event (1 in 100 year return period).

The Flood Zone 3b extent is not published publicly, therefore the only flood extent data available is Flood Zone 2 and Flood Zone 3a, which are both based on the same modelled event – the 0.1% AEP (1 in 1000 year return period). Figure 4-1 details this extent (labelled as Flood Zone 2). The quality of the topography and modelling used to produce this map is low, as can be seen in areas where the flood risk fails to follow the line of the watercourse and provides an indication rather than an accurate description of the true flood risk areas.

The EA were asked to provide flood depths for the flood risk areas but do not have any more detailed information, reflecting the low priority given to modelling flood risk in an Ordinary Watercourse.

Figure 4-1 Flood Zone 2 for planning, with proposed development extents



The majority of the Site is in Flood Zone 1 (that is outside of the extent of Flood zone 2) with an annual risk of fluvial flooding less than 1 in 1,000 year return period (0.1% AEP event) and therefore at low risk of flooding, but parts bordering the Ordinary Watercourse are within Flood Zones 2 and greater (at risk of fluvial flooding greater than the 1 in 1000 year return period – 0.1 % AEP event).

The planning flood zones only consider the risk of flooding from main rivers and some of their tributaries, therefore, only the Ordinary Watercourse on-site has been considered within the flood zone mapping, and not the tributary that flows into the Ordinary Watercourse.

As the catchment area is small, parts are excluded from the fluvial flood mapping produced by the EA and it is likely that the surface water flood mapping in the next section provides a more accurate description of flood risk along all the watercourses as this mapping covers the whole country in a greater detail and is more recent. The small tributary that joins the Ordinary Watercourse are considered in subsequent sections of this report.

4.2.2. Surface water flooding

Surface water flooding arises from rainfall intensities exceeding the rate at which the ground can absorb the water and the local drainage system has capacity for. Excess water will flow over the surface, generally following the topography but can also be diverted by walls and buildings and possibly directed preferentially along roadways. Surface water can collect in low areas and pond, causing localised flooding.

For a small watercourse where all the flood runoff is being generated locally the surface water flood maps give a more accurate representation of flood risk than the fluvial flood mapping.

Figure 4-2 shows modelled surface water flood extents for the 3.33% AEP, 1% AEP and 0.1% AEP events. This indicates a network of flow paths channelling excess water across the Site to the watercourse with some limited areas of ponding where surface water may collect before slowly infiltrating into the soil.

The likely depth of flooding in a medium risk event (1%) is shown in Figure 4-3 and indicates that outside of the river channel, these are less than 300mm.

4.2.3. Groundwater flooding

Groundwater flooding is caused when water held within porous strata rises to the land surface due to excess rainfall generally over a long time period.

The majority of the Site is underlain by a secondary B aquifer which is likely to hold very limited volumes of groundwater, and soils which are only slowly permeable. In areas where superficial deposits are present the volumes of groundwater will also be limited due to the limited extent of the deposit and these are also covered by slowly permeable soils.

Groundwater flooding is therefore considered a low risk on the Site.

Figure 4-2 Flood risk from surface water

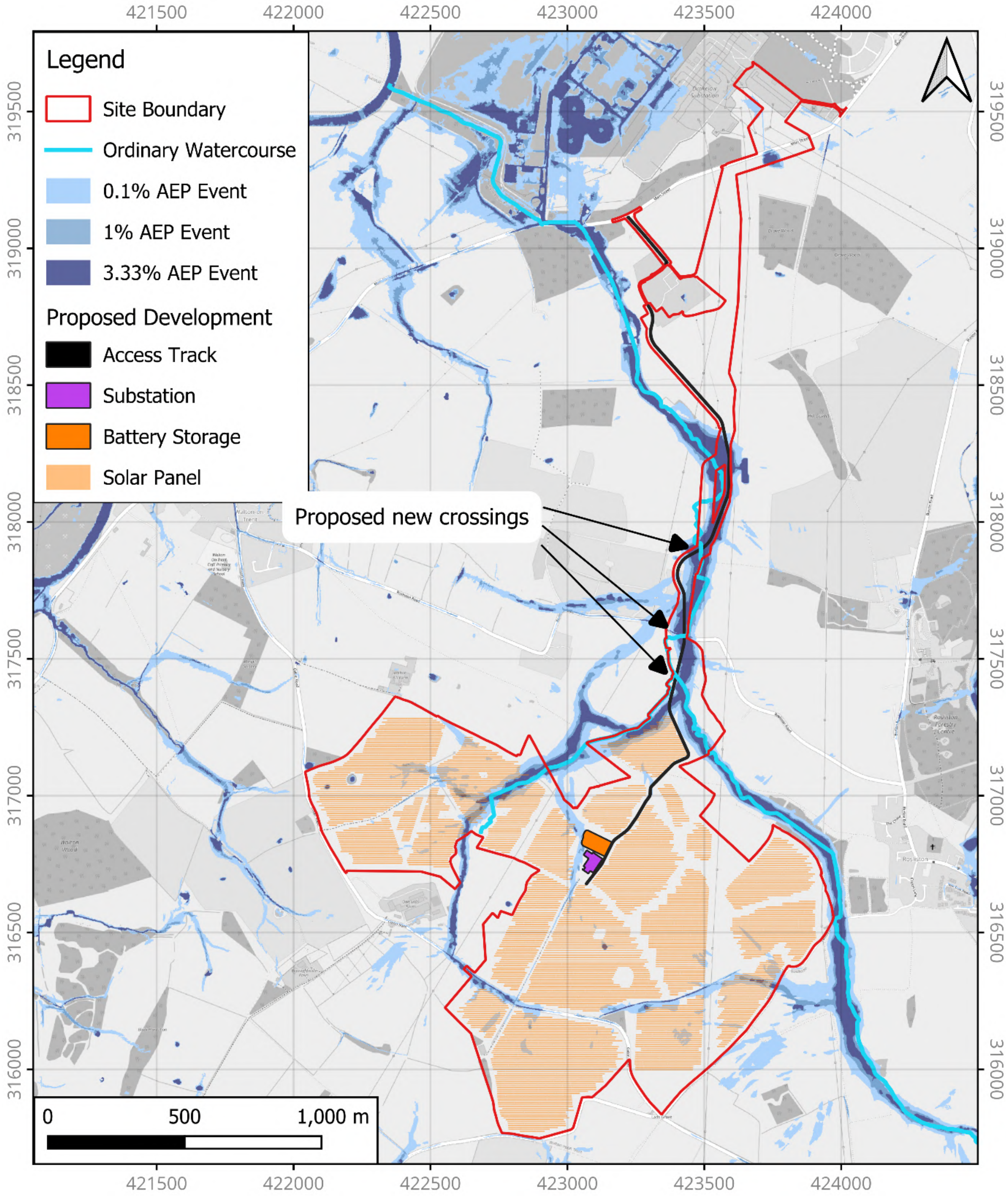
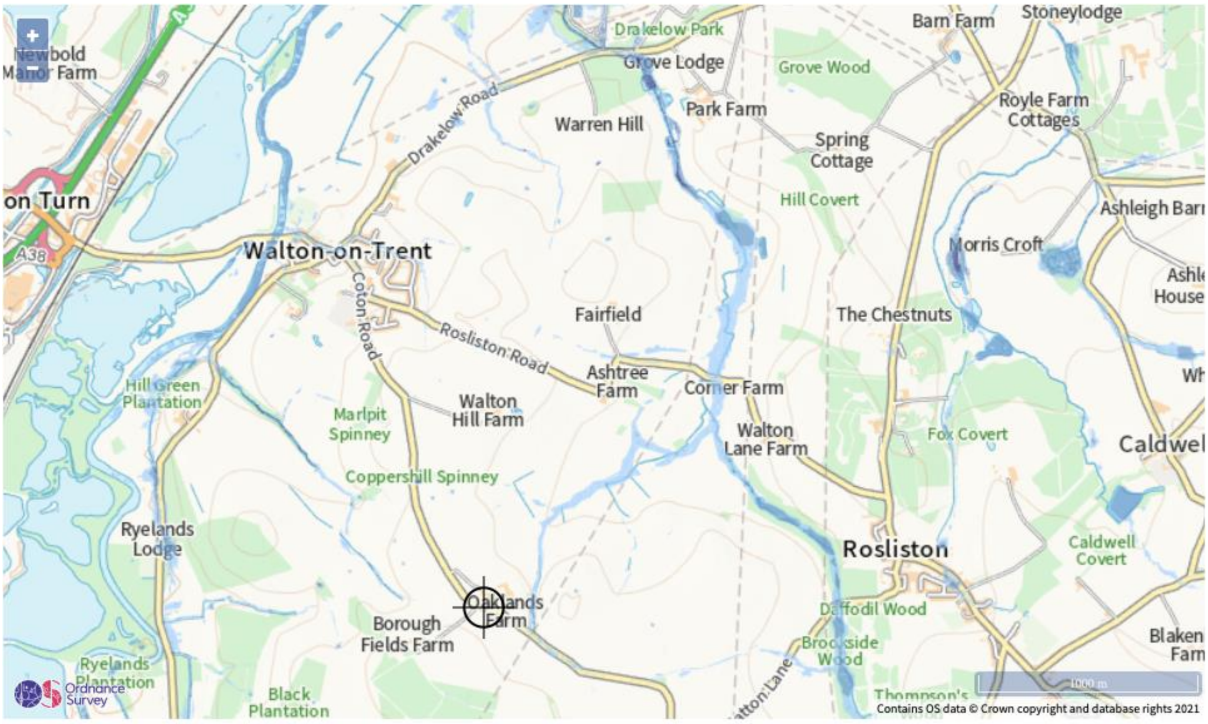


Figure 4-3 Depth of flooding in a 1% AEP surface water flood event



Surface water flood risk: water depth in a medium risk scenario

Flood depth (millimetres)

Over 900mm 300 to 900mm Below 300mm Location you selected

4.2.4. Catastrophic flooding

This source includes release of large volumes of stored water, such as in reservoirs and canals, due to catastrophic failure. The EA have mapped areas that are at risk of flooding from failure of large reservoirs and the Site is not shown to be potentially at risk from these sources.

There are no other identified large sources of stored water that may affect the Site and the risk of flooding from this source is considered to be negligible.

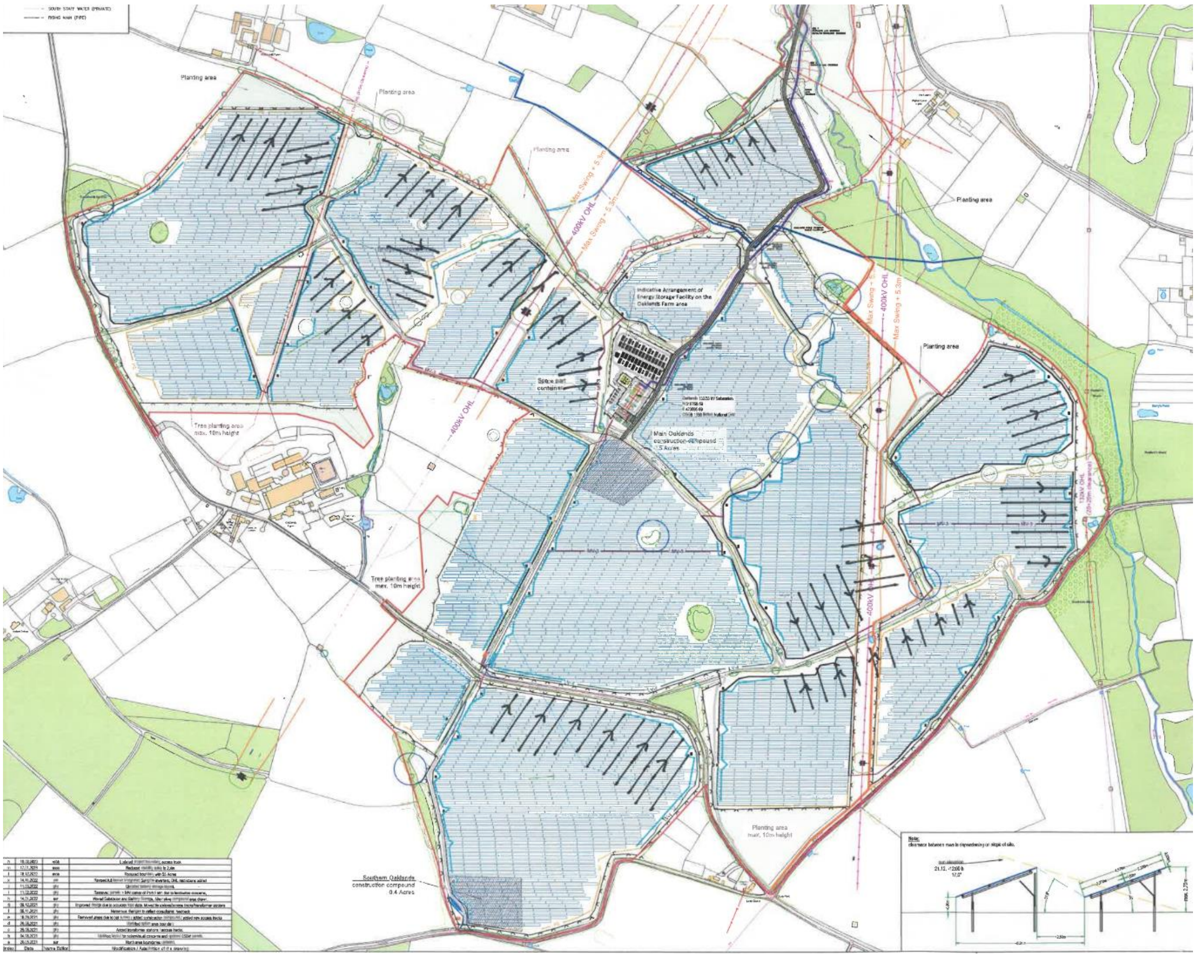
4.2.5. Land drains

Yellow Sub undertook a Site visit in June 2022 which was supplemented by a Site visit by Kernon Countryside Consultants Ltd in November 2022 to discuss and attempt to map field under drainage with the Site owner/ tenant farmer. This resulted in the map presented as Figure 4-4 which shows arrays of field drainage towards the lower margins of several fields. Whilst spacing of these is unknown, based on AHBD guidance[4] they are likely to be at least 40m apart.

[4] https://ahdb.org.uk/drainage

From a flood risk perspective, the presence of these underdrains represents a potential preferential pathway for surface run off and/ or shallow groundwater which may increase potential off-Site flood risk compared to true greenfield conditions.

Figure 4-4 Land drain locations



4.3. Climate Change

Climate will have a limited impact on flood risk over the lifetime of the Proposed Development. A worst case assessment[6] of the potential expansion of the 1% flood extent concluded it is unlikely to exceed the present day 0.1% flood extent.

Use of the 0.1% flood extent will therefore provide a conservative estimate of the future 1% flood, especially as the Site use is expected to be complete well within 100 years.

4.4. Overall Flood risk at the Site

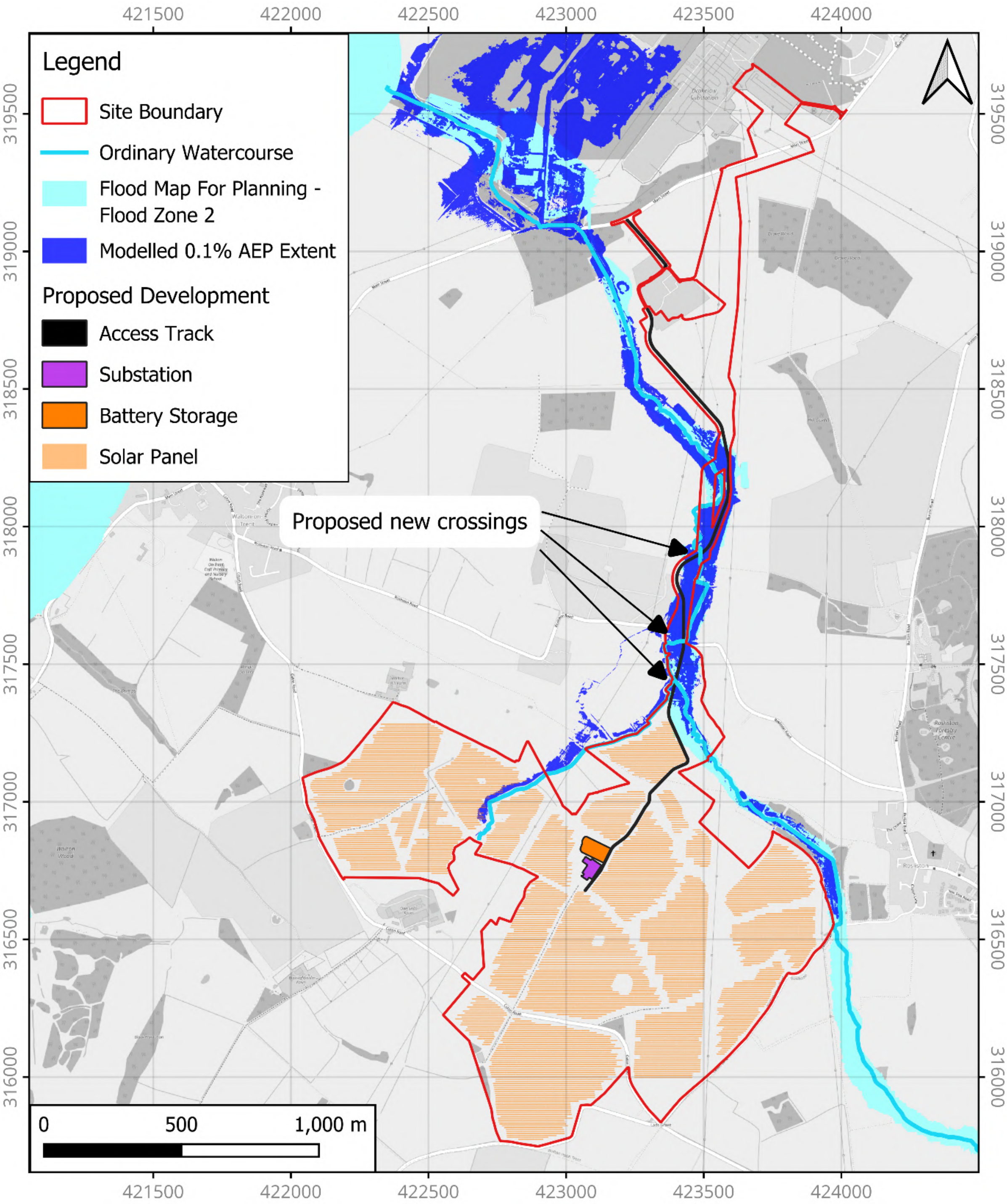
The above review has indicated that flood risk on the Site is restricted to the Ordinary Watercourse and a network of surface water flow paths, some of which are in channels and some overground or in isolated areas of ponding.

Outside of the watercourse channels the likely depth of flooding is less than 300 mm in a 1% AEP event. Flood risk from other sources considered is low or very low.

4.5. Hydraulic modelling

Due to the above indicated flood risk for the Ordinary Watercourse and tributary thereof running through the Site, a hydraulic model has been undertaken in agreement with the EA. The Hydraulic modelling report (ref: P20209_R5) is attached in Appendix G. This 1D-2D model has been analysed for 3.33%, 1%, 0.1% and 1% plus climate change events, baseline, sensitivity and proposed development model runs have been undertaken. Figure 4-5 provides a comparison of the modelled baseline 0.1% AEP flood extent against the existing Flood Zone 2. A more detailed discussion of the changes, and relevance to the Proposed Development is discussed in Section 5.1 and Section 8.1.

Figure 4-5 Comparison of modelled 0.1% AEP event against existing Flood Zone 2 extent



5. Detailed Development Proposal

5.1. Development Layout

The proposed development comprises solar panels, inverters, transformers, a substation and battery storage containers. There will be underground cabling connecting these elements and gravel tracks to provide access. Further details on each of these elements is provided below.

The proposed indicative layout is shown in the works plans found within Appendix 1.3 of the Environmental Statement, with the key features shown in Figure 4-1.

The solar panels are located outside of Flood Zone 2, shown in Figure 4-1, but not entirely out of the modelled surface water flood extent, which is more widespread. The detailed flood modelling (Appendix G) shows the modelled flood extents is in close proximity to the proposed infrastructure with flooding over the left (west) bank of the tributary into an area where panels are currently proposed for the 1% AEP event and larger. The maximum flood depth in this area is 0.15 m (0.1% AEP event). The bottom edge of the panels will typically sit 0.8 m above ground level, and therefore will be substantially above the maximum flood depth levels.

There will be a minimum 8 m easement between the top of any watercourse bank and any infrastructure (including panels, the substation and the BESS) to allow for maintenance access to river channels. Cable ducts will be located a minimum of 8 m away from the top of the bank of the watercourse, as far as possible. However, tracks may be constructed within 8 m as these do not prevent access to the watercourse.

Any watercourse crossings, or changes to existing crossings, may need Ordinary Watercourse Consent from the LLFA and should be designed so as not to impede flow or drainage. The LLFA were consulted in relation to the Proposed Development on the 8th June 2023.

5.2. Solar Panels

The solar panels are mounted on a frame supported by steel posts. The arrays are approximately 2.7 m in height, with the lower edge approx. 0.8 m above ground level (+/- 0.1m), which varies with local undulations in the ground surface. The frame foundations will consist of steel piles rammed/pushed into the ground, with a maximum piling depth of 2 m below ground level. Vegetation will be retained or re-sown under the panels which will then maintain a year-round cover of vegetation, unlike the current agricultural cropping regime which can result in bare ground exposed during winter and spring.

5.3. Access Tracks

Internal access tracks for construction purposes will be 3.5 – 6.0m wide and made up of 200mm of Type 1 compacted stone/gravel with a geotextile membrane or other surfacing solutions, and, where appropriate, may simply be mown grass corridors. The access tracks will have an edge gradient of 2.5° to facilitate surface runoff. Some of these temporary access tracks will be removed, whilst others remain for operations and maintenance following construction of the Proposed Development. A typical cross section is shown in Appendix C.

5.4. Watercourse Crossings

There are five proposed watercourse crossings of which at least two comprise existing crossings which may need to be reinforced for construction traffic. There are also three additional cable crossing which shall either be trenched across and reinstated or directionally drilled.

There will be three new access track crossings across the Ordinary Watercourse as locations shown in Figure 4-1. The crossings were proposed to be bottomless box culverts with an initial proposed width and height of 0.9 m and 1.0 m respectively. However, based on preliminary modelling results this resulted in an increase in flood levels to surrounding off-Site land. The hydraulic modelling (detailed in Appendix H) has therefore been used to refine these dimensions in order to limit potential off-Site impact. The following dimensions for the culverts are now proposed - a width of 1.5 m, height 0.8 m and a spill level consistent with the bank levels at the location of each crossing. The revised dimensions have reduced the adverse impact off-Site, however some impact still remains. This is further discussed in Appendix G and in Section 8.5.

5.5. Battery Storage

The BESS will comprise a fenced compound containing a series of batteries within containers, power conversion system units (which convert electricity between DC and AC during import or export processes), and an auxiliary transformer to provide necessary power for controls and monitoring systems. Details are provided in Appendix D. Note this drawing provides a general example and details of the base may not be included. Due to the potential risk of fire associated with these units, and the subsequent risk of contaminated firewater, the ground must be impermeable and water should be collected and contained within a storage area, which can be isolated if required.

5.6. Substation

The substation and welfare compound incorporates a number of features, including two substation transformers, Statcom Units, 132KV harmonic filter compound, substation control building, welfare unit, and fire water storage and deluge system. Details are provided in Appendix E. Note this drawing provides a general example and details of the base may not be included. Due to the potential risk of fire associated with these units, and the subsequent risk of contaminated firewater, at least part of the compound area must be impermeable and water should be collected and contained within a storage area, which can be isolated if required.

6. Site Drainage

6.1. Introduction

The following sections describe the outline SuDS Strategy for the proposed development with due regard to DEFRA's Non-Statutory Technical Standards for SuDS (DEFRA, 2015) which recommends the following hierarchy for the disposal of surface water:

- Discharge to ground via infiltration;
- Discharge to a surface water body;
- Discharge to a surface water sewer, local highway drain or another drainage system;
- Discharge to a combined sewer.

6.2. Greenfield runoff and permissible discharge rates

For greenfield sites, the peak runoff rate from the development should not exceed the peak greenfield runoff rate for the same event (DEFRA, 2015). Additionally, where reasonably practical, the runoff volume from the development in the 1% AEP 6-hour rainfall event should not exceed the greenfield runoff volume for the same event.

The existing greenfield runoff rates and volumes for the BESS (8,000m^2) and substation (6,000m^2) areas have been estimated and are summarised in Table 6-1. These were derived using the Revitalised Flood Hydrograph (ReFH2) model and a 6-hour storm duration assumed to calculate the volumes. The catchment descriptors at the Site were obtained from the FEH Webservice.

6.2.1. Climate change

The potential increase in rainfall intensity due to climate change needs to be considered when designing drainage strategies. The recommended allowances for rainfall intensity in the Adur and Ouse Management Catchment are included in Table 6-2.

The Proposed Development has a design life of 40 years, assuming development is completed in the next 5 years the Site will be in use until the 2060s. Therefore, based on the EA guidance for climate change allowances in flood risk assessments (Environment Agency, 2022), the central allowance for the 2070's epoch should be used (see Table 6-2).

Table 6-1 Greenfield runoff rates and volumes for BESS and substation areas

Flood event AEP	Runoff rate (l/s)		Runoff volume (m^3)	
	BESS	Substation	BESS	Substation
50% (1 in 2)	3.92	1.83	64	32
10% (1 in 10)	6.71	3.06	109	53
3.3% (1 in 30)	9.27	4.17	153	73
1% (1 in 100)	13.70	6.03	229	108
1% + 25% climate change	18.34	7.83	307	142

Table 6-2 Climate change allowances for rainfall in the Adur and Ouse Management Catchment

Epoch	Central allowance	Upper end allowance
3.3% AEP (1 in 30)		
2050s	20%	35%
2070s	25%	35%
1% AEP (1 in 100)		
2050s	20%	40%
2070s	25%	40%

6.3. Attenuation storage volumes

In order to achieve the above discharge rates within the BESS and substation areas, attenuation storage will be required. The estimated storage volumes are shown in Table 6-3.

These storage volumes were derived by calculating the flow exceeding the peak greenfield runoff rate for the 1% AEP event.

ReFH2 software has been used to calculate flow hydrographs for a 1% AEP + 25% storm event using a range of storm durations. Catchment descriptors at the site were obtained from the FEH Webservice. An imperviousness factor of 1.0 and 0.2 have been applied for the BESS and substation respectively, no allowance for urban creep has been applied as the hardstanding areas are unlikely to expand.

Volumes were then calculated from the flow exceeding the peak greenfield runoff rate for each storm duration, and the maximum value taken. An additional allowance of 25% has been applied to the volumes as recommended in the SuDS manual (CIRIA, 2015).

Table 6-3 Attenuation volumes for BESS and substation areas

Flood event AEP	BESS (m^3)	Substation (m^3)
1% + 25% climate change	442	66

6.4. Runoff destination and proposed SuDS design

The majority of the Site consists of solar panels mounted on a metal frame, underlain with vegetation. For these areas, no formal surface water collection system is proposed. The BESS and substation pose a theoretical risk of fire, with the potential of contaminant mobilisation due to the chemicals within the electrical units and/or firefighting fluids. Therefore, the surface water system has been designed with an automated pollution control valve (linked to the fire detection system) such that surface water runoff will not be discharged during a fire event in these areas, preventing it from leaving the locality and allowing the potential contaminants to be removed/ treated.

As detailed in the Environmental Statement, a Soil Management Plan will be compiled for the Proposed Development. The purpose of this document will be to demonstrate how damage to soil horizons and ground cover will be mitigated and remediated during and after construction and for future decommissioning. Detailed measures to manage runoff from the various areas in the proposed development are provided below.

6.4.1. Solar Panels

In these areas of the Site rainfall will be allowed to percolate into the underlying soil as occurs at present. This includes rain falling on the solar panels and the supporting infrastructure, which will be drained to ground.

The solar arrays contain frequent gaps up and along the arrays, to allow the individual panels to manage thermal expansion along the array, which are fundamental for thermal movement. These gaps allow rainwater to disperse through the array and avoid concentrated flows landing on the ground.

Runoff from the panels can therefore be intercepted and buffered by the vegetation growing underneath the panels and retained prior to infiltration as with the greenfield situation. The impact of the panels on runoff is therefore likely to be positive, as rainfall compaction of bare ground will be eradicated and soakage into the soil will be feasible throughout the year.

Overall runoff will be reduced as the vegetation will be in place all year round and the underlying soil will not be left bare or compacted by agricultural activities.

A typical example is shown in Figure 6.1. This example site is near Frome in Somerset and sited on mudstone bedrock, with soils described as "slowly permeable seasonally wet slightly loamy and clayey soils with impeded drainage", i.e. the same as at the proposed development. Rainfall is allowed to fall onto the ground beneath: there is no evidence of erosion or runoff from underneath the panels and sufficient vegetation occurs to prevent bare ground developing.

6.4.2. Access tracks

All field access tracks will be constructed of compacted gravel such that they are permeable to negate impacts to drainage. Each track shall be designed with a fall to a gravel filled longitudinal trench into which excess water will flow. These trenches will act as attenuation and treatment prior to infiltration.

Figure 6-1 Drainage of solar panels onto grass

6.4.3. BESS and substation

The proposed development will include inverter units and a main substation. Inverter units will be within cabins on concrete pads within the site, which will be connected to cables in backfilled trenches. Each inverter is positioned on legs raised above the base.

The site will also incorporate a BESS to satisfy the modern needs of solar farms. The BESS is made up of batteries in sealed shipping type containers, supported on legs on pads. A typical example is shown in Figure 6-2.

Figure 6-2 Typical battery containers used on a solar farm

Due to the potential risk of fire associated with these units, and the subsequent risk of contaminated firewater, infiltration is not considered a suitable SuDS measure in these areas. Instead, water should be collected and contained within a storage area, which can be isolated if required.

It is proposed that underground storage areas are created beneath the BESS and substation areas which are filled with single sized granular material. The BESS and substation will be surrounded by suitable bunds to separate runoff from adjacent areas and the storage provision lined to prevent the potential leaching of contaminants in the event of a fire. Under normal circumstances the storage areas will be drained to the northeast towards the existing drainage channel, approximately 300m north-west of the BESS/substation. However, automated pollution control devices (valves) will be fitted to the tank outfall to prevent the release of water when a fire is detected on Site.

Sizing of storage areas has been undertaken based upon a 100yr + 25% climate change scenario (see Table 6-4). This assumes that water would be released at a rate equivalent to the existing greenfield runoff rate of 13.7l/s and 6l/s at the BESS and substation respectively.

Additionally, storage volumes have been calculated to replicate a fire situation where no water is released from the storage areas. A 24hr storm duration has been used, based upon the assumption that this is the longest time period required for a tanker to arrive at the Site and pump out potentially contaminated water.

Table 6-4 shows the resulting volumes for a range of storm durations, including an additional 300m3 and 100m3 volume for firefighting water at the BESS and substation respectively.

The joint probability of a fire occurring simultaneously with a 1% AEP storm is very remote, therefore a 10% AEP event has been chosen to determine the storage requirements during a fire scenario. The fire scenario attenuation requirements are significantly larger than the normal conditions scenario, despite a smaller storm being considered. At the BESS the storage required to contain a 10% AEP + CC event during a fire scenario is 910m^3, whilst only 442m^3 is required for a 1% AEP + CC under normal conditions. Therefore the storage areas will generally be underutilised during normal conditions.

Table 6-4 Attenuation volumes for BESS and substation areas during a fire event

Flood event AEP plus fire	BESS (m^3)	Substation (m^3)
50% AEP+ 25% CC	753	314
10% AEP+ 25% CC	910	423
3.3% AEP+ 25% CC	1082	514
2% AEP+ 25% CC	1186	570
1% AEP+ 25% CC	1342	652

A preliminary design of the storage areas has been undertaken. It's assumed that the storage areas would be located beneath the BESS and substation areas, which are bunded and lined to prevent infiltration and filled with single sized granular material to provide attenuation. The amount of storage offered would be dependent upon the subgrade depth and Site gradient. The use of permeable surfacing should be considered at the detailed design stage.

An approximate area of 8,000m^2 and 6,000m^2 are available at the BESS and substation areas respectively. By creating storage areas with a depth of 0.4m and 0.3m and a void ratio of 30% within the granular fill material, a storage volume of 960m^3 and 540m^3 would be created at the BESS and substation respectively. Table 6-5 summarises the attenuation area dimensions. A layout of the proposed SuDS scheme is included in Appendix G.

Table 6-5 Preliminary sizing of BESS and substation attenuation areas

	BESS	Substation
Area (m^2)	8,000	6,000
Depth (m)	0.4	0.3
Volume (m^3)	960	540

6.4.4. Land drains

As noted in Section 4.2.5, parts of the Site are underdrained which may present a preferential flow path for surface water run off and/ or shallow groundwater under current, baseline conditions. Consultation with the EA, DCC and SDDC has recognised that land drains, where present, may be damaged by the proposed development including actions such as piling and trenching for cabling. Under the baseline conditions, the presence of underdrains, may, increase potential flood risk to off-Site receptors compared to true greenfield conditions. It is therefore considered that damage caused to land drains will act to 'slow the flow' and return affected areas back to or closer to greenfield conditions, encouraging surface water to infiltrate to the ground and thereby reduce the potential flood risk to off-Site receptors.

As shown in Figure 4-4, the underdrainage is shown to follow the natural topography of each field to an existing boundary ditch. Should a field drain be damaged, whilst surface water run-off will be slowed to greenfield rates, be filtered by the permanent grass sward and encouraged to infiltrate, should excess flows be generated, these will continue to follow the natural Site topography and ultimately discharge into the same existing ditch. Therefore, it can be concluded that, from a hydrological/ drainage perspective, localised damage to land drains may be viewed as a beneficial impact compared to the baseline conditions, slowing down the flow but maintaining the same overall flow path to the local boundary ditch network.

It is acknowledged that damage to land drains may impact the suitability of the soils for agricultural purposes which is covered within the Agricultural Land Assessment, outline CEMP and outline Decommissioning Plan.

6.5. exceedance

Storage at the BESS and substation areas has been provided for the 1% AEP + 25% climate change, as well as for the 10% AEP + 25% climate change under a fire scenario with no release of water. Storm events in excess of these will result in the storage areas being exceeded, the exceedance flows will be designed to follow the existing preferential surface water flow route towards the drain to the northeast. The flow route is detailed in Appendix F. A more detailed analysis of exceedance flows can be undertaken once the Site elevations and storage area design has been finalised and modelled.

6.6. Water quality

SuDS techniques can be used to effectively manage the quality of surface water flowing across a site. Different methods can be used to intercept pollutants and allow them to degrade or be stored in-situ without impacting the quality of water further downstream. Frequent and short duration rainfall events are those that are most loaded with potential contaminants (silts, fines, heavy metals and various organic and inorganic contaminants). Therefore, the first 5mm to 10mm of rainfall (i.e. the 'first flush') should be adequately treated using SuDS.

The proposed development will include low traffic roads, which the CIRIA SuDS manual categorises as presenting a low hazard rating. Table 6-6 shows the pollution hazard indices for each land use.

Table 6-6 Pollution hazard indices

Land use	Pollution hazard level	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Low traffic roads	Low	0.5	0.4	0.4

Where practical, runoff will be directed to permeable surfacing. Within the BESS and substation areas, water will be contained within a storage area prior to discharging to a nearby drainage channel. Table 6-7 below demonstrates that these SuDS methods provide sufficient treatment.

Table 6-7 SuDS mitigation indices

Type of SuDS	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Permeable surfacing	0.7	0.6	0.7
Detention basin	0.5	0.5	0.6

7. Maintenance schedules

7.1. Overview

This section outlines the maintenance and management schedules for the proposed stormwater drainage system. The schedules have been formulated in line with guidelines contained within the CIRIA SuDS Manual (C753) (Woods Ballard, et al., 2015).

There are three categories of maintenance activities (including inspections and monitoring) referred to in this report:

- **Regular maintenance** – tasks which are required to be undertaken on a weekly or monthly basis, or as required.
- **Occasional maintenance** – tasks which are required to be undertaken periodically, typically at intervals of 3 months or more.
- **Remedial maintenance** – tasks which are not required on a regular basis but are done when necessary.

This section is intended to give an overview of the operation and maintenance for the range of drainage features included within the surface water drainage strategy and in relation to typical/ standard details only.

Maintenance schedules for the proposed SuDS components are provided in the following tables. These schedules are not exhaustive and should be reassessed at regular intervals to determine if any additional maintenance requirements are required to preserve the performance and condition of the drainage system.

7.2. Maintenance schedules

7.2.1. Pipes and manholes

A typical schedule of maintenance activities for pipes and manholes is included in Table 7-1.

Table 7-1 Pipes and manholes

Maintenance schedule	Required action	Frequency
Regular maintenance	Remove any accumulation of silt, sediment, leaves and debris etc	Monthly, or as required
	Inspect for evidence of poor operation	Monthly (during the first year), then half yearly
Occasional maintenance	High pressure water jet removal of silt build-up and avoid blockages, particularly at bends or changes in direction	Six monthly, or as required
	Remove or control tree roots where they are encroaching pipe runs, using recommended methods	As required
Remedial actions	Clear pipework and gully grates of blockages	As required
	Replace any damaged or failed pipes, gullies or manholes	As required

7.2.2. Permeable paving

A typical schedule of maintenance activities for permeable paving is included in Table 7-2.

Table 7-2 Permeable paving

Maintenance schedule	Required action	Frequency
Occasional maintenance	Initial inspection	Monthly for three months after installation
	Inspect for evidence of poor operation and/or weed growth – if required, take remedial action	Three-monthly, 48 hours after large storm in first six months
	Inspect silt accumulation rates and establish appropriate jetting frequencies	Annually
	Monitor inspection chambers	Annually
	Stabilise and mow contributing and adjacent areas	As required

Maintenance schedule	Required action	Frequency
	Removal of weeds or management using glyphosate applied directly into the weeds by an applicator rather than spraying	As required – once per year on less frequently used pavements
Remedial actions	Remediate any landscaping which, through vegetation maintenance or soil slip, has been raised to within 50mm of the level of the paving	As required
	Remedial work to any depressions or ruts considered detrimental to the structural performance or a hazard to users.	As required
	Rehabilitation of surface.	As required

7.2.3. Granular Sub-base

A typical schedule of maintenance activities is included in Table 7-3.

Table 7-3 Granular sub-base

Maintenance schedule	Required action	Frequency
Regular maintenance	Inspect/ check all inlets, outlets, inspection/access chamber, vents to ensure that they are in good condition and operating as designed	Monthly for 3 months, then annually
	Inspect silt traps and note rate of sediment accumulation	Monthly in the first year and then annually
	Inspect and identify any areas that are not operating correctly. If required take remedial action	Monthly for 3 months, then annually
Occasional maintenance	Remove sediment from pre-treatment structures	Annually, or as required
Remedial actions	Repair/rehabilitate inlets, outlets, overflows, inspection/access chamber and vents	As required

7.2.4. Flow controls

A typical schedule of maintenance activities for flow control devices is included in Table 7-4.

Table 7-4 Flow control devices

Maintenance schedule	Required action	Frequency
Regular maintenance	Inspect/check pipework to ensure that the flow control is in good condition and operating as designed	Monthly
	Inspect for evidence of poor operation	Monthly, or as required
Occasional maintenance	High pressure water jet removal of silt build-up	Six monthly, or as required
Remedial actions	Replace the flow control if it becomes damaged	As required
	Clear pipework of blockages	As required

7.3. Inspections

In conjunction with the above maintenance schedules and in accordance with both the CEMP (Construction Phase) and management plan (Operational Phase), regular inspections of all stormwater drainage equipment and solar panel arrays will be undertaken to identify potential problems as early as possible. Routine inspections will be undertaken each quarter, with all array foundations, swales, ditches, drains, culverts and track crossing inspected for blockages and/or debris. All blockages are to be cleared immediately.

Swales, ditches, drains, culverts, track crossings and, where relevant, array foundations within Flood Zone 2/ 3 on-Site will also be inspected for blockages and/or debris after a storm event.

8. Flood Risk Management Measures

8.1. Mitigation for on-Site flooding

Outside of the fluvial flood zone 2, the area is not at significant flood risk and climate change will not alter this for the expected lifetime of the Proposed Development.

Due to potential impacts and the uncertainty in published flood risk mapping, at the request of the EA, 1D-2D hydraulic modelling has been undertaken (see Appendix H) for the likely flood extents and depths along the Ordinary Watercourse and tributary thereof running through the Site. The modelling has shown the following:

- The 0.1% AEP event has nearly identical peak flows with the 1% AEP with upper end climate change (51% increase) – therefore the 0.1% AEP event can be considered to be the largest event required to be assessed;
- The modelling has shown flood extents for the area to the east of the proposed panels (on the left/west bank of the Ordinary Watercourse) are substantially reduced in comparison to the existing EA flood zones. Proposed panels adjacent to this location are now outside of the largest modelled event (0.1% AEP); and,
- Baseline modelling has shown flooding for the 1% AEP event and larger over the left / west bank of the tributary in an area where panels are proposed however the maximum flood depth is 0.15 m whereas the panels are proposed to sit approximately 0.8 m above ground level.

The solar panels are raised approximately 0.8 m above ground level and therefore unlikely to be affected by this limited flooding on the left/west bank of the tributary, should it occur. No additional specific mitigation is therefore required to protect them.

Inverters, transformers and substations are not proposed to be sited within areas of fluvial flood risk and should not be sited within the surface water flood risk areas or, if this is unavoidable, vulnerable parts of these structures should be raised at least 0.3 m above the ground level. It is proposed to raise them by 0.6 m above ground level on piers as a precaution and this approach will also avoid any potential blockage or diversion of surface flow paths.

Gravel tracks will not be raised above the ground surface in the surface water flood risk areas to avoid diverting flow paths.

The Site will not be normally occupied. Maintenance will be timetabled and restricted to daylight hours. Maintenance visits should be cancelled, and any on-Site personnel withdrawn on receipt of a flood warning.

All runoff from the proposed structures will be dealt with locally with source control measures and the Site will not generate extra runoff. Further mitigation for flood risk is not considered to be required but a construction phase surface water management plan should be developed within the CEMP to ensure flood risks and flood runoff are not increased during construction.

8.2. Flood Compensation Volume

Occupation of the flood storage areas by structures will be minimal (as pathways rather than storage areas) and the alternative routes will offer similar storage characteristics. Explicit compensation for lost storage is therefore not required.

Moving vulnerable structures away from surface water flow paths avoids this requirement entirely.

8.3. Safe Access and Exit

Whilst Rosliston Road, and the access tracks off it, are located within the fluvial flood risk area, alternative routes outside of the flood risk area are available such as via Coton Road. The local road network may be affected by flooding where it crosses the unnamed watercourse and by surface water, particularly Coton Road between Oaklands Farm and Lad's Grave. Flood depths along these routes are expected not to exceed 300 mm however, and they should remain passable with care.

8.4. Flood Warning

Flood warning is unlikely to be of use in the area as the catchment is mostly out of the flood risk area and the response of the small watercourses to rainfall could be very rapid. Nevertheless, the site operators should sign up for the flood alert service provided by the EA in order to avoid working on Site when flooding is possible and have measures in place to inform any personnel on Site of the need to close and evacuate. Further information is provided here:

https://flood-warning-information.service.gov.uk/warnings

8.5. Off-site Impacts

The proposed development will not change any land profiles, reduce flood storage volume, increase discharge runoff or impede surface water flows, and therefore with the exception of the three new watercourse crossings of the Ordinary Watercourse it is very unlikely to impact on flood risk elsewhere.

The flood modelling undertaken of the proposed development has shown localised impacts for all events from the proposed crossings with the largest impact indicated for the 3.33% event. Adverse impacts from the two upstream proposed crossings are almost entirely contained within the Site boundary. The third proposed most downstream crossing causes some flow to overtop the right / east bank and increase flood depths off-Site by a maximum of 0.11m along pre-existing flow path, up to 0.13m where the floodplain filters into a network of drainage ditches (i.e. within the ditches themselves) and into an existing pond where depths increase by up to 0.18m. There are no changes in flood extents in the vicinity of the pond for any event, indicating that the increase in flood depth does not cause overtopping of this feature.

The proposed development modelling has also shown significant areas where flood depths have been reduced for all events. The area of reduced flood risk off-Site is greater in size than the area at increased flood risk for all events. Over 90% of the impacted off-site land is modelled to have reduced flood depths compared to baseline for the 1% with climate change and 0.1% AEP events – hence the overall impact on flood levels off-site is beneficial.

Impacts for the 3.33% AEP event are shown in Figure 8-1, and a summary of total land adversely and beneficially impacted is provided in Table 8-1. Whilst the 1% AEP with climate change and 0.1% AEP events show the greatest maximum increase in flood depths – these are over a very small area, and therefore it is still considered that the 3.33% AEP event is where the largest adverse impact is shown. A full set of figures comparing the baseline and proposed flood depths are provided in Appendix H with the hydraulic modelling report.

Table 8-1 Summary of off-Site impacts from proposed development

Event	**Off site land at increased flood depths (m^2)**	**Off-site land at decreased flood depths (m^2)**	**Percentage of impacted land at decreased flood depths**	**Maximum increase in flood depths off-Site (m)**
3.33% AEP	14133	18465	57%	0.18 (Pond / drains) 0.11 (floodplain)
1% AEP	13914	29333	68%	0.10 (floodplain)
1% AEP + 30% CC	5588	54728	91%	0.12 (within watercourse) 0.06 (floodplain)
0.1% AEP	6092	127172	95%	0.16 (within watercourse) 0.05 (floodplain)

Flood extents are shown to overall decrease off-Site, with more land (approximately double) removed from the flood extent than added for all events modelled. Figure 8-2 shows the extent of the increase and decrease of flood extent for the 3.33% AEP event, and Table 8-2 summarises the areas for each of the events.

As can be seen from Figure 8-2 the areas that are impacted by an increase in flood extent consist entirely of farmland or areas of woodland/ vegetation, with no properties impacted or close to being impacted.

Table 8-2 Summary of change in flood extent outside of Site boundary

	3.33% AEP	1% AEP	1% AEP + 30% CC	0.1% AEP
Decreased flood extent (off-Site) m^2	8,291	4,226	5,748	4,619
Increased flood extent (off-Site) m^2	3,640	3,715	2,301	2,471

Figure 8-1 Impact of Proposed Development on flood levels for the 3.33% AEP event

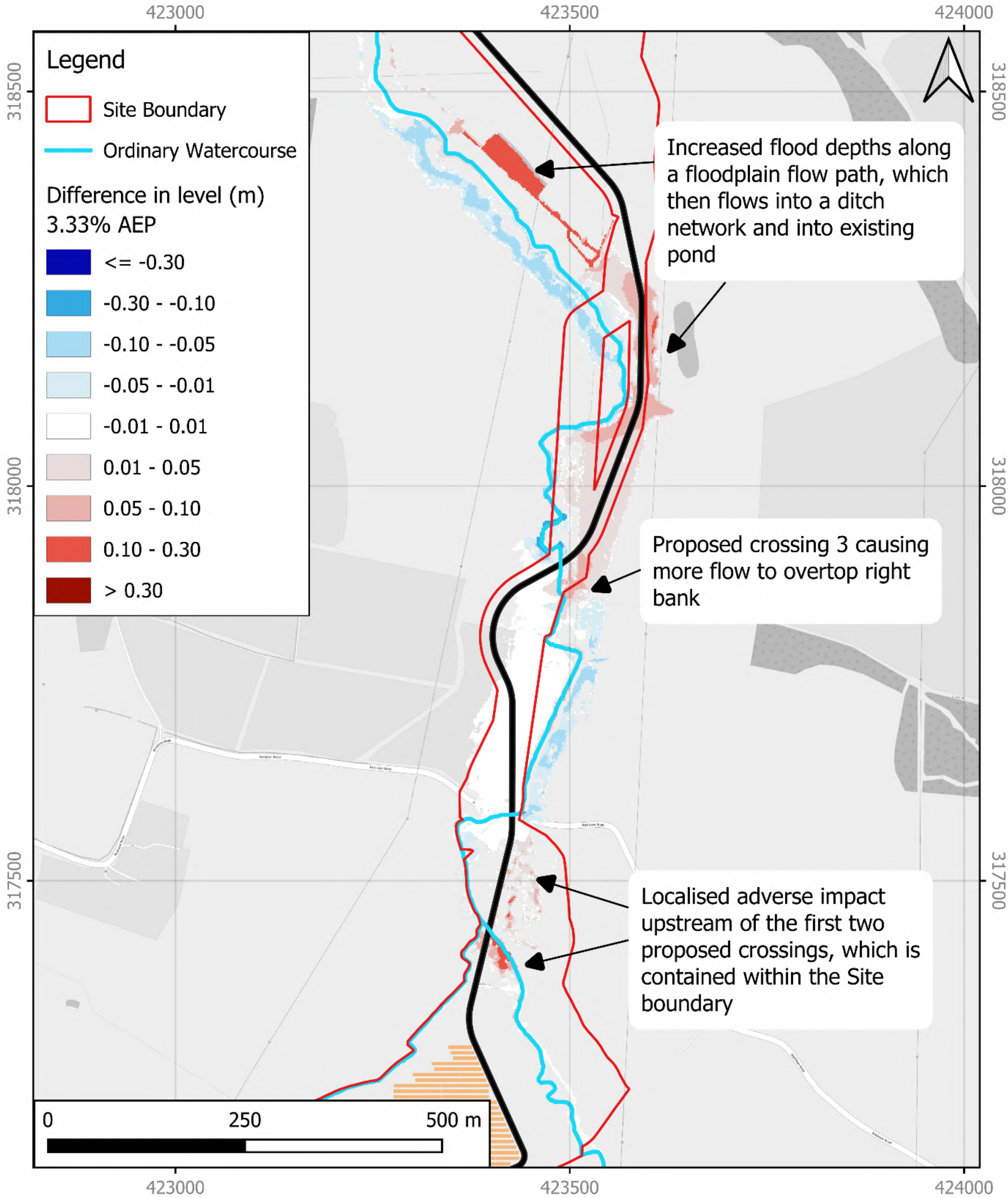
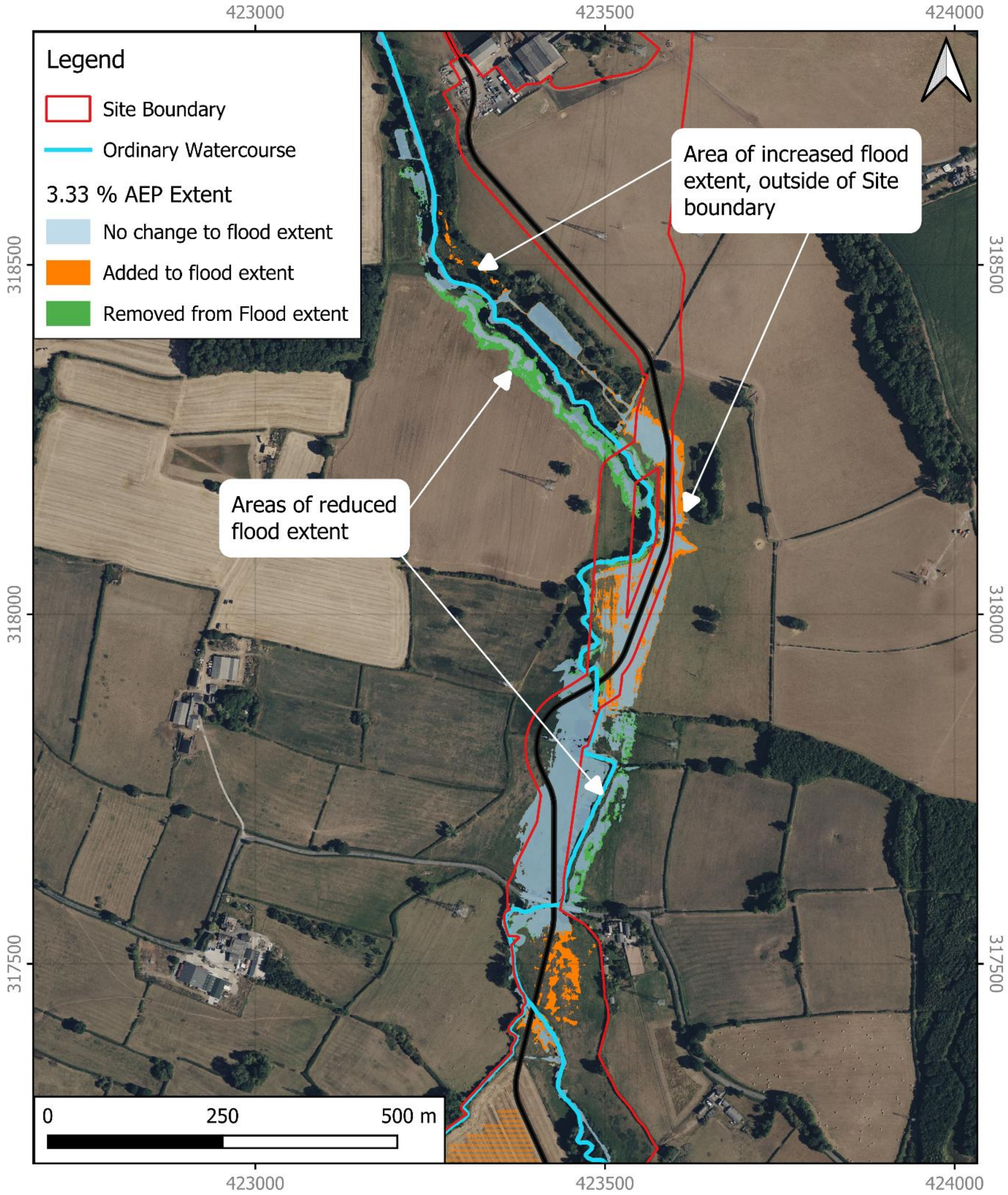


Figure 8-2 Impact of Proposed Development on flood extents for the 3.33% AEP event



Appendix A Report conditions

Report Conditions

This report has been prepared by Yellow Sub Geo Ltd. (Yellow Sub Geo) in its professional capacity as soil and groundwater specialists, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client and is provided by Yellow Sub Geo solely for the internal use of its client.

The advice and opinions in this report should be read and relied on only in the context of the report, taking account of the terms of reference agreed with the client. The findings are based on the information made available to Yellow Sub Geo at the date of the report (and will have been assumed to be correct) and on current UK standards, codes, technology, and practices as at that time. They do not purport to include any manner of legal advice or opinion. New information or changes in conditions and regulatory requirements may occur in future, which will change the conclusions presented here.

Where necessary and appropriate, the report represents and relies on published information from third party, publicly and commercially available sources which is used in good faith of its accuracy and efficacy. Yellow Sub Geo cannot accept responsibility for the work of others.

Site investigation results necessarily rely on tests and observations within exploratory holes only. The inherent variation in ground conditions mean that the results may not be representative of ground conditions between exploratory holes. Yellow Sub Geo take no responsibility for variation in ground conditions between exploratory positions.

This report is confidential to the client. The client may submit the report to regulatory bodies, where appropriate. Should the client wish to release this report to any other third party for that party's reliance, Yellow Sub Geo may, by prior written agreement, agree to such release, if it is acknowledged that Yellow Sub Geo accepts no responsibility of any nature to any third party to whom this report or any part thereof is made known. Yellow Sub Geo accepts no responsibility for any loss or damage incurred as a result, and the third party does not acquire any rights whatsoever, contractual, or otherwise, against Yellow Sub Geo except as expressly agreed with Yellow Sub Geo in writing. Yellow Sub Geo reserves the right to withhold and/ or negotiate the transference of reliance on this report, subject to legal and commercial review.

Appendix B Greenfield Runoff Calculations

HR Wallingford
Working with water

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Bob Sargent
Site name:	Oaklands Farm
Site location:	Rosliston

Site Details

Latitude:	52.75009° N
Longitude:	1.65737° W
Reference:	894148917
Date:	Dec 09 2021 17:08

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

Runoff estimation approach IH124

Site characteristics

Total site area (ha): 1

Methodology

Q_{BAR} estimation method: Calculate from SPR and SAAR

SPR estimation method: Calculate from SOIL type

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

Hydrological characteristics	Default	Edited
SAAR (mm):	639	639
Hydrological region:	4	4
Growth curve factor 1 year:	0.83	0.83
Growth curve factor 30 years:	2	2
Growth curve factor 100 years:	2.57	2.57

Notes

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

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

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

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

(3) Is SPR/SPRHOST ≤ 0.3?

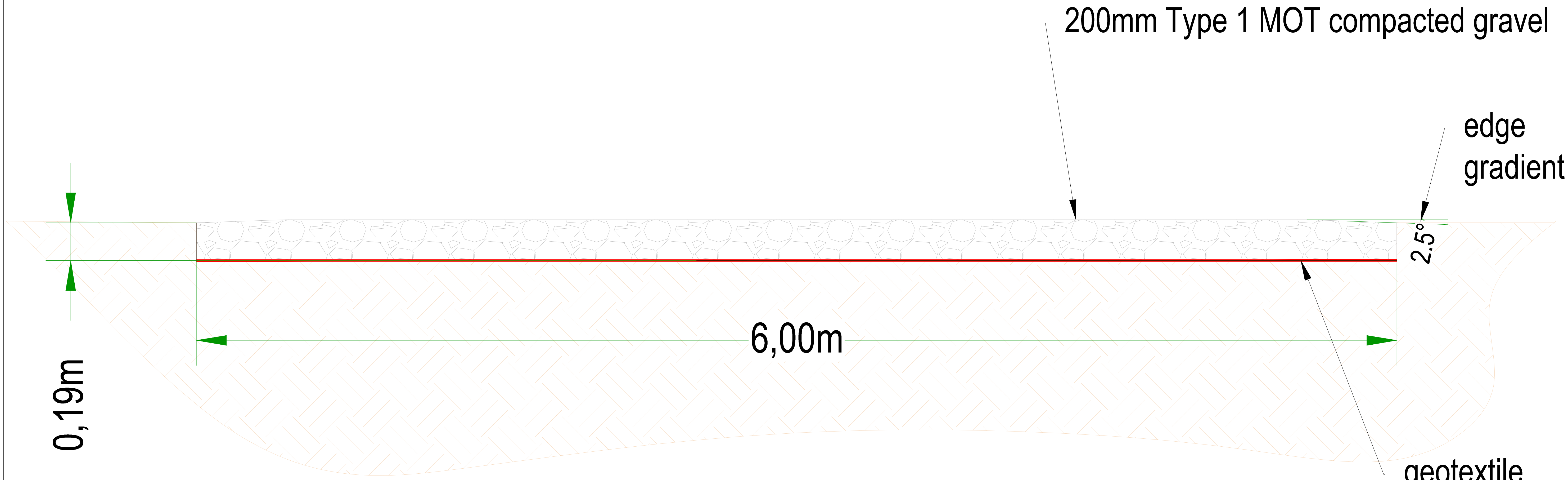
Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Growth curve factor 200 years:	3.04	3.04

Greenfield runoff rates	Default	Edited
Q_{BAR} (l/s):	4.34	4.34
1 in 1 year (l/s):	3.6	3.6
1 in 30 years (l/s):	8.68	8.68
1 in 100 year (l/s):	11.15	11.15
1 in 200 years (l/s):	13.19	13.19

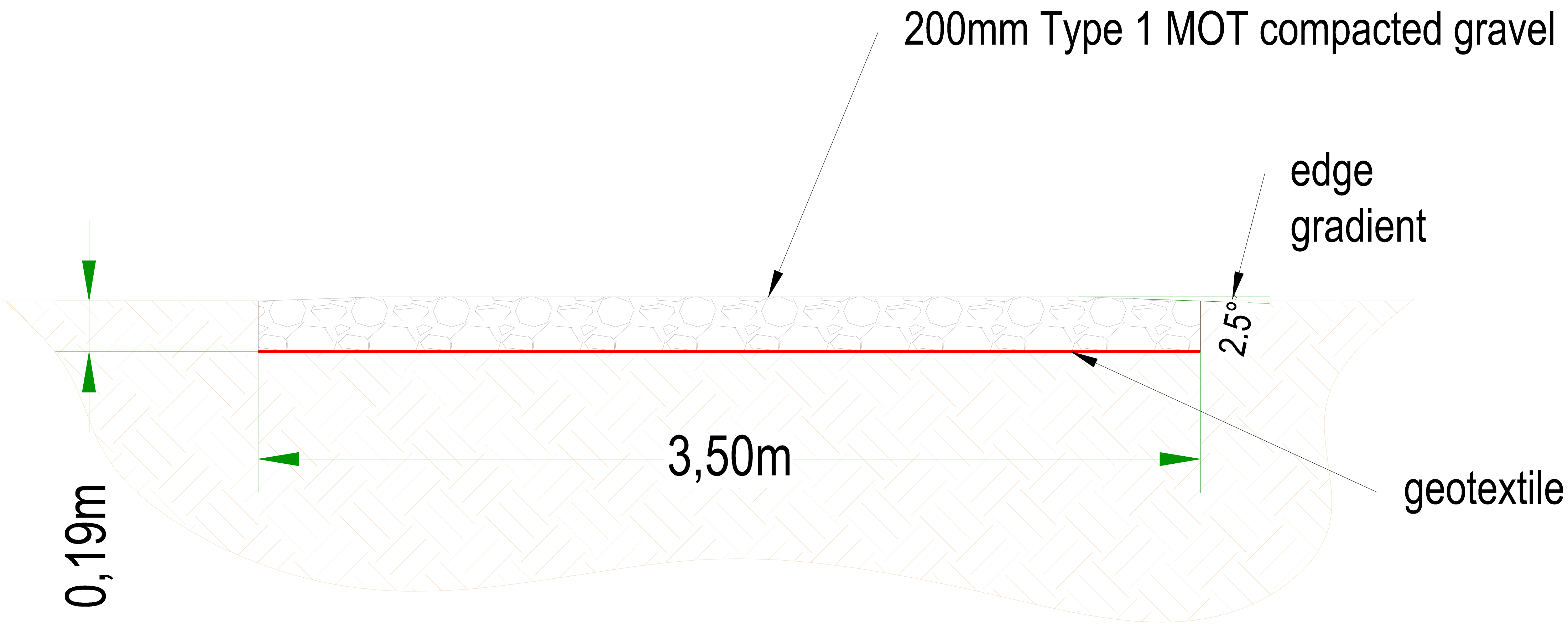
This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this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

Appendix C Access Track Cross-Section



BayWa r.e. BayWa r.e. Solar Projects GmbH Arabellastrasse 4 | 81925 München, Germany Tel: +49 89 383932-0 | www.baywa-re.com			Project: Oaklands UK PV Power Plant	
	Date	Name:	Client:	Scale
Drawn	01.02.2021	sur	Oaklands Farm Solar Limited	1:20
Changed	---	----		
Checked	---	----	Planning Period:	Format
Drawing Title:	04_detail_gravel road_6.0m		Preliminary	A3
file name:	BWre21-OSF_PD_V2.dwg			

Figure 4.16b Indicative Access Tracks 6m passing places



BayWa r.e.

BayWa r.e. Solar Projects GmbH
Arabellastrasse 4 | 81925 München, Germany
Tel: +49 89 383932-0 | www.baywa-re.com

			Project:	Oaklands UK PV Power Plant
	Date	Name:	Client:	Scale
Drawn	01.02.2021	sur	Oaklands Farm Solar Limited	1:20
Changed	---	----		
Checked	---	----	Planning Period:	Format
Drawing Title:	04_detail_gravel road_3.5m		Preliminary	A3
file name:	BWre21-OSF_PD_V2.dwg			

Figure 4.16c Indicative Access Tracks 3.5m wide

Appendix D Battery Storage Details

Legend:

- O&M access track (3,5m width)
- Impermeable hard standing
- BESS transformer and PCS
- BESS battery containers

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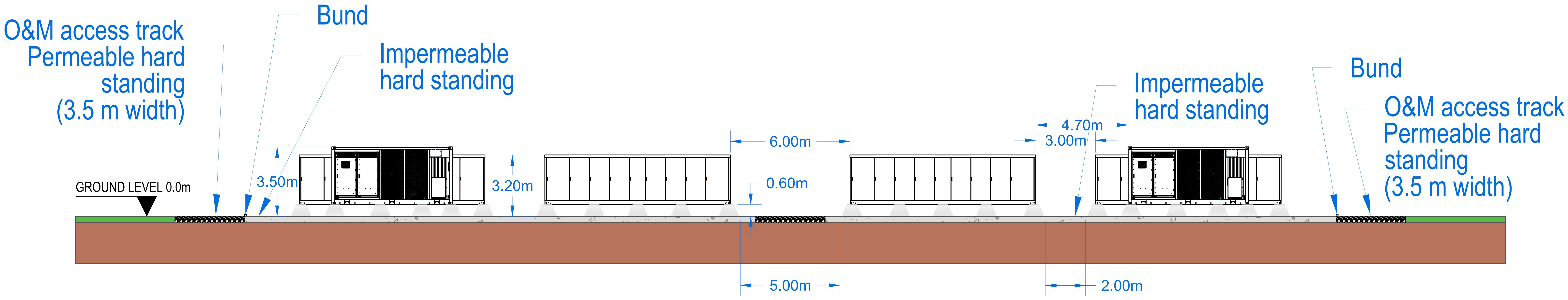
	Date	Name:
Drawn	08.03.2022	gky
Changed	19.10.2023	moa
Checked		
Drawing Title:	Indicative arrangement battery storage - Compound	
file name:	Battery Storage Elevation Drawing.dwg	

Project:	Oaklands UK Battery Storage
Client:	Oaklands Farm Solar Limited
Scale	1:250
Planning Period:	Preliminary
Format	A2

BESS - BATTERY STORAGE COMPOUND
SECTION A-A'



Note: See also drawing "Indicative arrangement battery storage - Compound" or "Figure 2 - BESS compound"

Legend:

BESS battery container

BESS transformer and PCS

BayWa r.e.

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	Date	Name:
Drawn	08.03.2022	gky
Changed	19.10.2023	moa
Checked		
Drawing Title:	Indicative arrangement battery storage Section A-A'	
file name:	Battery Storage Elevation Drawing.dwg	

Project: Oaklands
UK
Battery Storage

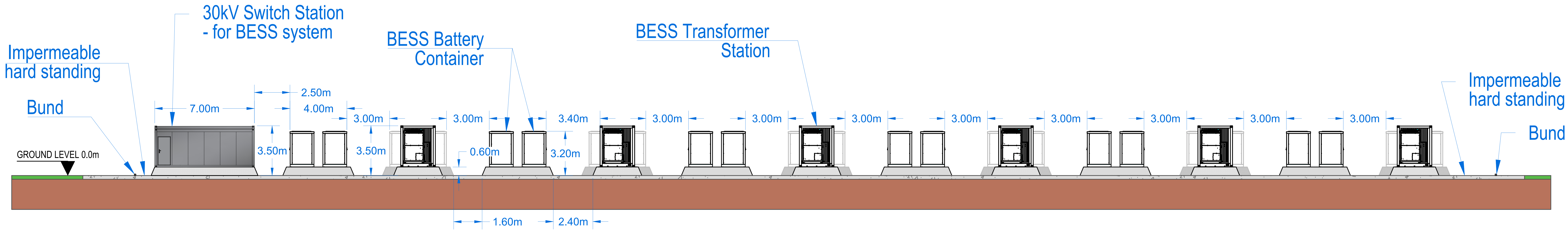
Client: Oaklands Farm Solar Limited

Scale: 1:50

Planning Period: Preliminary

Format: A2

BESS - BATTERY STORAGE COMPOUND
SECTION B-B'



Note: See also drawing "Indicative arrangement battery storage - Compound" or "Figure 2 - BESS compound"

Legend:

O&M access track (3,5m width)

BESS transformer and PCS

BESS battery container

BayWa r.e. — BayWa r.e. Solar Projects GmbH, Arabellastrasse 4 | 81925 München, Germany, Tel: +49 89 383932-0 | www.baywa-re.com			Project: Oaklands UK Battery Storage	
	Date	Name:	Client: Oaklands Farm Solar Limited	Scale 1:200
Drawn	08.03.2022	gky		
Changed	19.10.2023	moa		
Checked			Planning Period: Preliminary	Format A2
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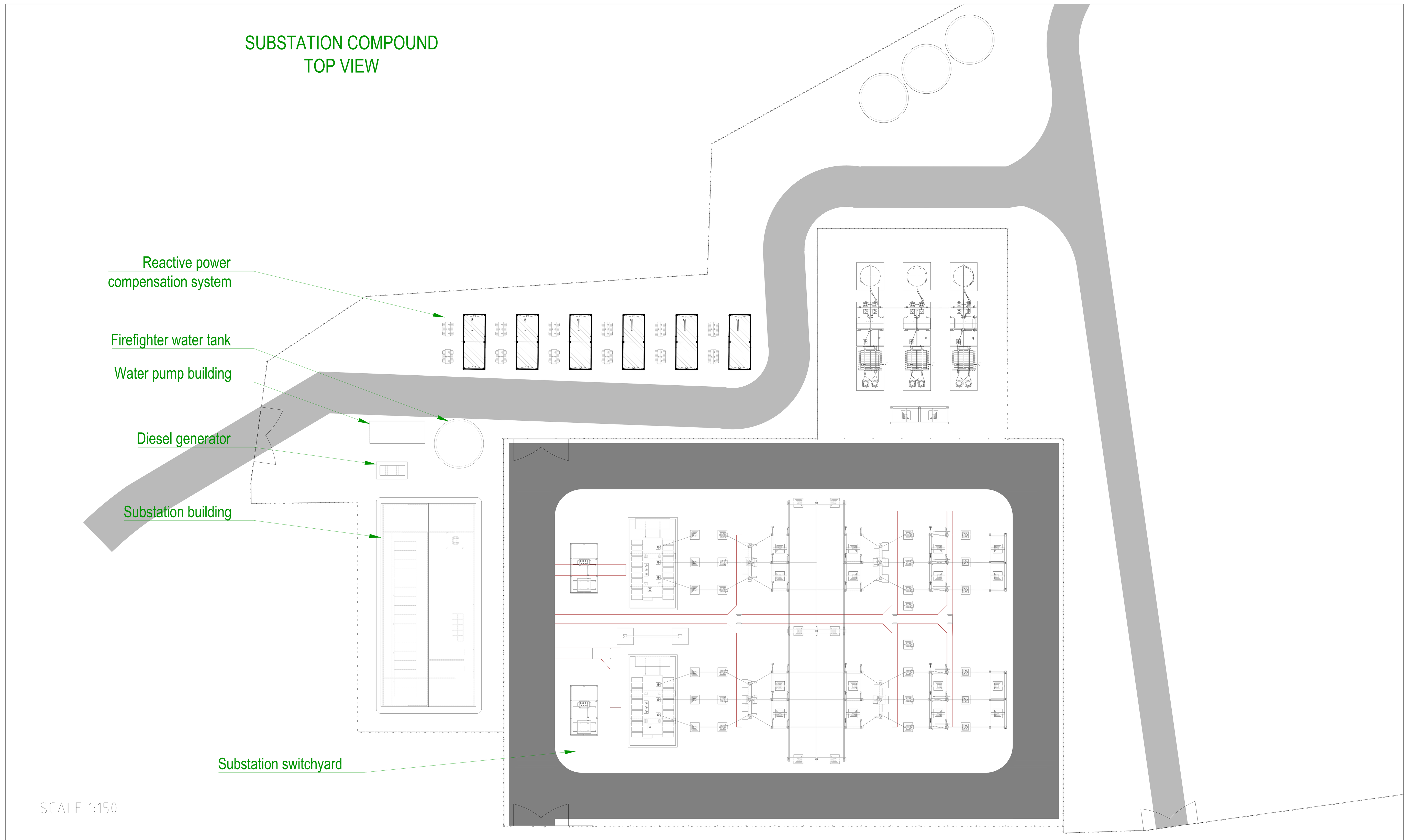
file name:	Battery Storage Elevation Drawing.dwg			

Appendix E Substation Details

SUBSTATION COMPOUND
TOP VIEW



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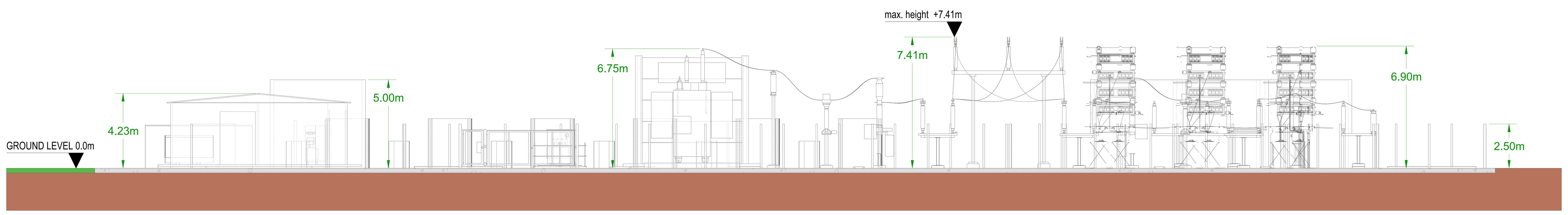
SIDE VIEW



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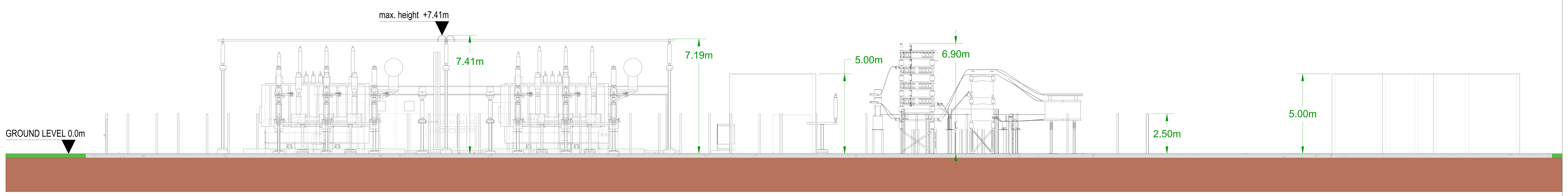
FRONT VIEW



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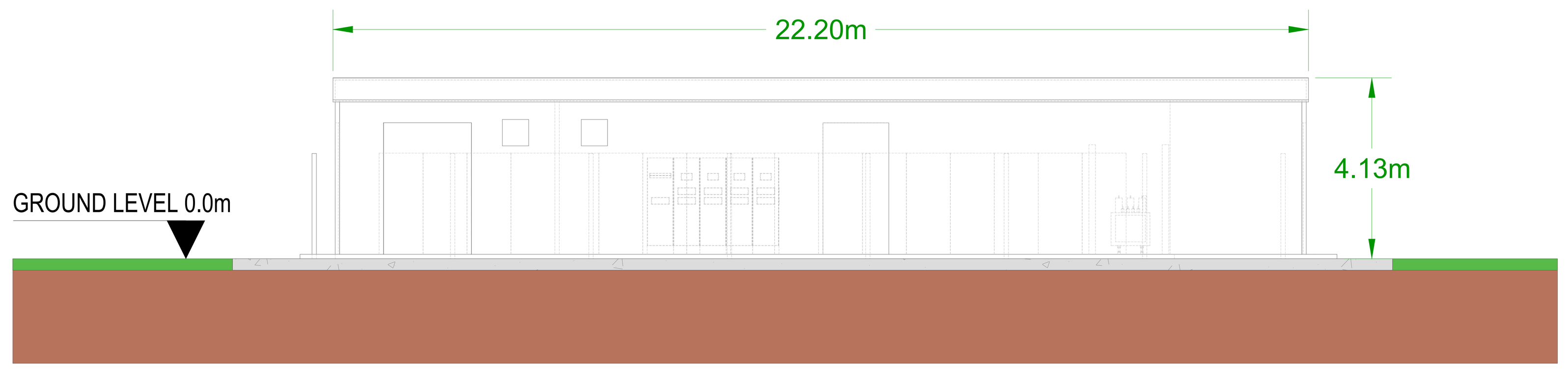
SUBSTATION BUILDING
SIDE VIEW



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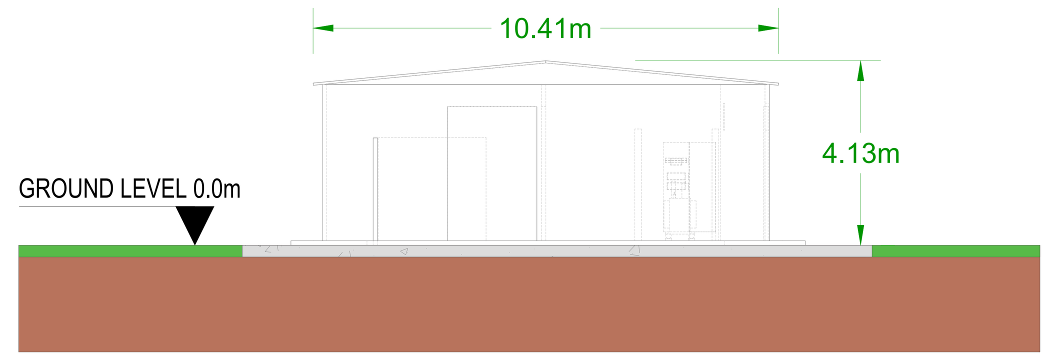
SUBSTATION BUILDING
FRONT VIEW



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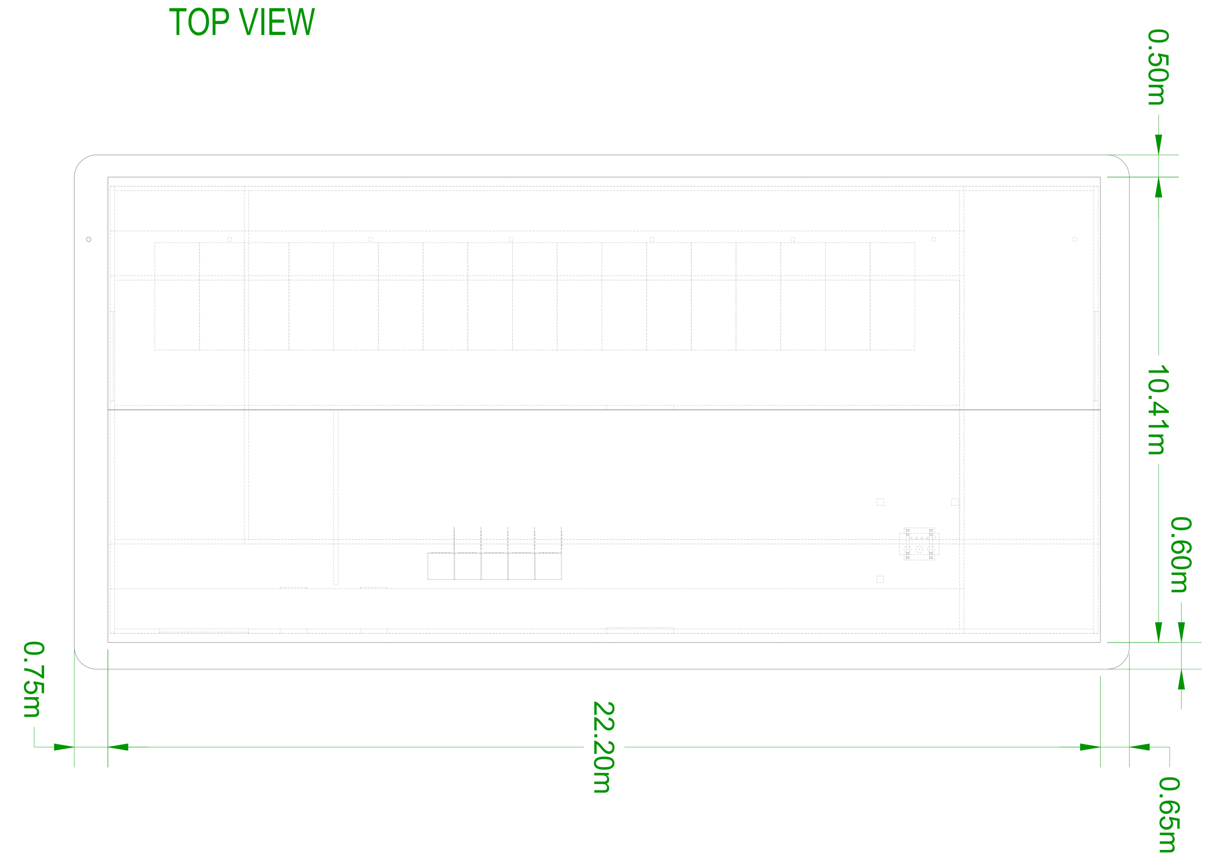
SUBSTATION BUILDING
TOP VIEW



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Index	Date	Name Editor	Modification / Adaptation of the drawing
j			
i			
h			
g			
f			
e			
d			
c			
b			
a	11.09.2023	moa	revised section views, added reactors and water tanks

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Project: Oaklands
The UK
Ground Mounted PV

	Date:	Name:
Drawn:	20.07.2021	sur
Changed:		
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Drawing Title:	01_details substation_v2	
file name:	BWre-OAKF-PD-dHV.dwg	

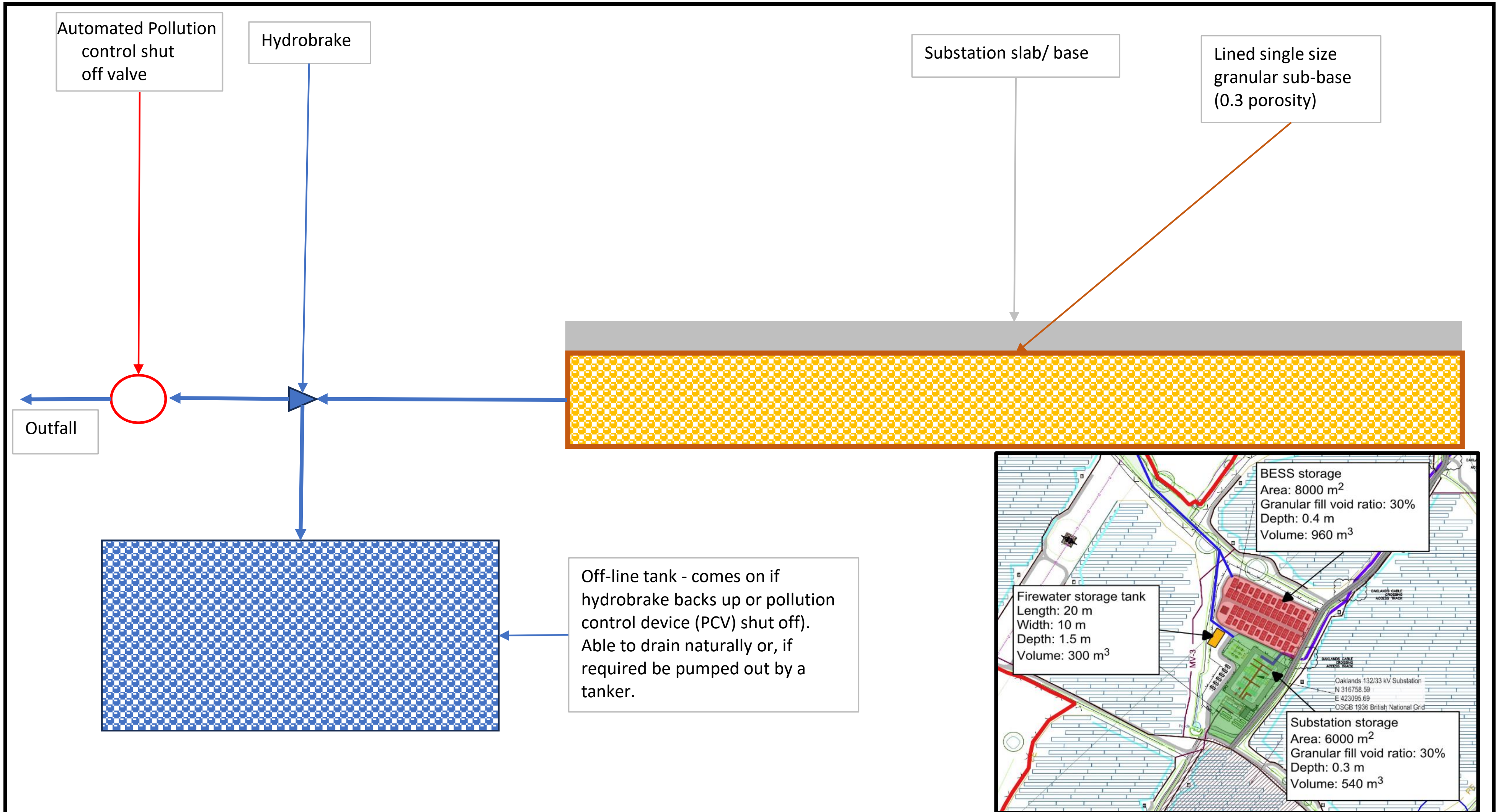
Client: BayWa r.e. Solar Project

Scale

Planning Period: Preliminary Designa

Format: A0

Appendix F SuDS layout



Drawing Title	**SuDS Schematic**	Scale	**NTS**
		Revision	**0**
Drawing Number	-	Drawn by	**JEM**
Project Name	**Oaklands Solar Farm**	Checked by	-
Project Number	**P20209**	Notes	© Crown copyright 2023
Client	**BayWa**		OS 100062269

YELLOW SUB GEO

Appendix G Flood Modelling report

AQUA TERRA | CONSULTING

Oaklands Farm Solar Park: Flood modelling

P20209_R5
October 2024

AQUA TERRA
CONSULTING

Document Control

Title

Oaklands Farm Solar Park: Flood modelling

Client

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Reference

P24022_R5

Status

Final

Document Reference	Issue Date	Comments	Written by	Reviewed by	Approved by
P24022_R5	August 2024	Final draft for comment	MJF	ACW	JEM
P24022_R5_Rev1	October 2024	Revision following EA comments	MJF	JEM	JEM

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1. Introduction

1.1. Instruction

Aqua Terra Consulting was instructed by BayWa R.E. UK Ltd (the Client) to undertake flood risk modelling for a parcel of land between Oaklands Farm, Fairfields Farm and Park Farm (the Site) to support the application for a Development Consent Order and Environmental Impact Assessment (EIA) for a proposed solar farm.

1.2. Background

The Site is located in Swadlincote to the south of Burton-on-Trent. The Proposed Development involves the installation of a solar farm comprising ground mounted photovoltaic (PV) panels across 37 agricultural fields with associated Battery Energy Storage System (BESS) and a connection established to the nearby former Drakelow Power Station.

Running through the Site is an Ordinary Watercourse, including a tributary thereof. Existing flood risk zones provided by the Environment Agency (EA) are based on a coarse nation-wide modelling and mapping exercise. The modelling does not incorporate an assessment of climate change or appropriate detail of the watercourse capacity. Therefore a site-specific assessment is required in order to support a Flood Risk Assessment for the Site.

1.3. Scope

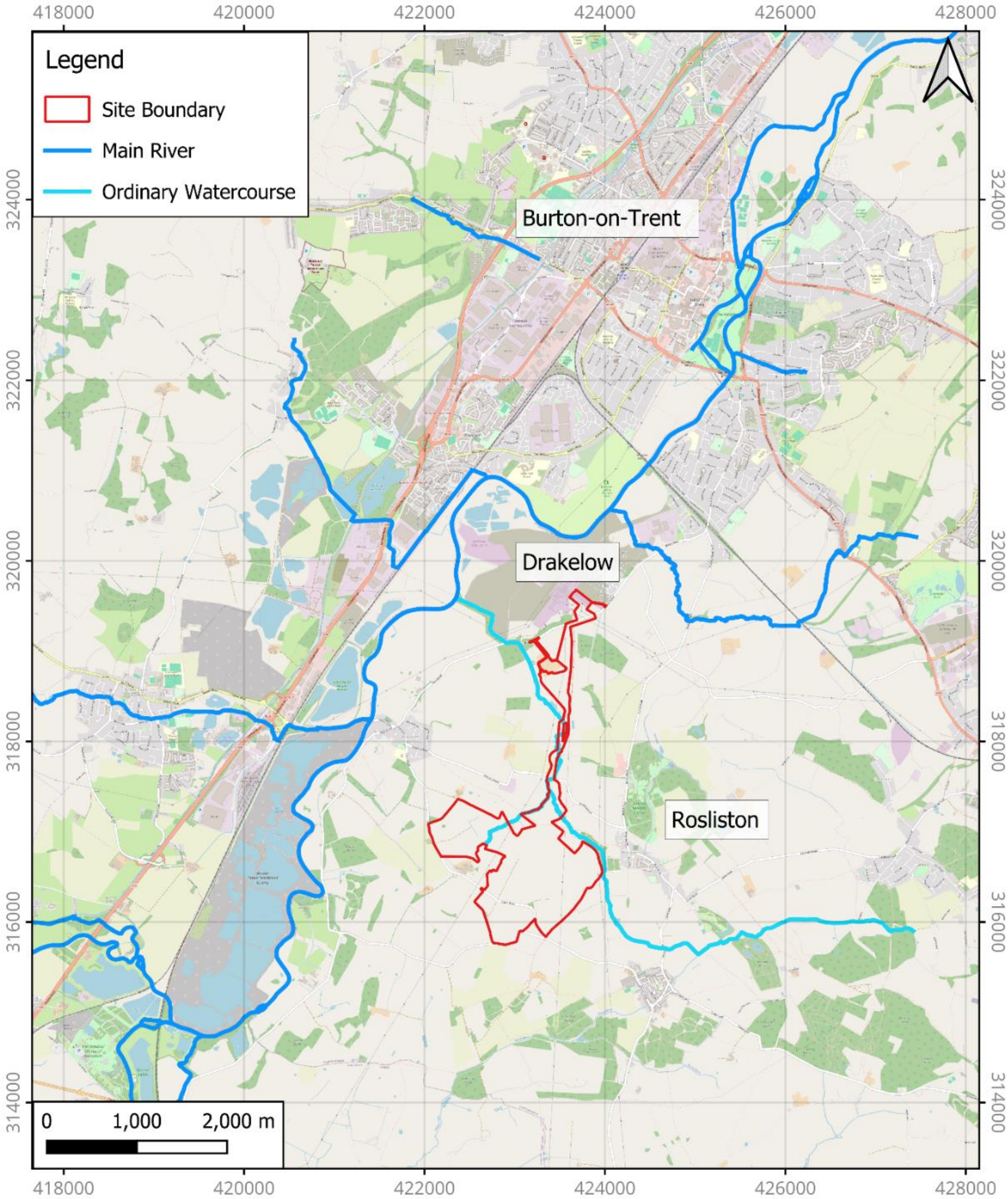
The scope of work was to:

- Commission a topographic watercourse survey for the study reach;
- Undertake a hydrological assessment for the Site including climate change analysis;
- Create a baseline 1D hydraulic model representing the watercourses through the Site;
- Create flood depth and flood extent mapping for the 0.1% Annual Exceedance Probability (AEP), 1% AEP and 3.3% AEP events with and without climate change;
- Update the hydraulic model with the proposed watercourse crossings and assess the impact of the proposed development on flood risk; and,
- Undertake sensitivity analysis on channel roughness, and upstream and downstream boundary conditions.

1.4. Site location

The Site (see Figure 1.1) lies within the administrative boundaries of South Derbyshire District Council (SDDC) and Derbyshire County Council (DCC), located approximately 0.25km west of the village of Rosliston and 0.7km southeast of Walton-on-Trent. The Site occupies a total area of approximately 127 hectares, and stretches from the former Drakelow Power Station, north of Walton Road, to the south of Coton Road. The Ordinary Watercourse running south to north through the Site drains into the Trent upstream of Burton-upon-Trent. A small tributary of the Ordinary Watercourse also flows across the site from south-west to north-east where it joins the main watercourse.

Figure 1.1 Site location

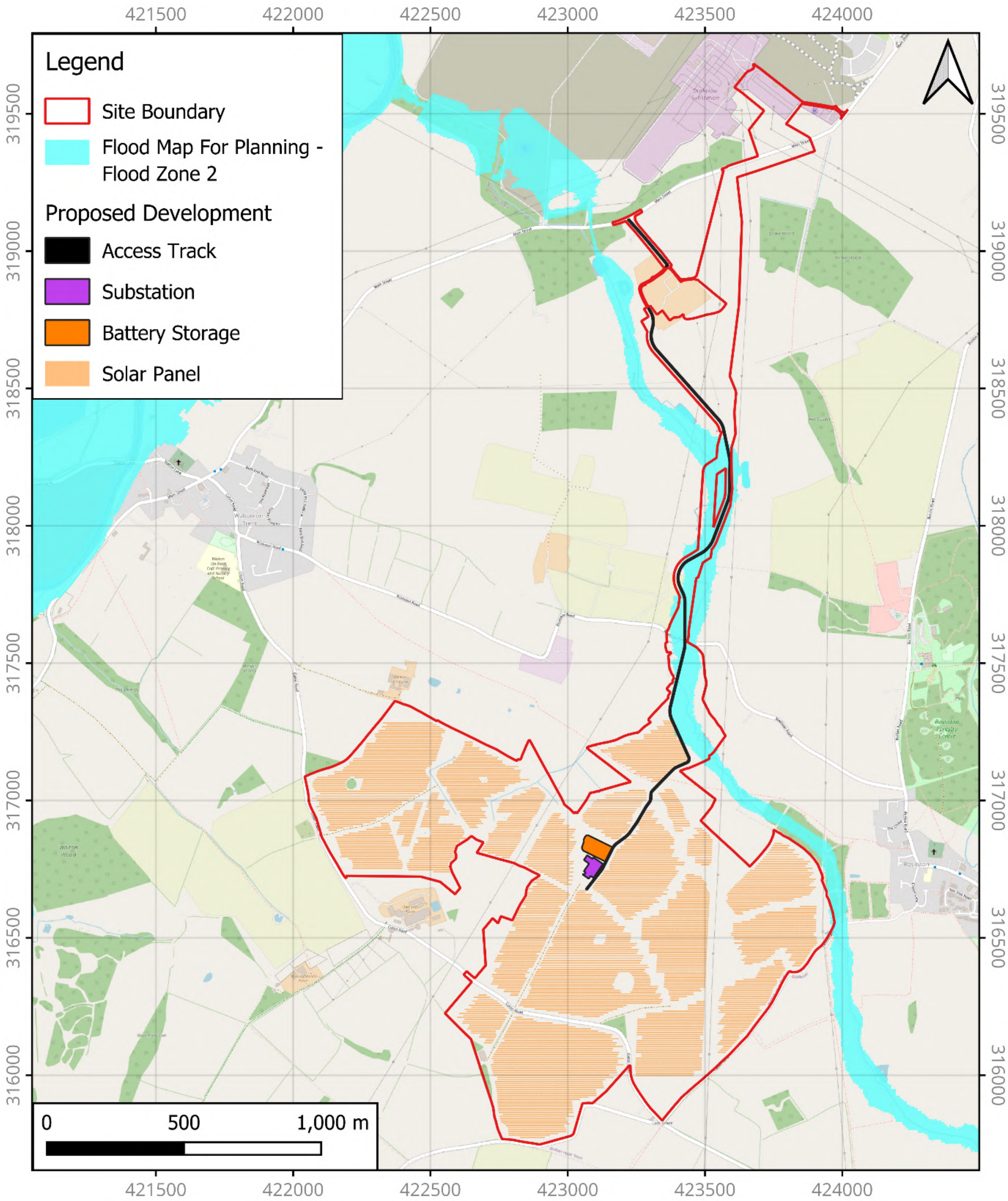


1.5. Proposed development

The Proposed Development comprises of photovoltaic panel arrays, a central electricity substation and Battery Energy Storage System together with access, landscaping and other works. A high voltage underground electricity cable will then run through land to the north to connect the solar farm to the national grid via an electricity substation located at the former Drakelow Power Station which sits south of Burton-upon-Trent. provides an overview of the

Proposed Development, with existing Flood Zone 2 extents provided for context. It should be noted however that the small tributary crossing the Site is not represented within the Flood Zone 2 extents.

Figure 1.2 Proposed Development and existing Flood Zone 2 extents



2. Data sources

2.1. Topographic data

LiDAR data from the National LiDAR Programme (Department for Environment Food & Rural Affairs, 2024) has been downloaded in the form of a Digital Terrain Model. The LiDAR was flown in 2021 and is at a 1m resolution. Figure 2.1 shows the LiDAR data.

2.2. Watercourse survey

A watercourse survey has been undertaken by Land Utility Group along the Ordinary Watercourse and a tributary that pass through the Site. Where the watercourse is in close proximity with the Proposed Development 50m spacing of cross-sections have been used and where the watercourse is set back from the Proposed Development, or outside of the Site a 100m spacing has been used, although precise chainages between sections vary from this to account for access limitations. The surveyed reach stops short of the full model extents at the downstream end of the study area as the extra expense and detail of undertaking watercourse survey to the confluence with the River Trent was not warranted for the purposes of this study.

Figure 2.2 details the locations where cross-section survey has been undertaken. It should be noted that the cross-section names do not necessarily correspond to their order from upstream to downstream. The survey of 5No. key structures was also undertaken, although it was not possible to survey the bridge adjacent to Section 29 due to access limitations, or a bridge located further downstream.

A spot check of LiDAR data against surveyed data has been undertaken at 5No. locations across the Site. These have primarily focused on areas where the watercourses come into close proximity with the Proposed Development. The purpose of this assessment is to:

1) Gain an understanding of the differences between the surveyed cross-section data and LIDAR data, and therefore uncertainty that may be inherent in using the LiDAR data for flood mapping.
2) Define a generic watercourse cross-section profile that can be used to adjust the LiDAR data to create model cross-sections where survey data was either unable to be collected or in the downstream reaches of the model which were outside of the scope of the survey.

The comparison between LiDAR and surveyed cross-sections is presented in Appendix B with an example provided as Figure 2.3. The comparison has demonstrated that the LiDAR data is representing the overall location and width of the channel well, however the depth of the channel is typically not fully captured by the LiDAR, likely due to vegetation and reflection from the water surface.

Figure 2.1 LiDAR data across the Site

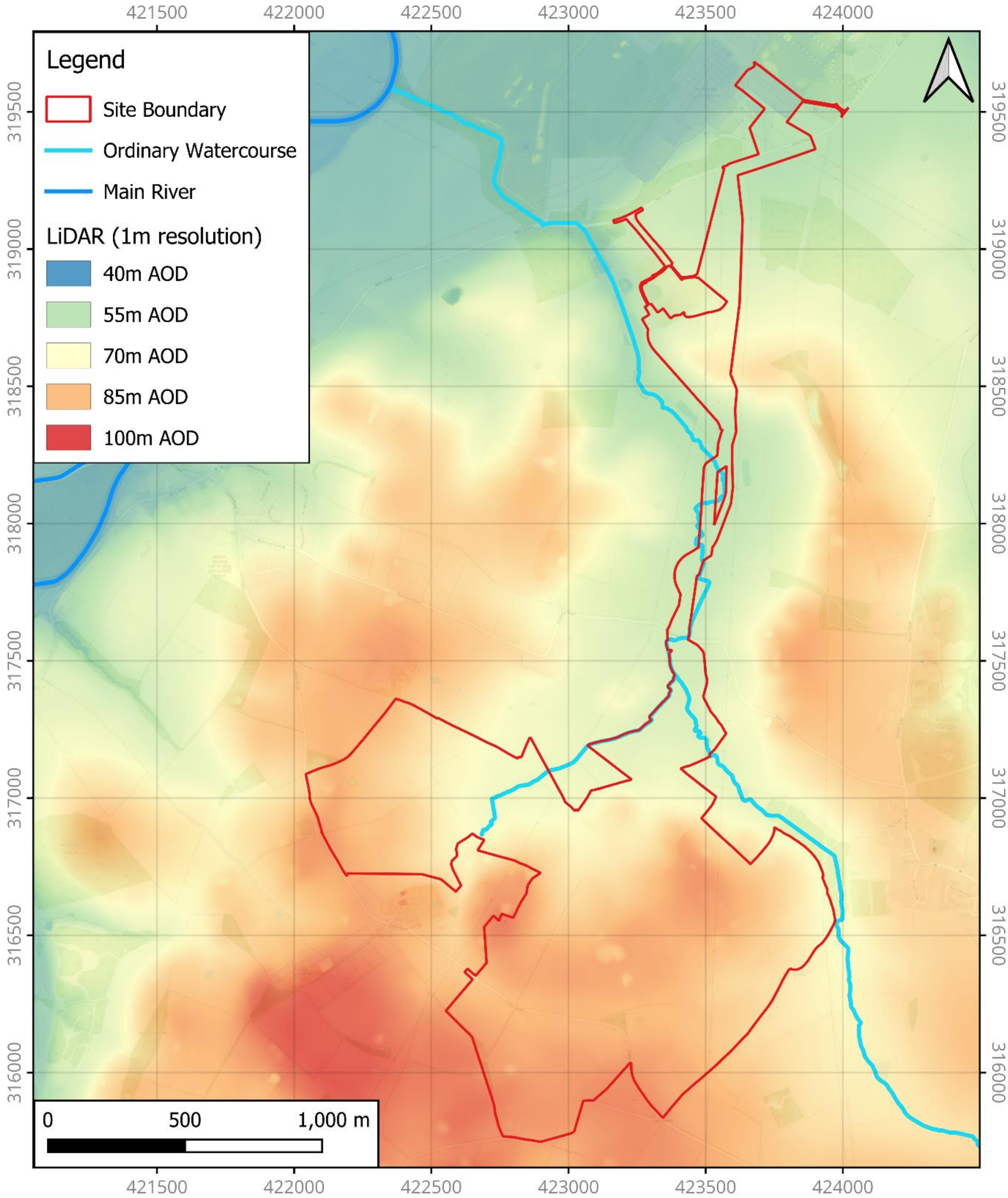


Figure 2.2 Watercourse survey locations

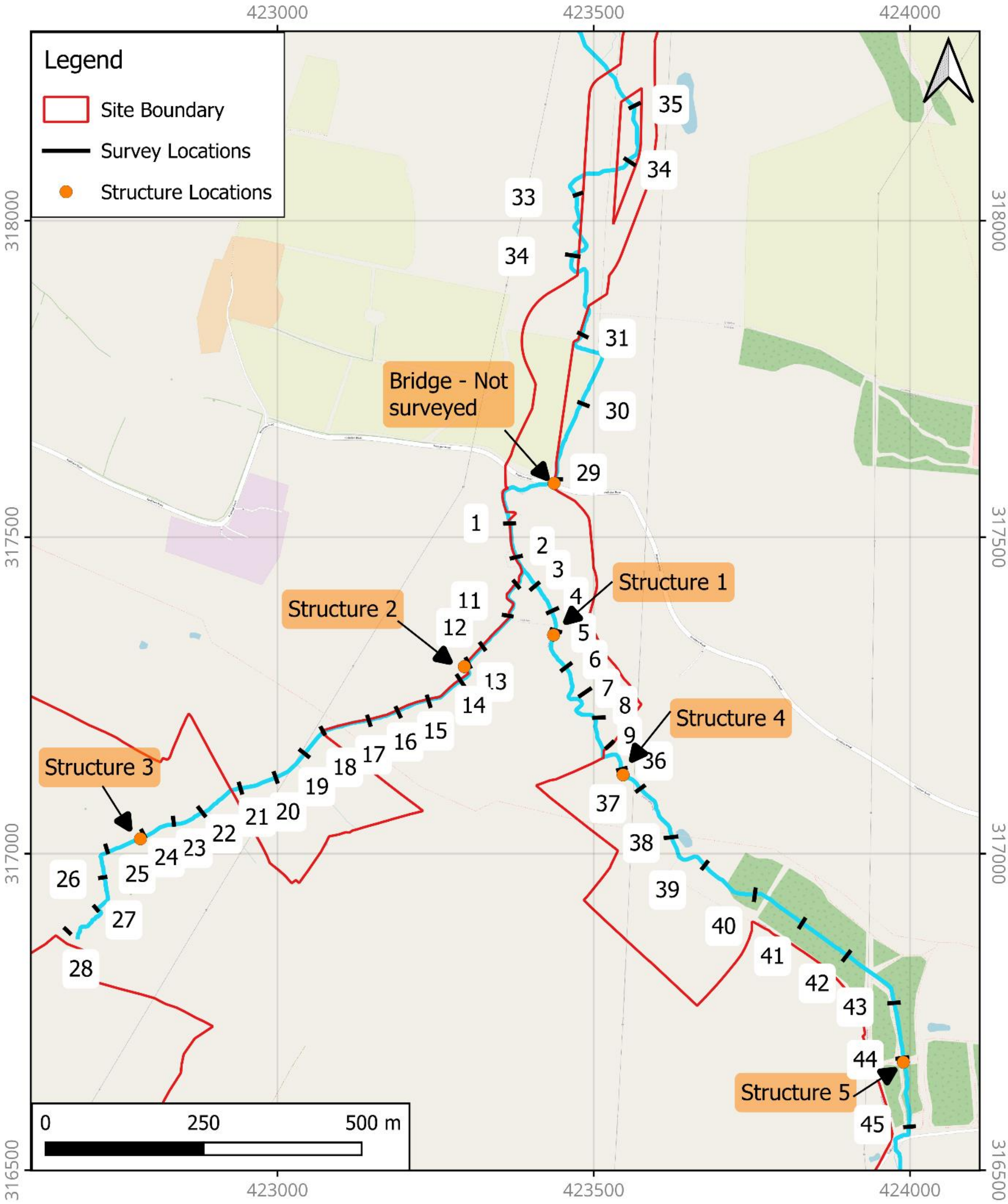
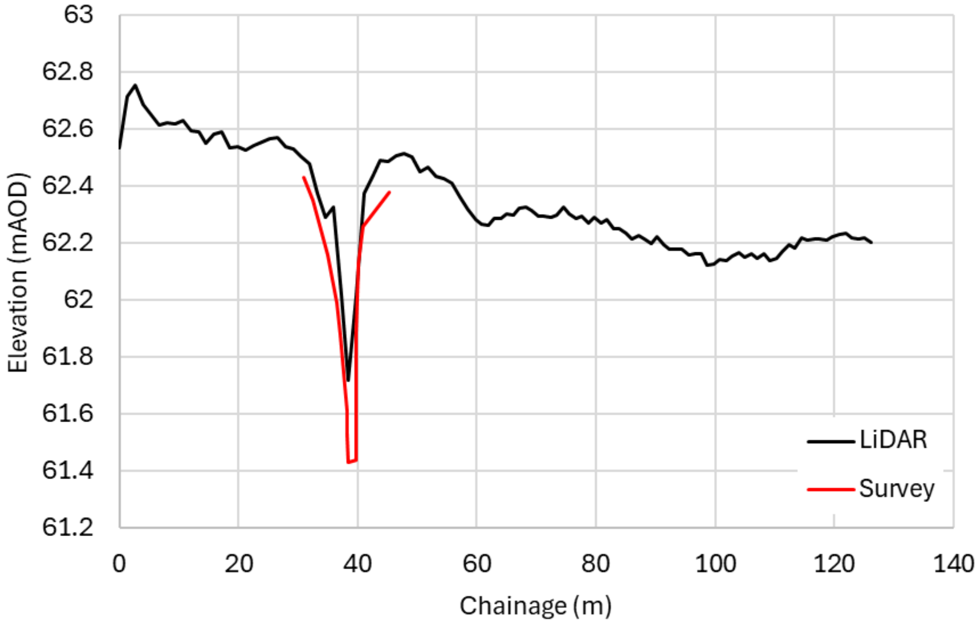


Figure 2.3 Comparison between surveyed cross-section and LIDAR data (S8ection 3)



2.3. Hydrological data

The Ordinary Watercourse through the Site and its tributary are ungauged. The nearest National River Flow Archive (NFRA) gauging station is Trent at Drakelow Park, close downstream of the watercourses confluence with the River Trent, however the characteristics of a large watercourse such as the Trent, and the Ordinary Watercourse on Site are very different.

With the absence of gauged data within the study area, or calibration data for which rainfall records may be beneficial, no further hydrological data has been collected. Catchment descriptors for the Site have been obtained from the FEH Web Service. These are discussed in further detail in Section 3.

Modelled water levels on the River Trent, near where the Ordinary Watercourse flows into the River Trent have been obtained from the Environment Agency. These are taken from Cross section 3161210850 from the Burton FRMS model (2022). The cross-section grid reference is 422360, 319615. Table 2-1 summarises the provided data.

Table 2-1 Modelled peak flows (River Trent) at confluence with Ordinary Watercourse

Annual Exceedance Probability (AEP)	Return period	Maximum level (m aOD)
5%	20	47.77
2%	50	48.05
1.33%	75	48.09
1%	100	48.19
1% + 22% CC	100 + 22% CC	48.43
1% + 30% CC	100 + 30% CC	48.52
1% + 51% CC	100 + 51% CC	48.69
0.5%	200	48.28
0.1%	1000	48.48

3. Hydrology

A hydrological assessment has been undertaken for the Ordinary Watercourse and its tributary which pass through the Site. A summary of the chosen methodology and outputs is provided below, with a flood estimation calculation record provided in Appendix C.

3.1. Catchment delineation and flood estimation points

A single flood estimation point (FEP 1) has been located at the downstream limit of the Site.

The study area has been delineated into several sub-catchments:

- Ordinary Watercourse Upstream (OW - 1): This is the Ordinary Watercourse at the upstream extent of the model;
- Ordinary Watercourse Intermediate 1 (OW – 2): This is the intervening catchment between the upstream limit of the model and the confluence with the tributary;
- Ordinary Watercourse Intermediate 2 (OW – 3): This is the intervening catchment between the confluence with the tributary and the FEP at the downstream limit of the Site;
- Tributary Upstream (TRIB – 1): This is the tributary catchment at the upstream extent of the model; and,
- Tributary Intermediate 1 (TRIB – 2): This is the intervening catchment between the upstream limit of the model along the tributary, and the confluence with the Ordinary Watercourse.

Catchment descriptors have been obtained from the FEH Web Service for the FEP. It is intended that a single flow hydrograph is to be derived using the catchment descriptors at the FEP, and that this will be distributed to other catchments based on area – therefore additional catchment descriptors for these sub-catchments is not required.

The catchment delineation is shown in Figure 3.1 and a table summarising the key catchment properties is provided in Table 3-1. Catchments have been delineated based on LiDAR data and in the case of the FEP, compared against the FEH catchment outline.
Overall the catchments delineated based on LiDAR compare well with the FEH catchment outline (difference in area of 3%), however within OW-1, there is a clear difference where a stream heads south-west through the village of Coton-in-the-Elms. This stream appears to originate very close (or even connected) to the course of the Ordinary Watercourse modelled within this Study. Figure 3.2 shows a detail of the LiDAR and aerial imagery of the location. As a conservative measure, the full catchment to the east of this location has been assumed to contribute to the modelled watercourse, with no flow lost to the neighbouring stream. This would be the case if either there is no connection between the two watercourses, or, if there were a connection, the culvert were to be blocked.

Figure 3.1 Flood estimation points and catchment delineation

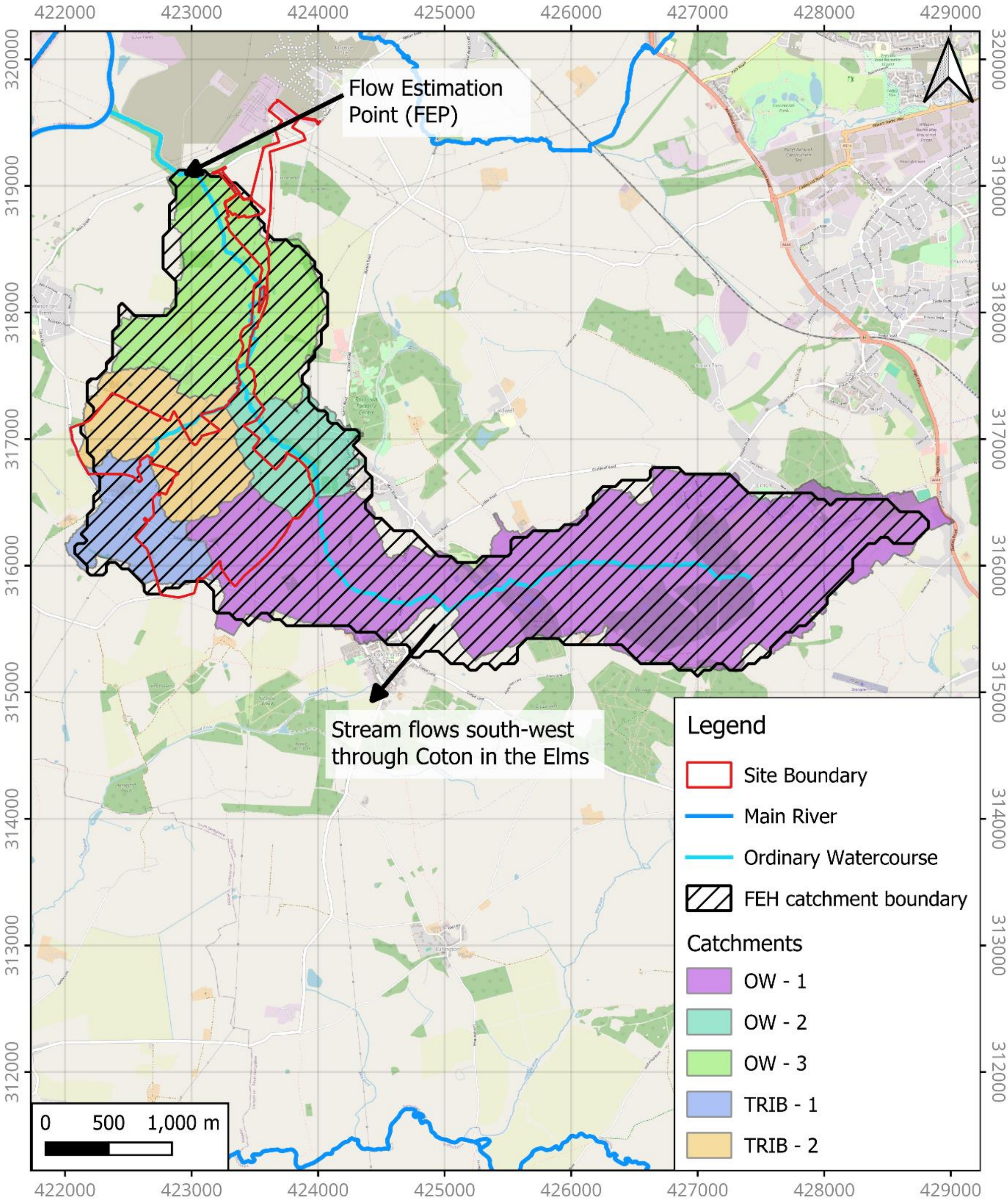
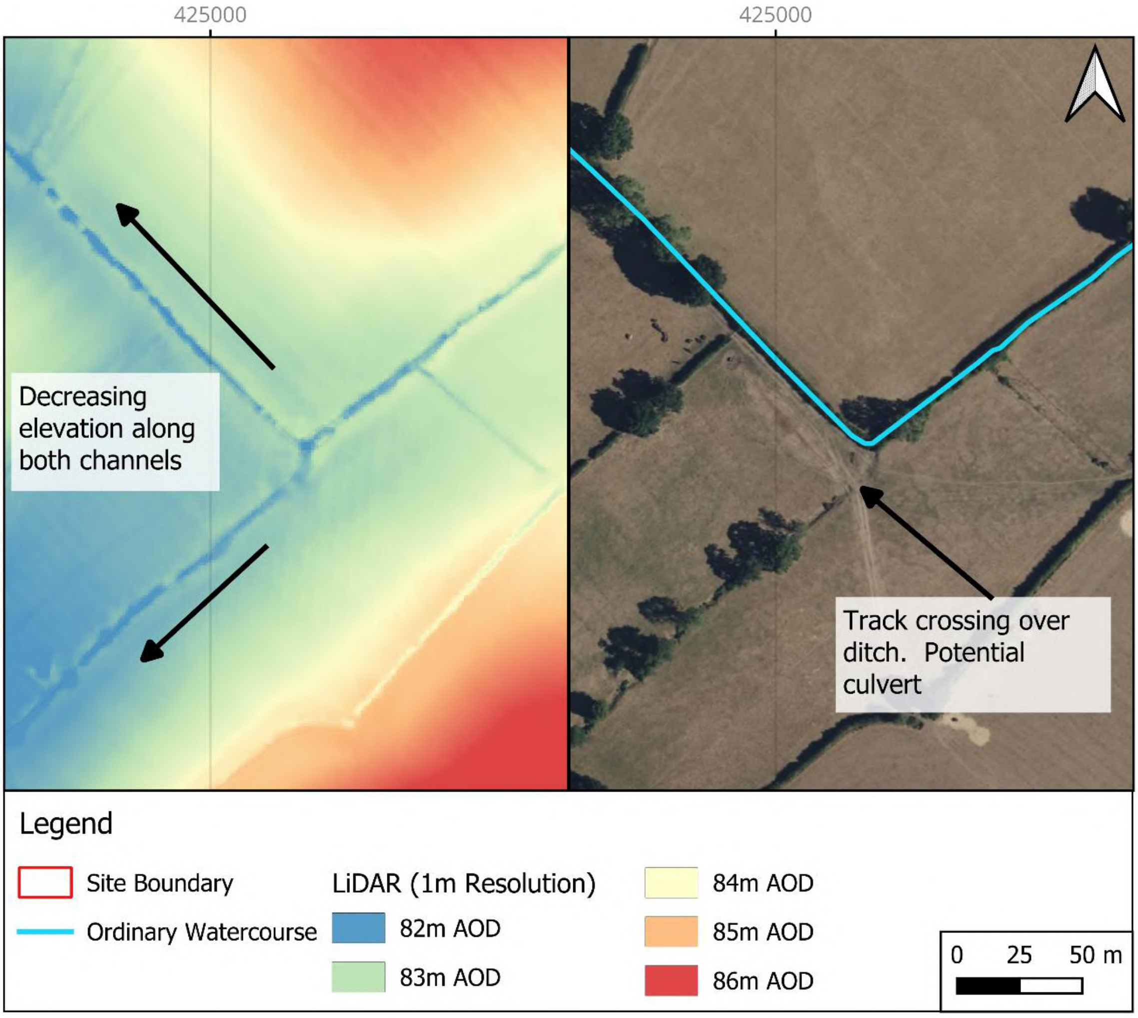


Table 3-1 Summary of Catchment Descriptors

Property	FEP 1	OW - 1	OW – 2	OW – 3	TRIB – 1	TRIB – 2
Catchment area (km2)	9.69 (FEH – 9.97)	5.46	0.69	1.83	0.67	1.04
Proportion of catchment (%)	100%	56%	7%	19%	7%	11%
BFIHOST	0.469					
BFIHOST19	0.455					
DPLBAR (km)	4.9					
DPSBAR (m/km)	28.9					
FARL	1					
FPEXT	0.0912					
PROPWET	0.3					
SAAR (mm)	641					
URBEXT 2000 (Updated to 2024)	0.021 (0.022)					

Figure 3.2 Detail of catchment drainage near Coton in the Elms



3.2. Geological properties

British Geological Survey (BGS) published geology indicates that the Site bedrock comprises the Edwalton Member (siltstone and very fine-grained sandstone) in the west, and the Gunthorpe Member (mudstone) in the east. This is partly overlain by superficial deposits, comprising fluvioglacial diamicton (Till) in the south and some areas of Alluvium in the north, typically along the watercourses through the Site. According to SoilScapes, the soils close to the watercourses are described as “slowly permeable, seasonally wet, with impeded drainage”, whilst those away from the watercourses are described as “loamy and clayey soils with slightly impeded drainage”.

3.3. QMED analysis

QMED (Median Annual Flood) analysis has been undertaken at the Flow Estimation Point near the downstream limit of the Site (FEP 1). Three approaches have been used, FEH – Catchment descriptors, FEH – Donor adjusted and ReFH2. Table 3-2 summarises the output QMED values with the details of the calculations provided in the FEH Calculation record.

Table 3-2 Comparison of QMED using different approaches

	FEH – Catchment descriptors	FEH – Donor adjusted	ReFH 2
QMED (m^3/s) at FEP 1	1.57	1.12	1.62

The FEH – Donor Adjusted approach provides the lowest QMED value, whilst the FEH – Catchment Descriptor and ReFH2 approaches are fairly comparable. The ReFH 2 estimate has been used going forwards to be consistent with the method adopted for the higher return period flow estimates and to err on the conservative side. QMED (representative of a flood with return period of 2 years) is not itself one of the return periods being modelled in this study, however it is a key hydrological value of importance during hydrological assessments.

3.4. Growth curve analysis

Target peak flow estimates have been derived using the ReFH2 method at FEP 1. These are summarised in Table 3-3. Several checks of these estimates have been undertaken and are detailed in the FEH Calculation Record. In particular the 1% AEP growth factor (ratio of 1% AEP to QMED) has a value of 3.07 which is within a very typical range. In addition the model outputs using these target flows has been reviewed against the existing flood zone outlines in Section 5.1.1 as a sense-check of their suitability, whilst also recognising that the purpose of this study is to update those flood zones based on improved data, and therefore they are unlikely to match exactly.

The ReFH2 method has been used based on the simple assessment required for this study and the focus on larger return periods (such as the 1% AEP and 0.1% AEP) which are heavily dependent on long data records when using methods such as the FEH Statistical method (which itself recommends using ReFH2 or a ratio of the 0.1% to 1% AEP ReFH2 peak flows to adjust the FEH derived flows for large events).

Table 3-3 ReFH2 estimates of Target peak flows at Flood estimation point

Location	3.3% AEP	1% AEP	0.1% AEP
FEP 1	3.6	5.0	7.9

3.5. Climate change analysis

The study area lies within the Tame Anker and Mease Management Catchment. Table 3-4 summarises the peak river flow allowances for the central, higher and upper climate change scenarios.

The Proposed Development comprises of "Essential Infrastructure" under the NPPF vulnerability classification, and therefore in accordance with the guidance the higher central allowance should be used within flood zones 2, 3a or 3b. The upper end allowance has also been used to assess sensitivity.

The solar farm has a design life of 40 years, assuming development is completed in the next 5 years the Site will be in use until the 2060s. Therefore the 2080s epoch has been used, with allowances of 30% (Higher Central) and 51% (Upper End) applied to the 1% AEP event.

Table 3-4 Climate Change peak river flow allowances

Epoch	Central	Higher Central	Upper End
2020s	10%	15%	24%
2050s	11%	17%	30%
2080s	22%	30%	51%

Source: https://environment.data.gov.uk/hydrology/climate-change-allowances/river-flow?mgmtcatid=3090

3.6. Final Design Target Flows and Hydrographs

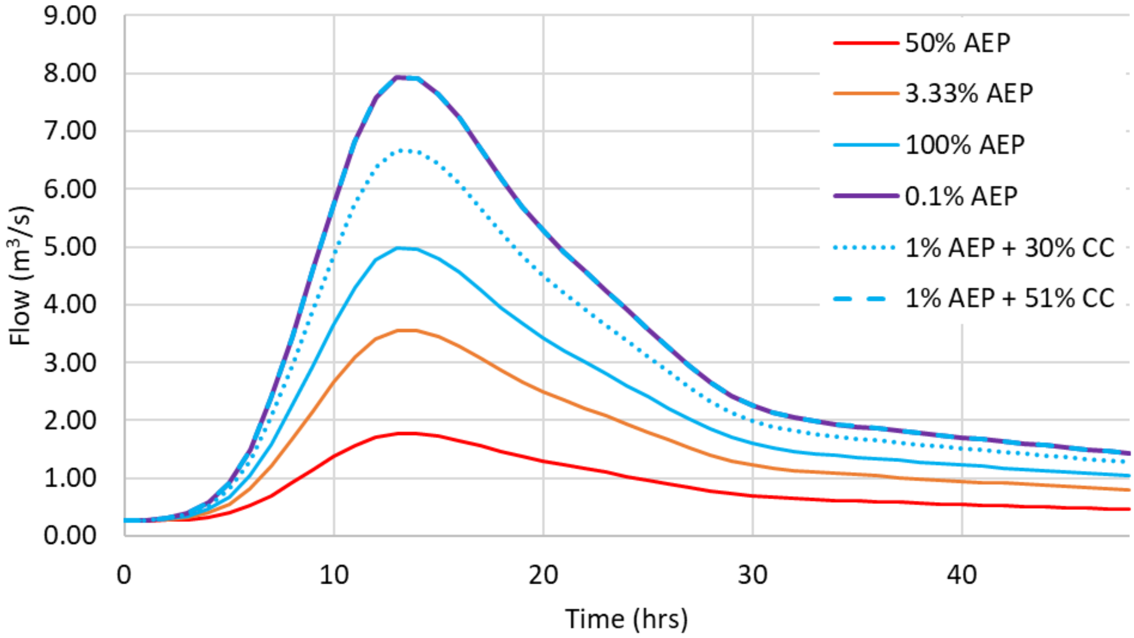
Table 3-5 summarises the final target peak flows at FEP 1, and Figure 3.3 the design hydrographs. The 1% AEP including 51% Climate Change peak flow and hydrograph is indistinguishable from the 0.1% AEP hydrograph (with a peak flow difference of 0.01 m^3/s). It is therefore considered suitable to drop the 1% AEP including 51% AEP event, and to use the 0.1% AEP model outputs in its place (for example for sensitivity testing of model inflows).

The design hydrographs have been derived using ReFH2 at the flood estimation point, using default parameters – this provides a critical storm duration of 11 hours.

Table 3-5 Target peak flows at Flood estimation point

Location	3.3% AEP	1% AEP	0.1% AEP	1% AEP inc 30% CC	1% AEP inc 51% CC
FEP 1	3.6	5.0	7.9	6.7	7.9

Figure 3.3 Design Hydrographs



3.7. Application to hydraulic model

The hydrological inflows have been applied to the hydraulic model by distributing the inflow hydrograph to each of the sub-catchments identified in Figure 3.1.

based on area weighting. Where a sub-catchment coincides with the upstream limit of either the Ordinary watercourse or its tributary this has been applied as a point inflow. Where the sub-catchment is an intervening catchment, the flow has been distributed lateral along the watercourse within the sub-catchment.

The input hydrograph have been scaled to ensure that the target peak flows at the flow estimation point are met within 1%. This therefore means that the input hydrographs do not exactly match those presented in Figure 3.3 as there is some attenuation within the model requiring slightly higher inflows in order to match the target flows downstream. All sub-catchments inflows have been scaled uniformly to meet the target flows, and the scaling factors used are discussed in Section 4.5.

3.8. Downstream boundary

The downstream boundary has been represented by a constant water level taken from the Burton FRMS model on a like-for-like return period basis where possible. The provided water levels for the River Trent did not include a 3.33% AEP therefore for that event the 2% AEP water level has been used. Due to the similarity in inflow hydrograph for the 1% AEP + 51%CC event and the 0.1% AEP event resulting in these runs being combined into a single run, the more conservative River Trent water level associated with the 1% AEP + 51% CC event has been used. This is a level of 48.69m AOD, compared to 48.48m AOD for the 0.1% AEP event.

Table 3-6 Summarises the downstream boundary water levels that have been used for each modelled event.

Table 3-6 Downstream boundary water levels applied to hydraulic model

Annual Exceedance Probability	Return Period	Downstream boundary level (mAOD)
3.33%	30	48.05 (taken from 2% AEP event)
1%	100	48.19
1% + 30% CC	100 + 30% CC	48.52
1% + 51% CC / 0.1% AEP	100 + 51% CC / 1000	48.69 (greater of the 1% + 51% AEP and 0.1% AEP events)

4. Hydraulic modelling

4.1. Modelling software

The hydraulic model has been built in HEC-RAS v6.5 (developed by the U.S. Army Corps of Engineers). Initially a 1D only approach using extended sections was considered as sufficiently detailed for the purposes of this study. Following initial modelling however a 1D-2D approach has been taken forward with a simple 2D domain due to a few locations where the watercourse does not lie within the base of the valley and therefore overland flow paths and ponding become important.

4.2. Model extents

Figure 4.1 shows the model extents, and location of boundary conditions which have been applied to the model. The model starts at the upstream extent of the Site, and therefore area of interest, and extends to the watercourses confluence with the River Trent. The model has been extended as far as the River Trent primarily to provide a convenient location for a downstream boundary. As such the level of detail within the model in the lower reaches (i.e. beyond where survey data was collected) is reduced.

4.3. Model geometry

4.3.1. 1D model

The survey data has been used to define the cross-section profiles within the upper half of the model. This is at an approximate 50m spacing through the primary area of interest, and 100 m spacing further downstream. Appendix C provides a table of each cross-section, source data, and model chainage.

In the lower half of the model where survey was not undertaken, the survey data that has been collected has been used to inform a typical cross-section profile. Figure 4.2 compares the surveyed profiles of the 9 most downstream sections, standardised to have a relative bank level of 0. There is a clear deepening and widening of the channel between those sections upstream of chainage 2617 and those downstream. The sections downstream have therefore been used, as more representative of the typical watercourse profile in the lower reaches, to create a standardised section. This standardised section has then been enforced into a LiDAR extracted section.

In some locations, particularly between a chainage of 0 and 1000, the LiDAR profile showed a deeper channel section than the standardised section. In these cases the LiDAR profile has been retained as likely to be more representative. An example of this is shown in Figure 4.3.

Figure 4.1 Model Extents and Boundaries

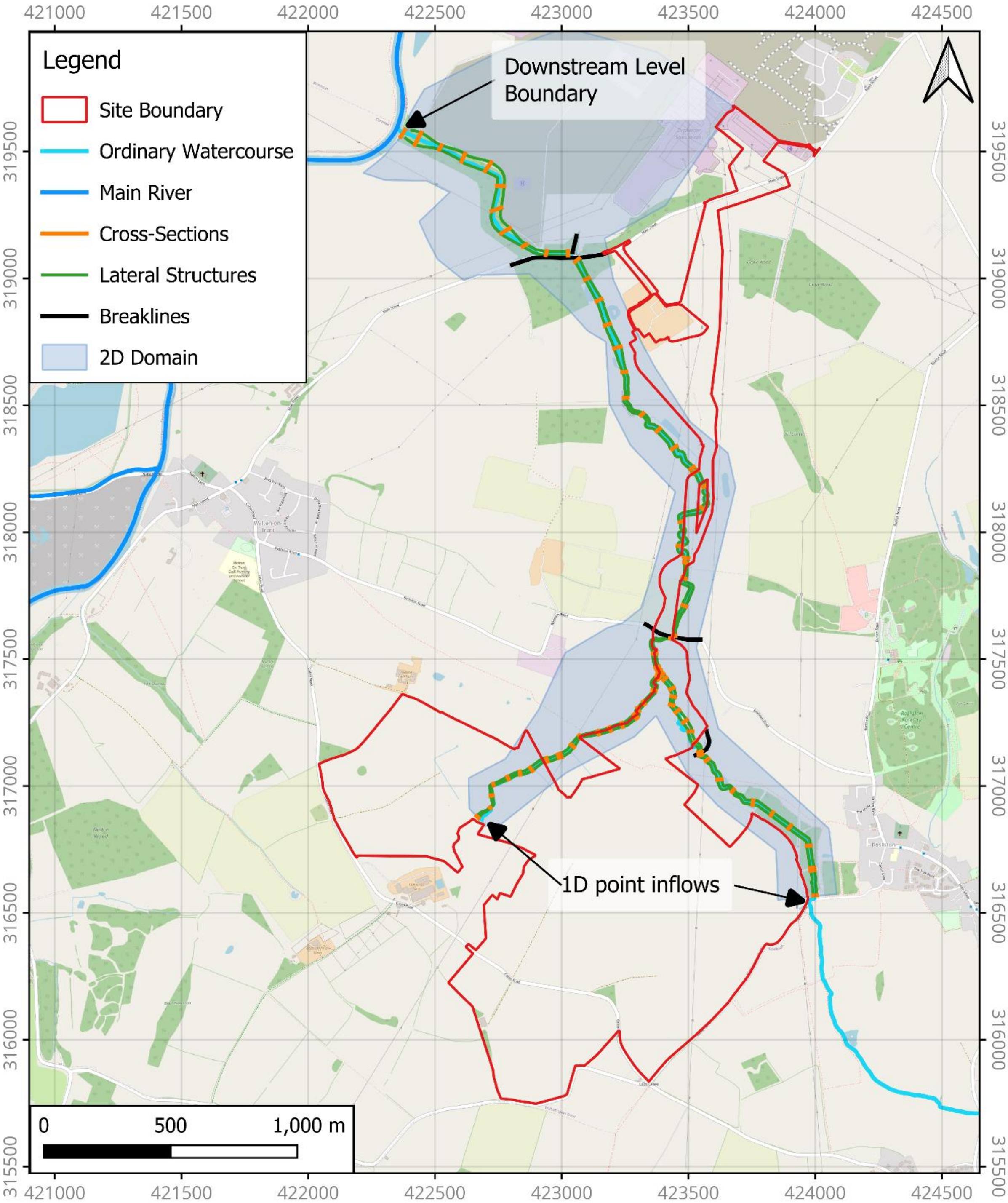


Figure 4.2 Comparison of surveyed channel sections

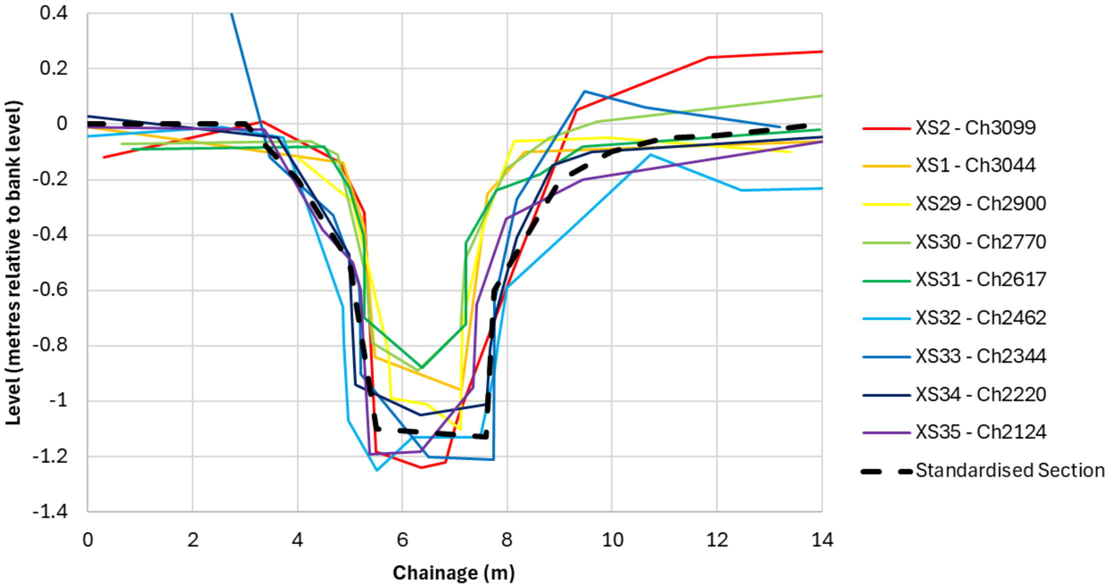
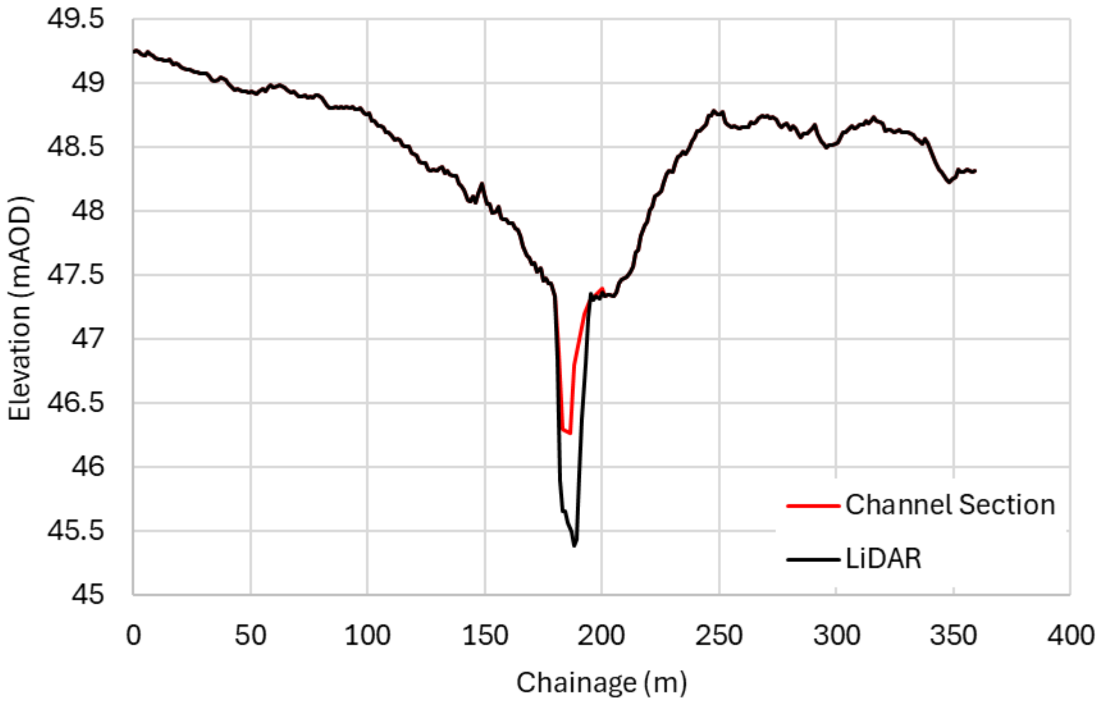


Figure 4.3 LiDAR profile and standardised section at Chainage 0 where the LiDAR profile has been retained



4.3.1.1. 1D Structures

Survey data has been collected for the structures within the key area of interest and this is summarised in Table 4-1 below.

It was not possible to get access to the culvert under Rosliston Road, however the surveyors have estimated the culvert dimension as 0.45m. This along with the channel survey shortly downstream of the culvert have been used to estimate the invert level. Survey data was also not available for the two bridges further downstream of Rosliston Road. A typical culvert dimension has been estimated for these two locations due to their distance from the key areas of interest.

Due to the uncertainty in dimensions for these three structures, an additional sensitivity test with a 50% blockage has been undertaken to understand how this may affect the flood extents and water levels upstream within the Site.

Table 4-1 Summary of 1D structures within model

River reach & chainage	Data source	Details	Photo
Ordinary WC – upper 4250	Structure 5 (LUG* – 2024)	Twin culverts protruding out of stone headwall Invert: 67.66 and 67.60 mAOD Diameter: 0.30 m Length: 5.2 m Spill level: 68.36 mAOD	
Ordinary WC – upper 3548	Structure 4 (LUG – 2024)	Invert: bed profile Soffit: 65.3 – 65.44 mAOD Width: 2.85 m Length: 2.75 m Spill level: 65.63 mAOD	

River reach & chainage	Data source	Details	Photo
Ordinary WC – upper 3236	Structure 1 (LUG – 2024)	Invert: bed profile Soffit: 63.1 mAOD Width: 3.5 m Length: 2.7 m Spill level: 63.3 mAOD	
Ordinary WC – lower 2915 Rosliston Road	Area 4 – topo survey (LUG–2024)	Invert: 59.95 mAOD (estimated from bed data) Diameter: 0.45 m (estimated by surveyors) Spill level: 61.24 mAOD	
Ordinary WC – lower 2050	No access	Estimated box culvert with width: 1.7m, height 0.7m Spill level: 56.94 mAOD (LiDAR)	
Ordinary WC – lower 1035	No access	Estimated box culvert with width: 1.7m, height 0.7m Spill level: 51.60 mAOD (LiDAR)	

River reach & chainage	Data source	Details	Photo
Tributary – upper 575	Structure 3 (LUG – 2024)	Single brick circular culvert Invert: 65.78 mAOD Diameter: 0.65 m Length: 3.44 m Spill level: 67.00 mAOD	
Tributary – upper 190	Structure 2 (LUG – 2024)	Single brick arch culvert Invert: 62.72 mAOD Width: 0.8 m Height: 0.8 m Length: 4.0 m Spill level: 63.60 mAOD	

* Land Utility Group (LUG)

4.3.2. 2D domain

A 10m x 10m 2D domain has been used to represent the out of bank domain. HEC-RAS uses a sub-grid bathymetry approach which allows for larger grid cells, whilst maintaining higher resolution detail such as hydraulic radius, volume and cross-sectional areas calculated from a finer grid terrain data (in this case 1m LiDAR data). The mapping is then also undertaken based on the finer grid terrain data to provide a higher level of detail.

The 1D river reaches have been linked to the 2D domain through lateral structures. The elevation of the lateral structures has been extracted from the LiDAR data, although in some locations it has been necessary to raise the lateral structure level to be nominally (0.01 m) above the adjacent 2D cell. HEC-RAS provides an automated tool for ensuring this.

The river banks do not form clear defined embankments, and therefore the overflow computation method has been chosen to be a 'Normal 2D Equation Domain' in preference to using a Weir Equation. A Weir Equation would be more appropriate if the banks were well defined and typically higher than the floodplain to either side.

Three breaklines have been used to align the 2D grid along features that could cause a barrier to flow and therefore pick up the higher elevations along the interface between the cells appropriately. These locations are where roads cross the floodplain and watercourse.

4.3.3. Roughness

The quantity of vegetation varies considerably within the channel even at nearby cross-sections (for example survey sections 19 and 22 in Figure 4.4 and Figure 4.5 respectively). A general approach has therefore been taken with assigning a Manning's N roughness value of 0.06 to the in-bank areas and 0.05 out of bank. No variation in Manning's N value has been applied across the 2D domain with a default value of 0.05 used. This is due to the predominantly consistent land use across the model (agricultural fields), and the relatively simple level of detail required for this study where a 1D only approach was originally considered.

Figure 4.4 Photo of river at section 19 (taken during survey)

Figure 4.5 Photo of river at section 22 (taken during survey)

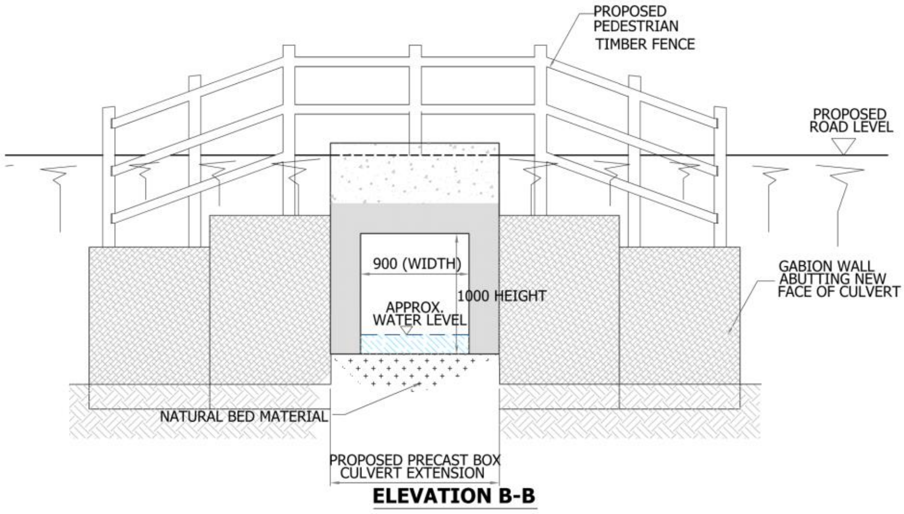
4.4. Proposed Development updates

The baseline model has been updated to incorporate the Proposed Development. This has consisted of including three proposed access track crossings along the Ordinary Watercourse at chainages 3140, 2930 and 2530. The crossings are proposed to be bottomless box culverts with an indicative design drawing provided in Figure 4.6. The initial proposed width and height of the culvert (0.9 m and 1.0 m respectively) resulted in an increase in flood levels to surrounding off-site land, and insufficient freeboard above the 1% AEP + climate change flood level. The model has therefore been used to refine these dimensions in order to limit off-Site impact, and ensure culvert soffit levels are 600mm above the 1% AEP + climate change flood level. Final proposed dimensions for the culverts are a width of 1.5m, varying height (as detailed in Table 4-2), and a spill level 0.2m above the culvert soffit level. Section 5.3 provides a discussion of the modelled impact due to the proposed crossings.

Table 4-2 Proposed culvert heights

Crossing	Model Chainage	1% AEP inc 30% CC Modelled flood level (mAOD)	Proposed culvert soffit level (mAOD)
Crossing 1 (Upstream of confluence with tributary)	3140	62.63	63.30
Crossing 2 (Near Rosliston Road)	2930	61.35	62.00
Crossing 3 (Downstream of Rosliston Road)	2530	59.41	60.00

Figure 4.6 Indicative design drawing of proposed crossings



4.5. Model runs and model performance

Table 4-3 summarises the model runs that have been undertaken. All sensitivity tests have used the baseline model. The sensitivity to flow test has not been undertaken as the intention was to use the 1% AEP with 51% Climate change (Upper end allowance) inflows however these are identical to the 0.1% AEP flows.

Table 4-3 Model Runs

Scenario	3.3% AEP	1% AEP	0.1% AEP & 1% AEP inc 51%CC	1% AEP inc 30% CC
Baseline	✓	✓	✓	✓
Proposed Development	✓	✓	✓	✓
Sensitivity – Roughness +/- 20%		✓		
Sensitivity – Downstream boundary +/- 0.25m		✓		
Sensitivity – Un-Surveyed Structures 50% blocked		✓		
Sensitivity - Inflow	No specific model run (1% AEP inc. 51% CC, represented by the 0.1% AEP model run, to be used as part of sensitivity analysis)			

Table 4-4 summarises the model performance for each run. Overall mass balance is good, with most runs showing no non-convergence. A few computational options where changed from there default values in order to achieve this level of model performance. These are detailed below, and are considered suitable for this study and unlikely to adversely affect the results:

- Maximum number of 1D iterations – 40. Whilst this has been increased to 40, the majority of the simulation time 1D iterations are 0 or 1.
- Maximum iterations between 1D and 2D – 20. This is to improve the mass balance calculations between the 1D and 2D domains.
- Number of Time Slices for the 2D flow options – 8. This increases the number of sub-timesteps that each 1D timestep can be divided into to provide a smaller timestep within the 2D domain as 2D domains typically require a smaller timestep than the 1D calculations.

The blockage sensitivity run resulted in a temporary spike in water levels as flows overtopped the blocked structures (primarily the Rosliston road culvert). This spike has been manually removed from any comparison of peak modelled water levels, as is most likely to be a numerical artifact due to the sudden increase in flows.

Table 4-4 Model Performance

Scenario	AEP	Mass Balance Error	Convergence	Comments
Baseline	3.33%	0.24%	Good	
	1%	0.35%	Good	
	1% + 30%CC	0.33%	Good	
	0.1% 1% + 51%CC	0.33%	Good	
Sensitivity – Roughness +20%	1%	0.41%	Good	
Sensitivity – Roughness -20%	1%	0.20%	Good	
Sensitivity – Downstream boundary +0.25m	1%	0.39%	Good	
Sensitivity – Downstream boundary -0.25m	1%	0.36%	Good	
Sensitivity – Un-Surveyed Structures 50% blocked	1%	0.25%	Single timestep non-	Maximum water surface error of

Scenario	AEP	Mass Balance Error	Convergence	Comments
			converged (8hrs)	0.022m at time 8hrs
Proposed Development	3.33%	0.28%	Good	
	1%	0.33%	Good	
	1% + 30%CC	0.28%	Good	
	0.1% 1% + 51%CC	0.30%	Good	

Table 4-5 summarises the hydrograph scaling factors that were required for each event, and compares the modelled and target peak flow at the flood estimation point. Modelled flows have matched target flows within 1% for all events, and overall scaling factors are small as would be expected in a small system with limited attenuation.

Table 4-5 Summary of Hydrograph Scaling factor and comparison of modelled and target flows at FEP 1

AEP	Hydrograph scaling factor						Target Flow (m^3/s)	Modelled Flow (m^3/s)
	Overall	OW – 1	OW – 2	OW – 3	TRIB – 1	TRIB – 2		
Area weighted factor:	N/A	0.56	0.07	0.19	0.07	0.11	N/A	N/A
3.33%	1.021	0.572	0.071	0.194	0.071	0.112	3.6	3.59
1%	1.018	0.570	0.071	0.193	0.071	0.112	5.0	4.98
1% + 30%CC	1.018	0.570	0.071	0.193	0.071	0.112	6.7	6.68
0.1% 1% + 51%CC	1.00	0.560	0.070	0.190	0.070	0.110	7.9	7.87

5. Results

5.1. Baseline results

Baseline modelled flood depth maps are provided in Appendix F. For the 1% and 0.1% AEP events these are compared against the existing Flood Zones 2 and 3 respectively.

5.1.1. Comparison against existing flood zones

Flood extents have significantly decreased compared to the Flood Zones 2 and 3 particularly in the upper reaches of the model. The Flood Zones are thought to have been derived from coarse national modelling, and do not represent the detail of the channel capacity or surrounding floodplain as they are likely to have been based on coarser resolution LiDAR data (for example 5m resolution).

In the reach on the Ordinary Watercourse immediately upstream of where the tributary joins, the outlines match on the right bank, but differ significantly on the left bank (adjacent to proposed solar panels). Assessment of the LiDAR data here suggests that the flood zones are unlikely to be correct, as the corresponding ground elevation level at the outer extents of the flood zones varies from 63.43 mAOD on the right bank to 64.18 mAOD on the left bank (both Flood Zone 2 and 3 are identical in this area). By contrast the modelled flood levels at this location are 63.24 for the 1% AEP event and 63.34 for the 0.1% AEP event. This location is further discussed and presented in Section 5.1.2 due to its proximity to the panel locations.

Downstream of Rosliston Road there is fairly good agreement in the width of the floodplain between the existing Flood Zones and updated modelling, particularly for the 0.1% AEP event.

The existing Flood Zones did not include the tributary, and therefore it is not possible to compare the new modelling against existing outlines in this area.

5.1.2. Detailed analysis in proximity to proposed development

The modelled flood extents come into close proximity to the proposed infrastructure (with exception of access tracks) in two key locations. The first is on the Ordinary Watercourse immediately upstream of the confluence with the Tributary. At this location the Flood zones showed that the panels were within the flood zones, although they are now shown to be at least 40m from all flood events modelled.

The second location is on the left bank of the tributary where flood zone information was not available. In this area the watercourse does not strictly follow the base of the valley, and therefore when flow overtops the left bank, it fills the area at lower elevation and forms a shallow flow path down the valley.

Figure 5.1 to Figure 5.4 provide more detail of the flood levels in relation to the terrain, and proposed development at three different representative locations. Table 5-1 details the flood depths, flood levels and compares these to the ground elevation and base level of the nearest panel. Panels will be 0.8m (+/- 0.1m) above ground level.

Figure 5.1 Location of cross-section profiles and point data

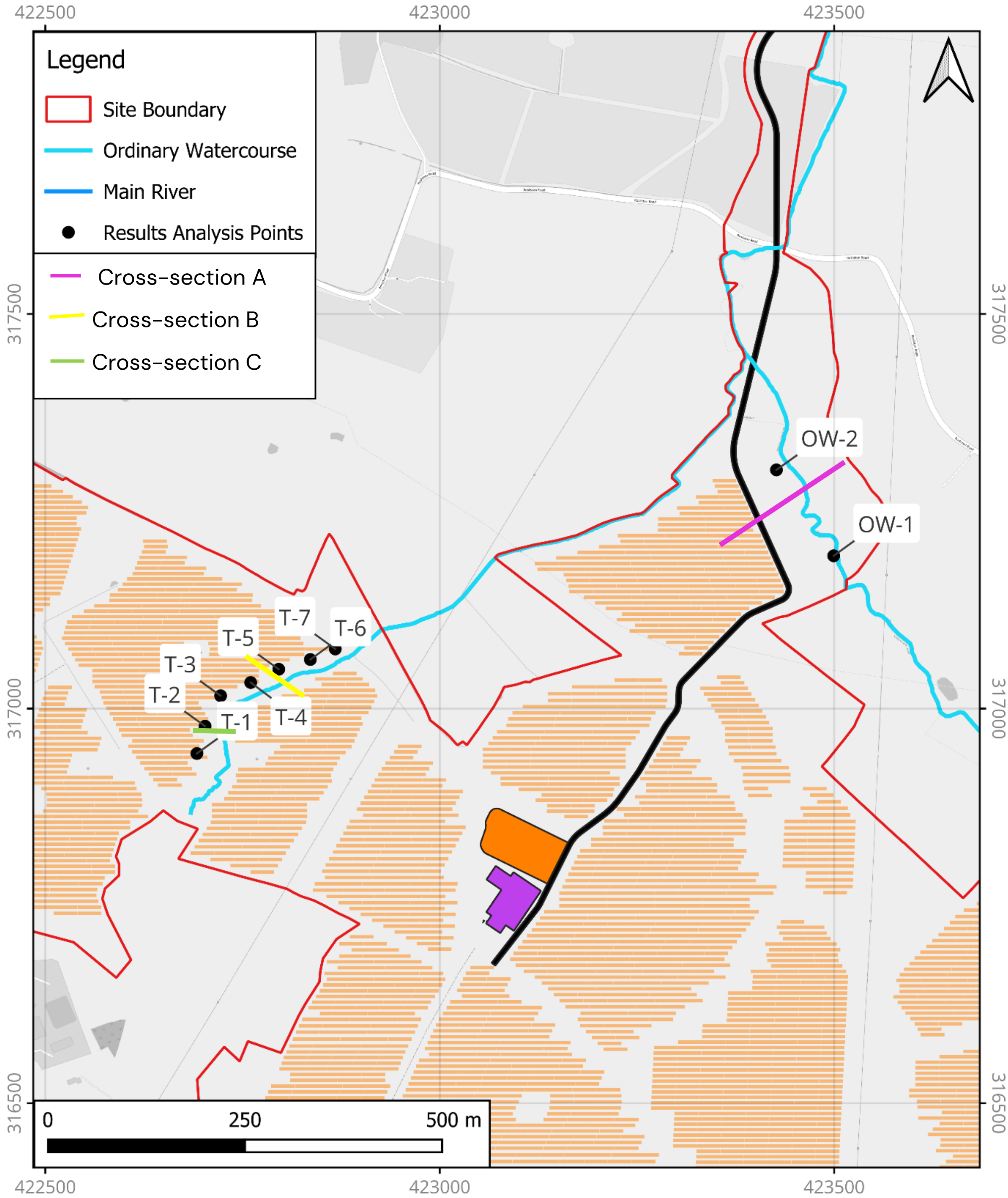


Figure 5.2 Modelled flood levels at Cross-section A

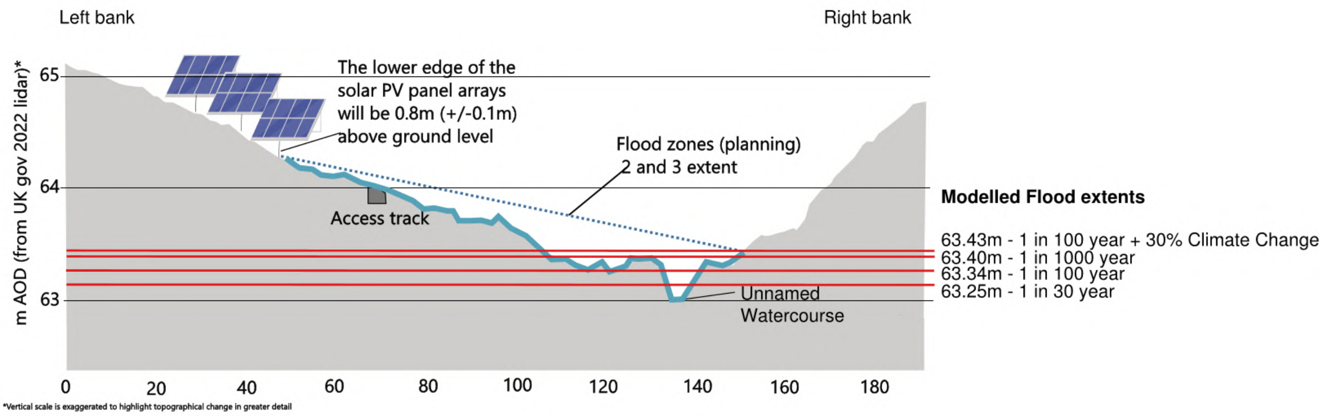


Figure 5.3 Modelled flood levels at Cross-section B (m AOD)

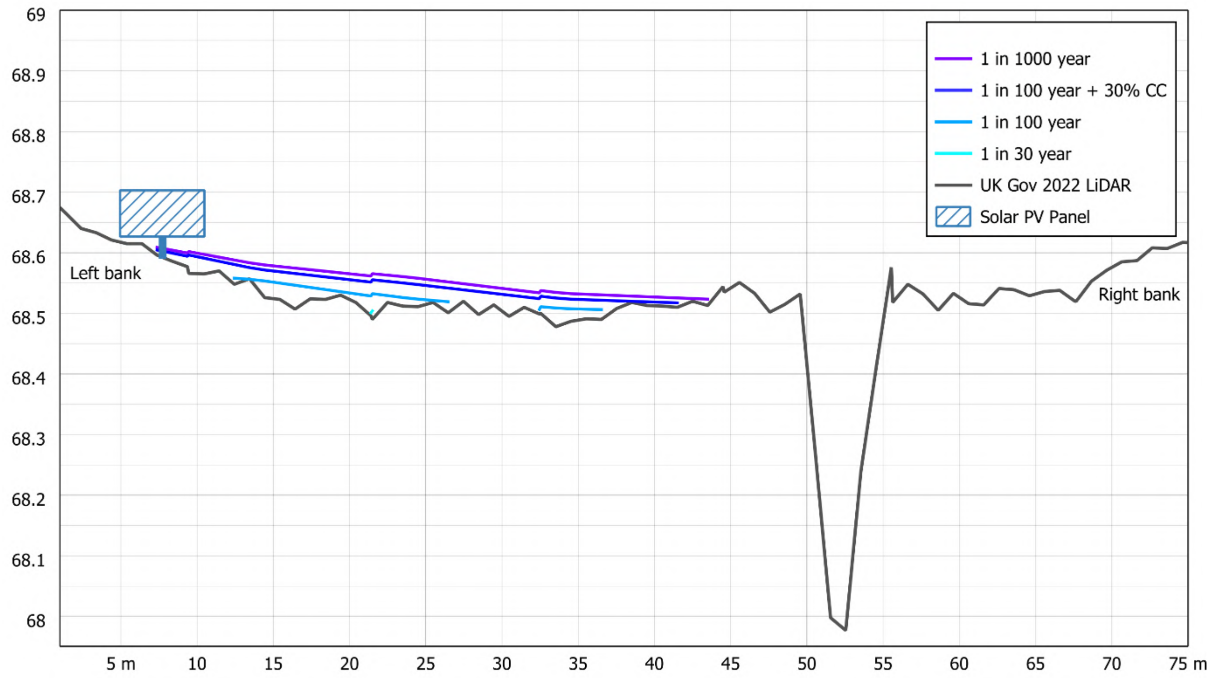


Figure 5.4 Modelled flood levels at Cross-section C (m AOD)

Table 5-1 Summary of modelled flood levels and depths at key locations

Location	Ground level (mAOD)	Ground level at nearest panel (mAOD)	Base of panel level (mAOD)	Modelled flood level (mAOD)				Modelled flood depth (m)			
				3.33%	1%	1%+30% CC	0.1%	3.33%	1%	1%+30% CC	0.1%
OW-1	64.34	65.29	66.09	64.42	64.46	64.52	64.57	0.08	0.12	0.18	0.23
OW-2	63.13	63.89	64.69	N/A	63.19	63.25	63.29	N/A	0.06	0.12	0.16
T-1	68.80	68.80	69.60	N/A	N/A	68.82	68.83	N/A	N/A	0.02	0.03
T-2	68.43	68.43	69.23	N/A	68.48	68.51	68.52	N/A	0.05	0.08	0.09
T-3	68.22	68.22	69.02	N/A	68.25	68.28	68.30	N/A	0.03	0.06	0.08
T-4	67.99	67.99	68.79	N/A	68.02	68.04	68.06	N/A	0.03	0.05	0.07
T-5	67.83	67.95	68.75	N/A	67.92	67.96	67.98	N/A	0.09	0.13	0.15
T-6	67.83	67.87	68.67	N/A	N/A	67.86	67.89	N/A	N/A	0.03	0.06
T-7	67.72	67.72	68.57	N/A	67.76	67.80	67.83	N/A	0.04	0.08	0.11

* Cells are shaded where the flood level exceeds the ground level at the nearest panels

5.2. Sensitivity testing

Maps comparing flood extents of the sensitivity model runs against the baseline are provided in Appendix E along with long-section profiles of the peak water levels. The long-section profiles have been limited to the areas of interest for each particular sensitivity test (e.g. the downstream reaches for the downstream boundary test. The following conclusions can be drawn:

- The model is sensitive to Manning's N Roughness with increases of 20% resulting in a peak level increase of 0.09 m (average 0.04 m), and a decrease of 20% resulting in a peak level decrease of 0.14 m (average 0.06 m) compared to baseline. The differences are most pronounced in areas of open channel with few structures where water levels. Differences become minimal near structures where the flow is being controlled by the structure rather than channel conveyance. Changes in flood extent are small, but spread out across the full model extent.
- The model is sensitive to changes in the downstream boundary up to a distance of 700m (for increased and decrease of boundary by 0.25 m). This is significantly downstream of the Site and areas of interest, and therefore the model results within the area of interest can be considered to be in-sensitive to the choice of downstream boundary level. Changes in flood extent are minimal and limited to the floodplain in close proximity with the River Trent.
- The model is sensitive to the structure dimensions of the three un-surveyed structures where dimensions have had to be estimated, however this is limited to areas in close proximity to each structure. Changes in peak water level are not seen upstream of where the tributary joins the main watercourse. Water levels increase (by a maximum of 0.04 m) upstream of each blocked structure, and decrease (by a maximum of 0.13 m) immediately downstream of each structure. There are some significant increases in flood extent as flow is forced onto the floodplain rather than passing through the structures. The increases are all downstream of Rosliston Road, and cause a minor increase (approximately 40 m) in the length of the access track that is passing through the floodplain. No other proposed infrastructure is located within the increased extents of this sensitivity test.

5.3. Proposed development results

The proposed development results show the impact of 3 access track watercourse crossing. Flood depth maps and a flood level comparison against baseline are provided in Appendix F, along with some long section profiles of the ordinary water-course through the area that is impacted.

The watercourse crossings do cause some localised impact to modelled peak water levels. The majority of these occur within the Site boundary, however there is also some impact (both adverse and beneficial) outside of the Site boundary. The impact is greatest is the 3.33% AEP event, and generally decreases with the larger events.

5.3.1. Flood depths

Figure 5.5 shows the impact on flood depths for the 3.33% AEP and Table 5-2 summarises the amount of area shown to be impacted, split by whether the area is within the Site or outside of the Site, and by different bands of depth change for all events.

For the 3.33% AEP event, the impact from the upstream 2 crossings is localised, and falls within the Site boundary.

The third proposed crossing (most downstream) results in additional flow overtopping the right bank, and increasing flood depths along a floodplain flow route. This flow path filters into a network of drains, and through a pond before rejoining the watercourse. The increase in flood depth along the drain network and pond is up to 0.13 m, and accounts for the vast majority of the 3279 m^2 shown to have an increase in flood depth between 0.1 and 0.3 m. The flood extents in this particular location do not show an increase, indicating that the increase in flood depth is not resulting in overtopping of the drainage network or pond. The increase in flood depth on the right bank, is mirrored by a reduction in flood depths on the left bank as the watercourse no longer overtops the left bank with the same volume as in the baseline scenario.

The impact from the larger events becomes progressively reduced with the overwhelming impact in the 0.1% AEP event being a reduction in flood levels outside of the Site boundary as flow is held back within the Site. Table 5-2 indicates that there is 955 m^2 outside of the Site where levels are increased by between 0.1 and 0.3 m. The vast majority of this is within bank between proposed crossing 2 and 3, with a reduction in flood levels in the floodplains on either side (Figure 5.6). Figures for the other events and long-section profiles can be found in Appendix F.

For all events the area outside of the Site at reduced depth of flooding exceeds the area at increased depth of flooding. For the 3.33% AEP event the proportion of land at reduced depth of flooding is 57%, this increases to 68%, 91% and 95% for the 1%, 1% with climate change and 0.1% AEP events respectively. Land that is shown to have a flood level difference of +/- 0.01 m has not been included as this has been considered to be indicative of no change.

Figure 5.5 Change in flood levels with Proposed Development – 3.33% AEP

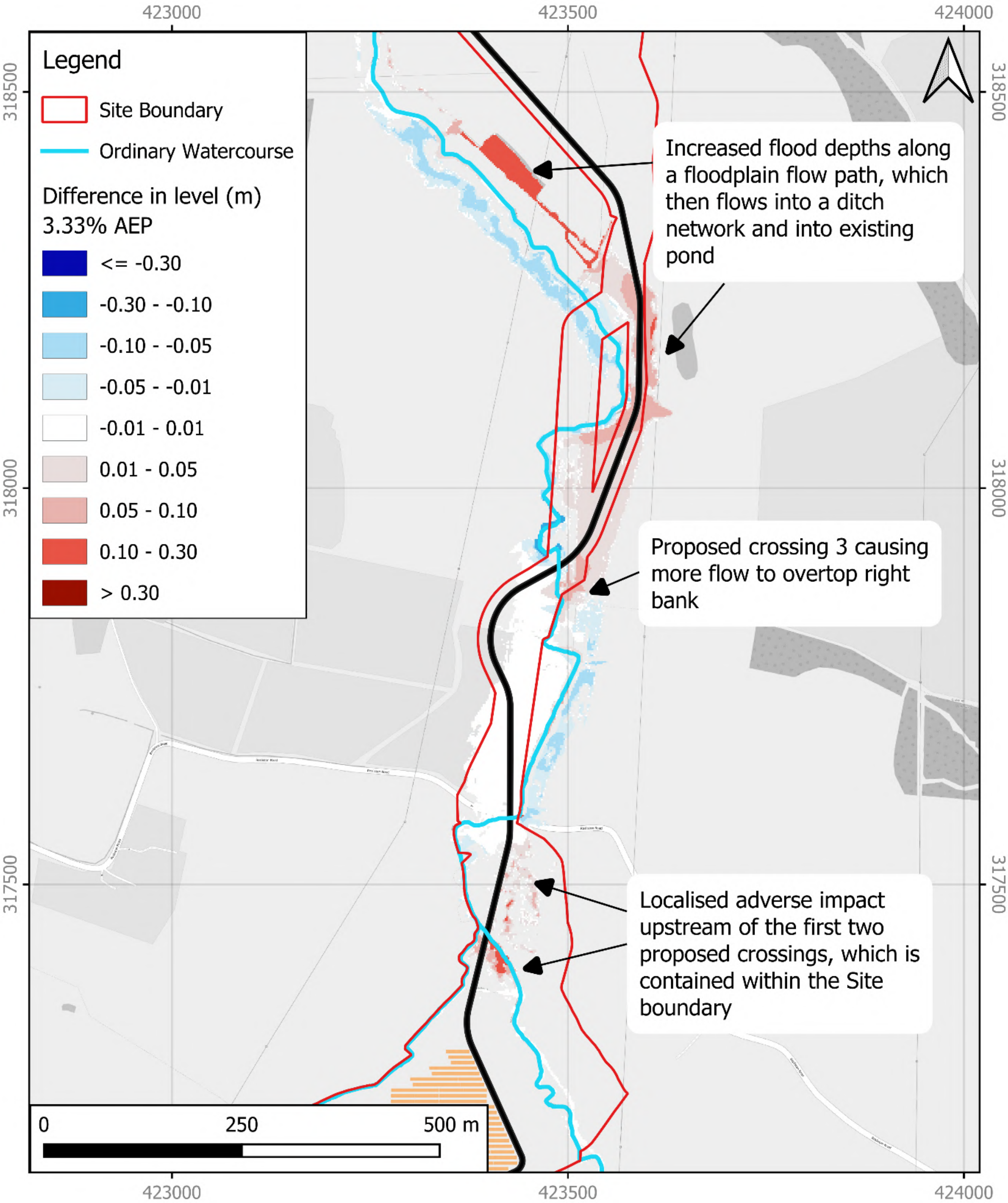


Figure 5.6 Change in flood levels with Proposed Development – 0.1% AEP

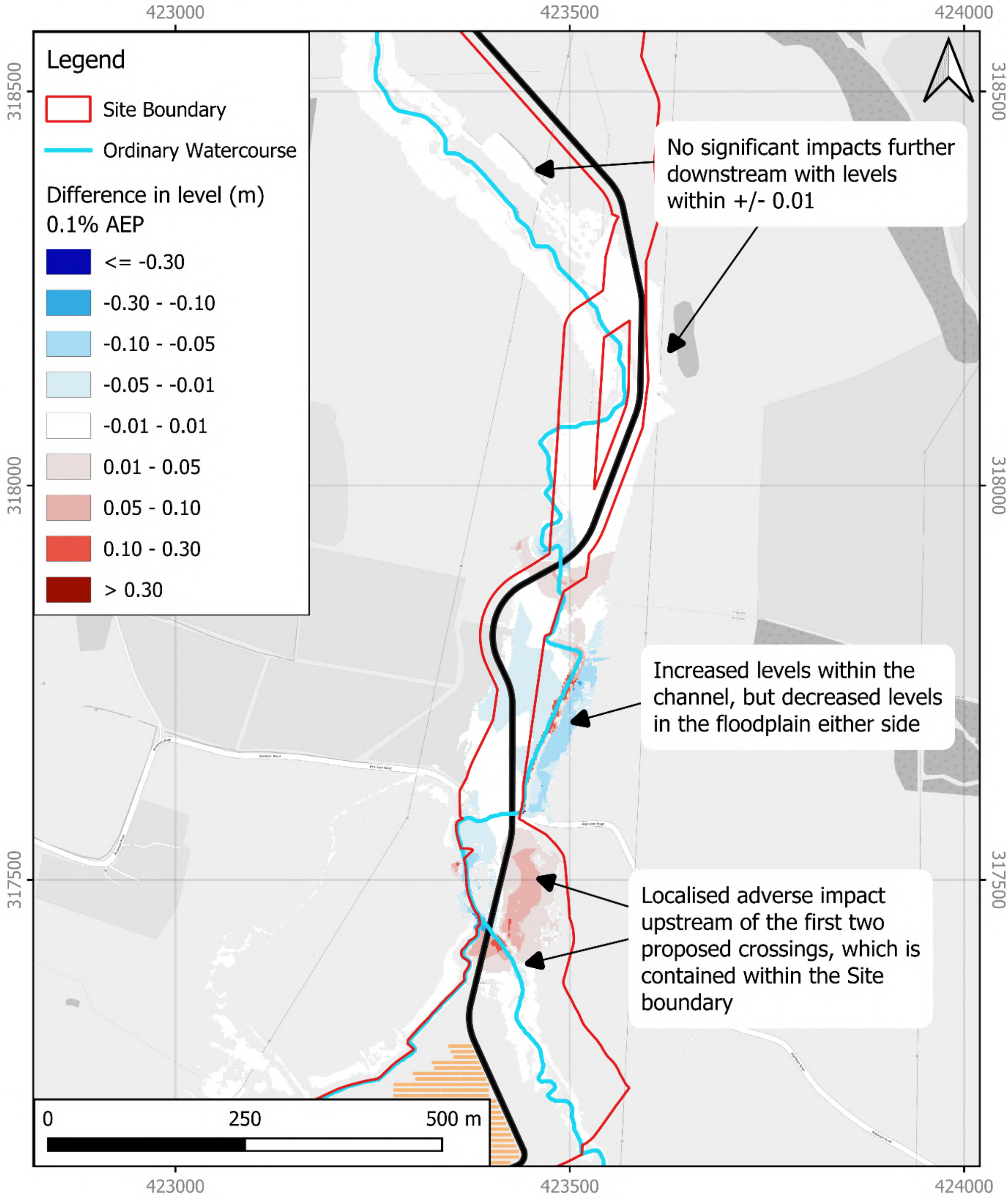


Table 5-2 Summary of impact of proposed watercourse crossings within and outside of the Site Boundary

	3.33% AEP	**1% AEP**	**1% AEP + 30% CC**	**0.1% AEP**
Total area within Site Boundary (m^2)				
Decrease greater than 0.3m	0	0	0	0
Decrease between 0.3 and 0.1 m	286	79	3	0
Decrease between 0.1 and 0.05 m	1327	1097	1020	1074
Decrease between 0.05 and 0.01 m	2900	3822	3431	10451
Increase between 0.01 and 0.05 m	12036	18378	8990	9251
Increase between 0.05 and 0.1 m	5636	3773	4100	3906
Increase between 0.1 and 0.3 m	540	547	357	278
Increase greater than 0.3m	0	0	0	0
Total area outside of Site Boundary (m^2)				
Decrease greater than 0.3m	0	0	328	0
Decrease between 0.3 and 0.1 m	359	17	173	88
Decrease between 0.1 and 0.05 m	8249	2385	2859	3922
Decrease between 0.05 and 0.01 m	9858	26930	51368	123162
Increase between 0.01 and 0.05 m	6923	12266	4365	4243
Increase between 0.05 and 0.1 m	3927	1611	724	874
Increase between 0.1 and 0.3 m	3279	25	484	955
Increase greater than 0.3m	4	12	16	20

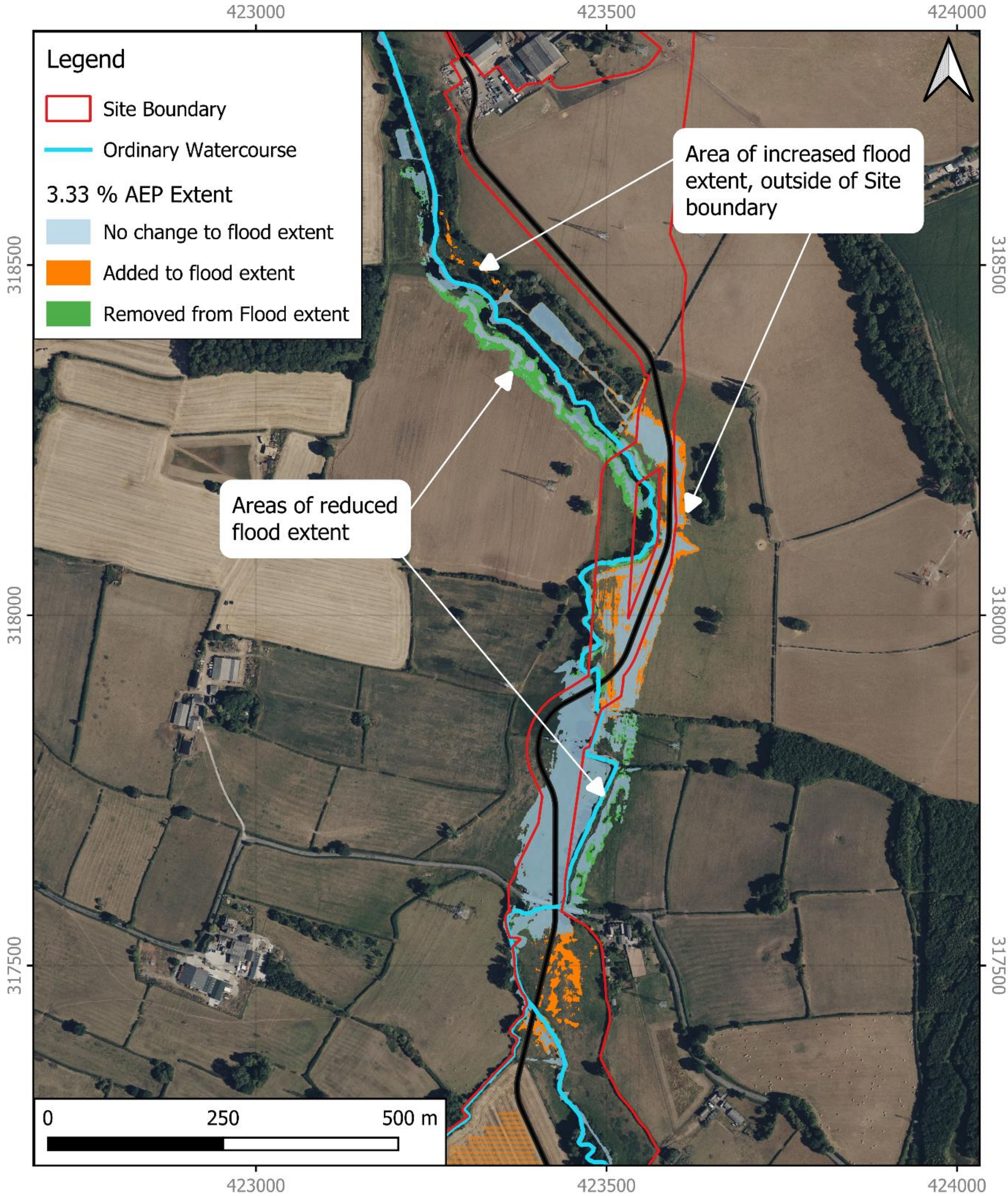
5.3.2. Flood extents

Figure 5.7 shows the change in flood extents for the 3.33% AEP event (Figures for the other events can be found in Appendix F), with areas in green removed from the flood extent, areas in orange added, and areas in blue where there is flooding in both the baseline and proposed scenarios. Table 5-3 summarises the change in flood extent areas for all events. Consistently across all events more land (approximately double) is removed from the flood extent than added when considering the impacts off-Site – therefore there is an overall net reduction in flood extent off-Site. The areas that are impacted by an increase in flood extent consist entirely of farmland or areas of woodland/vegetation, with no properties impacted.

Table 5-3 Summary of change in flood extent due to proposed watercourse crossings within and outside of the Site Boundary

	3.33% AEP	1% AEP	1% AEP + 30% CC	0.1% AEP
Decreased flood extent (on-Site) m^2	1932	1195	853	1142
Increased flood extent (on-Site) m^2	7034	5734	2521	1648
Decreased flood extent (off-Site) m^2	8291	4226	5748	4619
Increased flood extent (off-Site) m^2	3640	3715	2301	2471

Figure 5.7 Change in flood extents with Proposed Development – 3.33% AEP



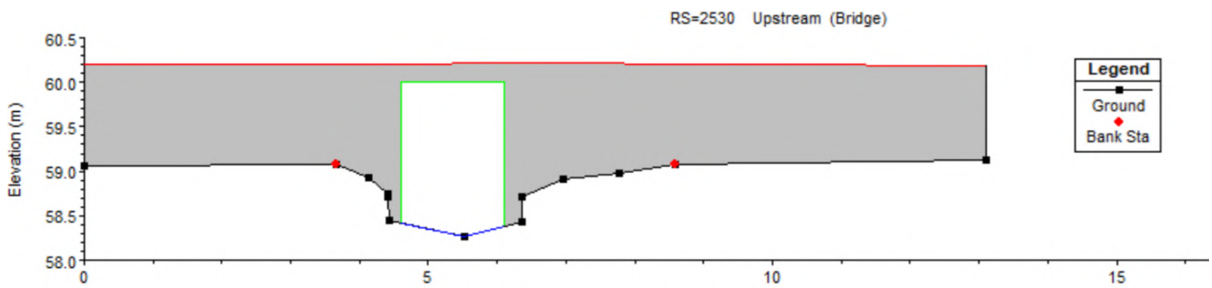
5.3.3. Design evolution of proposed culverts

A number of different options for the sizing of culverts was explored as part of the proposed development modelling before settling on the size presented above. Initially the original sizing (0.9 m x 1.0 m) was implemented however this produced excessive adverse impacts off-Site, and failed to provide 600 mm freeboard between the 0.1% AEP with climate change flood level and the culvert soffit levels. A second iteration widened the culverts to 1.5 m, but retained a low soffit height to keep the spill level over the culverts similar to bank level. This reduced the off-Site impacts, however still posed a increased risk of blockage due to not achieving the 600 mm freeboard.

The final iteration (as presented in the sections above) keeps the 1.5m width, but ensures culvert soffits are 600mm above the 0.1% AEP with climate change flood level. This scenario has produced slightly increased impacts (≈ +0.01 m in areas where flood depths are increased compared to baseline, and ≈ -0.01 in areas where flood depths are decreasing compared to baseline) compared to the second iteration, however it does reduce the potential for blockages. The bridge deck levels are typically around 1m above surrounding ground elevations, therefore there will be a requirement to incline the tracks on the approach to the crossings. This has been replicated to some extent within the hydraulic model with the bridge spill levels maintaining a higher elevation above the ground within the 1D cross-section (typically 5m either side of the banks). If it is likely that the incline will need to extend significantly beyond this, then it is recommended that small pipe culverts are laid under the track, at ground level, either side of the watercourse to allow additional floodplain flow to pass under the tracks.

The majority of the off-Site impacts are related to the third crossing (most downstream crossing). Increasing the width of the culvert was considered, however there is limited potential to widen the proposed culvert at the third crossing, without carrying out channel widening, which may have additional ecological impacts, as it is already modelled to take up the majority of the width of the watercourse (see Figure 5.8). This option was therefore not exported further.

Figure 5.8 Proposed culvert at Proposed Crossing 3



6. Assumptions and Limitations

This report is written strictly for the benefit of the Client and bound by the conditions presented in Appendix A.

6.1. Assumptions

Key assumptions relating to the hydraulic modelling for this project are outlined below. These are appropriate for the level of detail required for this project.

- Hydrological assessment
    - The approach to using ReFH2 for determining peak flows and hydrograph shapes, as opposed to undertaking a full comparison using the FEH Statistical method was agreed with the EA as appropriate for this modelling due to the limited data available and high level assessment of QMED comparisons showing ReFH2 to be conservative.
    - The distribution of flow based on an area weighted method to the modelled sub-catchments, ensuring target peak flows are met at the single flood estimation point a the downstream limit of the Site will ensure sufficiently representative flows and resultant modelled levels through the Site.
- Model Build
    - Where topographic data has not been able to be collected – due to either dense vegetation or limited land access, analysis of the profile and depth of channel compared to LiDAR data (where both topographic survey and LiDAR data are available) has been used to inform adjustments to LiDAR based cross-sections in inaccessible locations.
    - LiDAR data has been used to create flood extent and flood depth mapping for both the existing and post development scenarios. This assumes that the LiDAR data is an accurate reflection of existing ground levels and that there will be no significant changes in land elevation as part of the Proposed Development.

6.2. Limitations

Key limitations relating to the hydraulic modelling for this project are outlined below. These are considered appropriate for the level of detail required for this project.

- Hydrological assessment
    - The hydrological assessment considers only a single 'critical' storm duration and volume. This may not be representative of all likely storm profiles for the catchment.
- Model Build
    - Topographic survey could not be collected at all locations due to dense vegetation and limited land access. Suitable assumptions (as described above) have been made in these locations.
    - A generalised approach has been taken to determining Manning's 'n' roughness values along the watercourses. This is due to the limited calibration data

available, and potential for roughness values to change as the land use surrounding the watercourses is changed as part of the Proposed Development – thereby limiting the benefit of introducing additional detail to the modelling. The roughness values assigned can also only represent a 'typical' roughness which is likely to vary during the seasons with vegetation growth and maintenance activities.

- No calibration data was available for calibrating or verifying the modelling results, therefore the sensitivity analysis in particular should be used to inform the level of uncertainty likely to be present in the modelling outputs.

7. Conclusions

A 1D-2D hydraulic model has been developed for the Oaklands Farm Solar Park to model the likely flood extents and depths along the Ordinary Watercourse and its tributary thereof which flow through the Site. Hydrological analysis has been undertaken for the 3.33%, 1%, 0.1% and 1% plus climate change events for a single flow estimation point near the downstream limit of the area of interest with in the model. The hydraulic model has then been used to undertake baseline, sensitivity and proposed development model runs. The following are key conclusions from the modelling study:

- Hydrological analysis has shown that the 0.1% AEP event has near identical peak flow as the 1% AEP with upper end climate change – therefore not specific runs for the 1% AEP with upper end climate change allowance have been run (although 1% with central climate change allowance have been run)
- The baseline model shows:
  - flood extents along the Ordinary Watercourse through the area to the east of the proposed panels to be substantially reduced compared to the existing flood zone extents. This is most likely due to an improved resolution of LiDAR data, model, and appropriate representation of the 1D watercourse. The results are now more consistent with the terrain data available. The proposed panels in this location are now outside of the largest modelled event (0.1% AEP)
  - flooding over the left bank of the tributary into an area where panels are currently proposed for the 1% AEP event and larger. The maximum flood depth in this area is 0.15 m (0.1% AEP event). The bottom edge of the panels will typically sit 0.8m above ground level, and therefore will be substantially above the flood levels.
- Sensitivity analysis has shown that:
  - The model is sensitive to Manning's Roughness values with average level increases of 0.04m for an increase of 20% in roughness, and an average level decrease of 0.06m for a decrease of 20% in roughness.
  - The area of interest within the model is not sensitive to changes in the downstream boundary, and only locally sensitive to a 50% blockage assessment on three structures whose dimensions have had to be estimated.
- Proposed development model shows:
  - Localised impacts for all events, with the largest impact indicated for the 3.33% AEP.
  - Adverse Impacts from the two upstream proposed crossings is almost entirely contained within the Site boundary.
  - Impacts from the third proposed crossing (most downstream) causes additional flow to overtop the right hand bank and increases flood depths by up to 0.09 m along a floodplain flow path, and up to 0.14 m where the floodplain flow filters into a network of drainage ditches before flowing via a pond (with an increase of 0.17 m) back into the watercourse.
  - Reductions in flood depths are also modelled for all events, and outside of the Site, the area at reduced depth of flooding is greater than the area at increased depth of flooding for all events. For the larger events (1% AEP with climate change,

and 0.1% AEP) over 90% of the impacted land is modelled to have reduced flood depths.

- Overall flood extent off-Site is reduced compared to baseline, however there are areas of increased flood extent, corresponding with where flood depths are increased.
- All areas within either an area of increased flood extent, or increased flood depth are either farmland or vegetated woodland/brush. No properties are affected. The areas showing the most increase in flood depths are a series of existing ditches and a pond. None of the flood events show the increase in flood depth at these locations to cause a change in flood extent (i.e. overtopping of the ditch network or pond).

Appendix A Report conditions

Report Conditions

This report has been prepared by Aqua Terra Consulting Ltd. (Aqua Terra) in its professional capacity as soil and groundwater specialists, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client and is provided by Aqua Terra solely for the internal use of its client.

The advice and opinions in this report should be read and relied on only in the context of the report, taking account of the terms of reference agreed with the client. The findings are based on the information made available to Aqua Terra at the date of the report (and will have been assumed to be correct) and on current UK standards, codes, technology, and practices as at that time. They do not purport to include any manner of legal advice or opinion. New information or changes in conditions and regulatory requirements may occur in future, which will change the conclusions presented here.

Where necessary and appropriate, the report represents and relies on published information from third party, publicly and commercially available sources which is used in good faith of its accuracy and efficacy. Aqua Terra cannot accept responsibility for the work of others.

Site investigation results necessarily rely on tests and observations within exploratory holes only. The inherent variation in ground conditions mean that the results may not be representative of ground conditions between exploratory holes. Aqua Terra take no responsibility for variation in ground conditions between exploratory positions.

This report is confidential to the client. The client may submit the report to regulatory bodies, where appropriate. Should the client wish to release this report to any other third party for that party's reliance, Aqua Terra may, by prior written agreement, agree to such release, if it is acknowledged that Aqua Terra accepts no responsibility of any nature to any third party to whom this report or any part thereof is made known. Aqua Terra accepts no responsibility for any loss or damage incurred as a result, and the third party does not acquire any rights whatsoever, contractual, or otherwise, against Aqua Terra except as expressly agreed with Aqua Terra in writing. Aqua Terra reserves the right to withhold and/ or negotiate the transference of reliance on this report, subject to legal and commercial review.

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Appendix B Comparison of LiDAR and Survey

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The following graphs compare the surveyed cross-section data against elevation data extracted from LiDAR at 5 selected locations as detailed in Figure 2.2.

Figure B. 1 – Section 3, located on Ordinary Watercourse

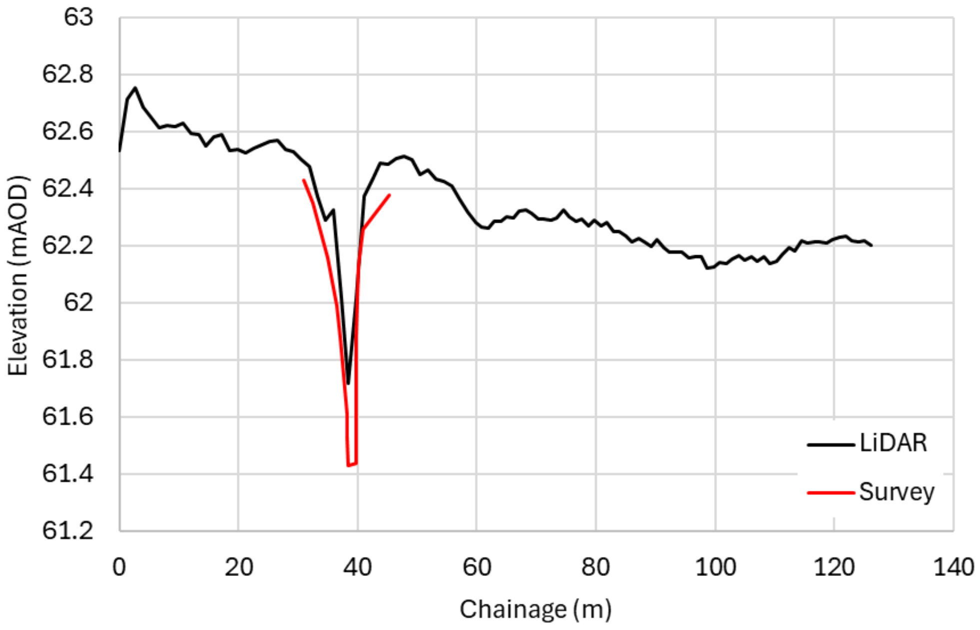


Figure B. 2 – Section 7, located on Ordinary Watercourse

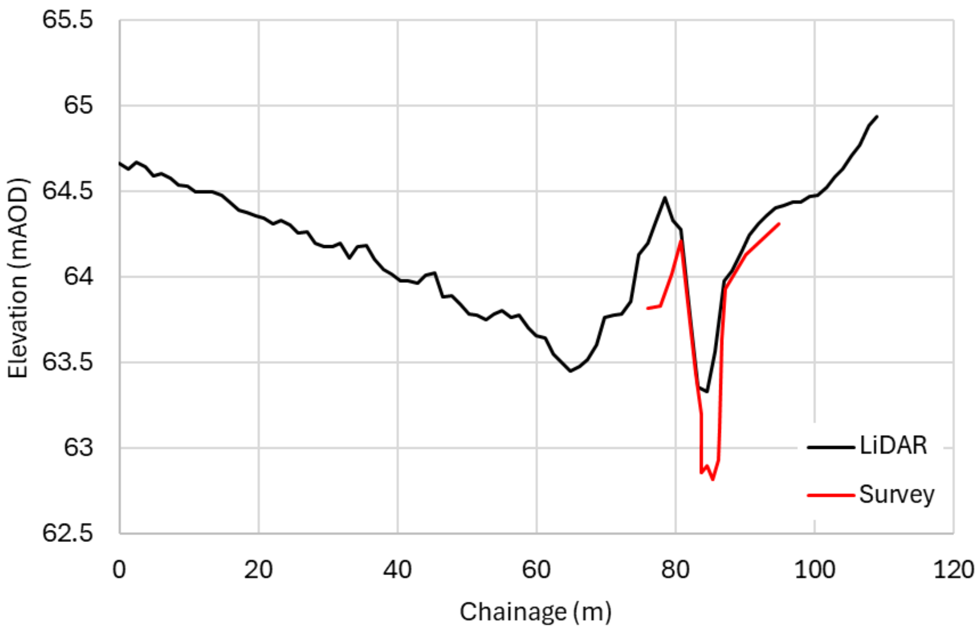


Figure B. 3 – Section 9, located on Ordinary Watercourse

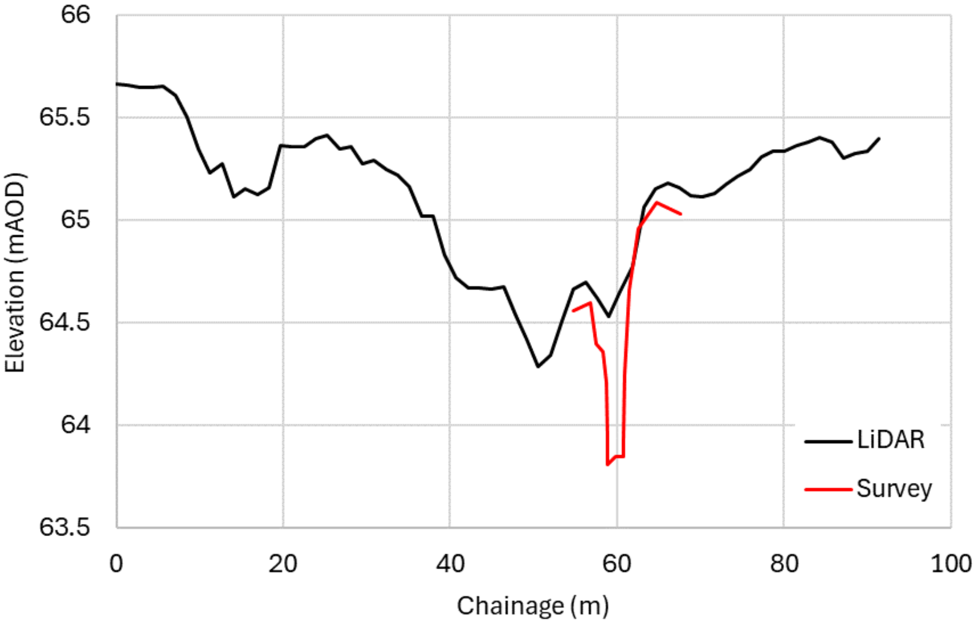


Figure B. 4 – Section 14, located on Tributary

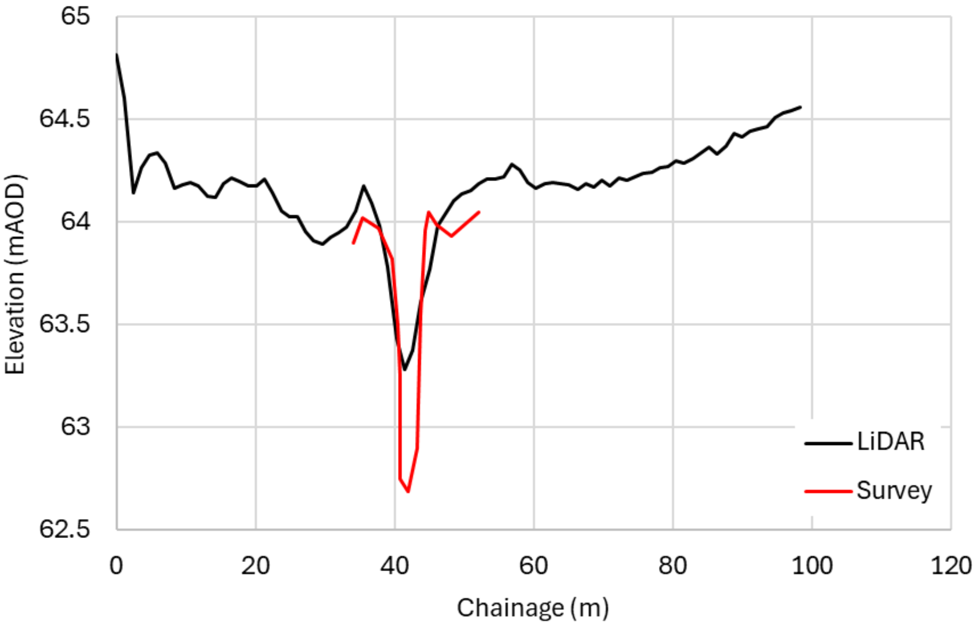
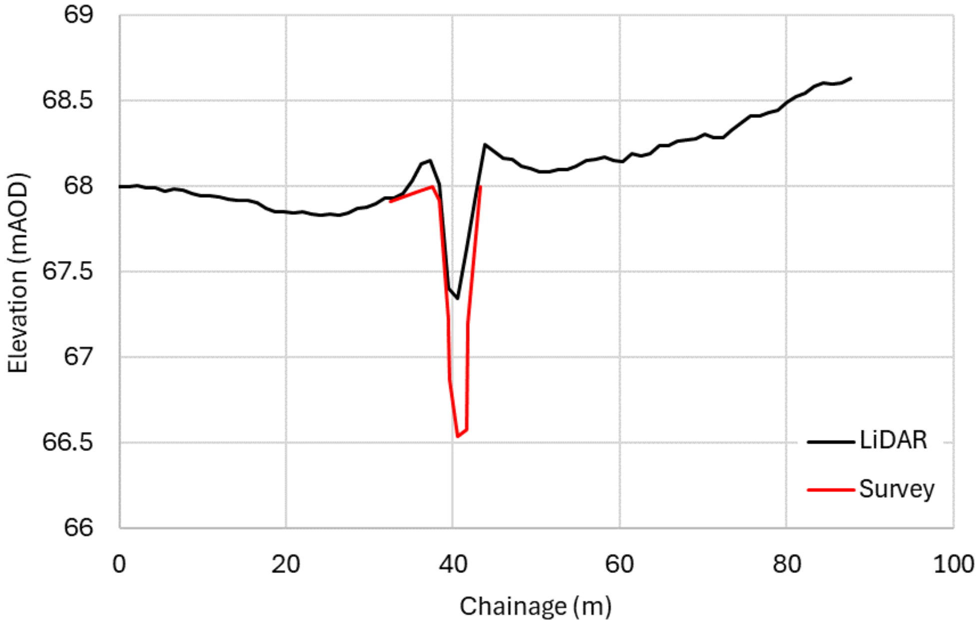


Figure B. 5 – Section 24, located on Tributary



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Appendix C Hydrology Proforma

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Flood estimation report: Oaklands Farm Solar Park

Introduction

This report template is a supporting document to the Environment Agency's Flood Estimation Guidelines. It provides a record of the hydrological context, the method statement, the calculations and decisions made during flood estimation and the results. This document can be used for one site or multiple sites. If only one site is being assessed, analysts should remove superfluous rows from tables.

Guidance notes (in red text) are included throughout this document in column titles or above tables. These should be deleted before finalising the document. Where relevant, references to specific sections of the Flood Estimation Guidelines document are included to indicate where further useful information can be found.

Note: Column size / page layout can be adapted, where necessary, to best present relevant information, for example, maps do not need to be within the tables if they would be better as a separate page.

Contents

Approval

Revision stage	Analyst / Reviewer name & qualifications	Amendments	Date
Method statement preparation	MJF (MSc, MA – cantab, CMath, MIMA)	N/A	N/A
Method statement sign-off	JM (B.Eng, FGS)	N/A	
Initial calculations preparation	MJF (MSc, MA – cantab, CMath, MIMA)	Completion of calculations following method statement approval	N/A
Initial calculations sign-off	JM (B.Eng, FGS)	N/A	
Calculations - Revision 1 preparation			N/A
Calculations - Revision 1 sign-off		N/A	

Abbreviations

AEP................................annual exceedance probability
AM..................................Annual Maximum
AREA ....Catchment area (km^2)
BFI ....Base Flow Index
BFIHOST ....Base Flow Index derived using the HOST soil classification
CPRE..............................Council for the Protection of Rural England
FARL...............................FEH index of flood attenuation due to reservoirs and lakes
FEH................................Flood Estimation Handbook
FSR................................Flood Studies Report
HOST ....Hydrology of Soil Types
NRFA ....National River Flow Archive
OS..................................Ordnance Survey
POT................................Peaks Over a Threshold
QMED ....Median Annual Flood (with return period 2 years)
ReFH ....Revitalised Flood Hydrograph method
ReFH2 ....Revitalised Flood Hydrograph 2 method
SAAR ....Standard Average Annual Rainfall (mm)
SPR................................Standard percentage runoff
SPRHOST ....Standard percentage runoff derived using the HOST soil classification
Tp(0) ....Time to peak of the instantaneous unit hydrograph
URBAN ....Flood Studies Report index of fractional urban extent
URBEXT1990 ....FEH index of fractional urban extent
URBEXT2000 ....Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH ....Windows Frequency Analysis Package – used for FEH statistical method

1 SUMMARY OF ASSESSMENT

1.1 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections. The aim of the table is to enable quick and easy identification of the type of assessment undertaken. This should assist in identifying an appropriate reviewer and the ability to compare different studies more easily.

Catchment location	Swadlincote, south of Burton-on-Trent
Purpose of study and scope	Simple hydrological assessment of ordinary watercourse flowing through proposed solar farm
Key catchment features	No key features – rural, gravity fed catchment. BFIHost – 0.469
Flooding mechanisms	Predominantly fluvial flooding from the ordinary watercourse. Some surface water flooding potential, but constrained primarily to the gulleys that feed / become the ordinary watercourse.
Gauged / ungauged	Ungauged catchment
Final choice of method	ReFH2
Key limitations / uncertainties in results	Uncertainty in flow estimate due to ungauged catchment

1.2 Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval. Results tables in this document contain both return period and AEP titles; both rows can be retained or the relevant row can be retained and the other removed, depending on the requirement of the study.

The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

2 METHOD STATEMENT

2.1 Requirements for flood estimates

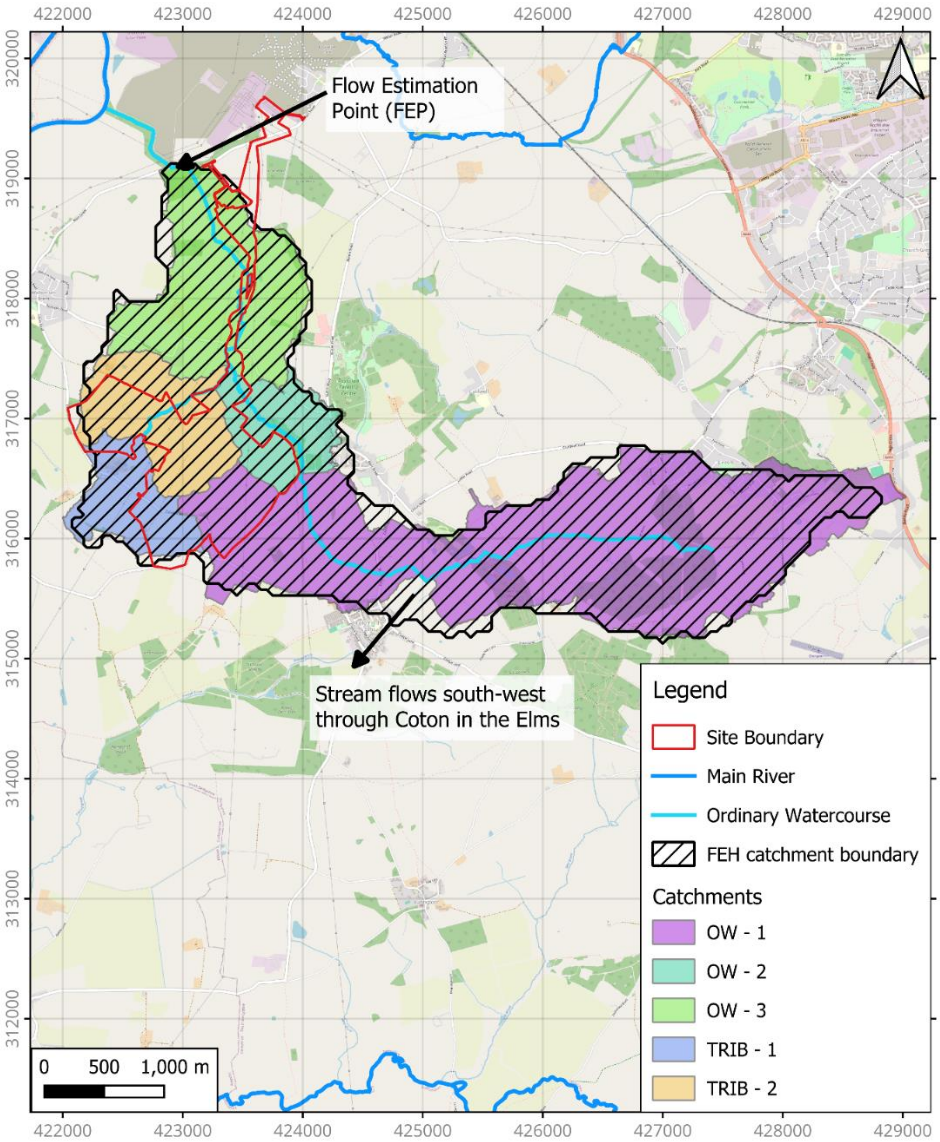
Overview	The purpose of the study is to define flow estimates for the 3.3%, 1% and 0.1% AEP events to provide inputs to a hydraulic model of an Ordinary Watercourse that flows into the Trent. The model is in support of a Flood Risk Assessment for a proposed solar farm. Existing flood zones are based on a coarse national flood risk modelling and do not sufficiently represent the detail of the Site. Both peak flows and hydrographs are required. The hydrographs are to be applied to the model and scaled to achieve a peak flow derived near the downstream limit of the study area. Climate change (Higher Central – 30% and Upper End – 51%) for the 2080s is to

	be applied for the 1% AEP event. (https://environment.data.gov.uk/hydrology/climate-change-allowances/river-flow?mgmtcatid=3090) Due to the small nature of the study area, a single flow estimate point has been chosen near the downstream limit of the study area.
Project scope	The complexity of the study is simple, and is primarily to update the flood zones within the study area and therefore give confidence that the majority of the proposed development (solar panels, sub-station etc) are located within flood zone 1. Properties at risk from flooding from the Ordinary Watercourse and/or impacts from the proposed development are negligible. No existing studies exist for the catchment (except the coarse national flood modelling) and there is no available data on the flood history of the Ordinary watercourse. Given the lack of existing data, no review of existing studies, rating reviews or flood history will be undertaken. It is also not possible to undertake ReFH model parameter estimation. Whilst joint probability with water levels on the Trent (to which the Ordinary watercourse flows into) is a possibility – the study will take a conservative approach with like-for-like return period levels assigned as a downstream boundary, and undertake sensitivity analysis on the downstream boundary.

2.2 The catchment

Description	Figure 1 provides an overview of the catchment, with the Ordinary Watercourses identified (they do not have names) along with the extent of the Site which forms the key study area of interest. The main branch of the Ordinary watercourse flows from east to west and joins the Trent downstream of the study area. There is also a tributary to the Ordinary watercourse draining the south-western corner of the catchment that flows through the Site. Topographically, the catchment is gravity drained with elevations ranging from around 120 mAOD to 40 mAOD. There is a stream which appears to flow south-west through Coton-in-the-Elms which may share the same upper catchment as the Ordinary watercourse within the study area. This is discussed in further detail in section 3.3. The catchment is predominantly rural agricultural land, however does incorporate the western limit of the village of Rosliston (mid catchment), southern portion of Linton (in the upper catchment), and the northern limit of Coton-in-the-Elms (mid catchment). The catchment is underlain by the Edwalton Member (Siltstone and very fine-grained sandstone in the west and the Gunthorpe Member (Mudstone) in the east. Superficial deposits cover a portion of the site, comprising fluvioglacial diamicton in the south and some areas of alluvium in the north, typically along the watercourses through the Site. The soils close to the watercourses are described as “slowly permeable, seasonally wet, with impeded drainage”, whilst those away from the watercourses are described as “loamy and clayey soils with slightly impeded drainage”.

Figure 1 – Catchment map including Flow Estimation Points



2.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 12.1, released 2nd November 2023. This contains data up to end of September 2022.

2.4 Gauging stations (flow or level)

Water-course	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
Blithe	Hamstall Ridware	4002	28002	163	Rated	01/1937 - present

2.5 Data available at each flow gauging station in Table 2.4

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
Blithe @ Hamstall Ridware	1937 - 2022	No	Yes	No	No	Rating does not account for bypassing and is increasingly uncertain beyond QMED, however excellent fit to gaugings and QMED estimates thought to be reliable. Only data pre 1952 can be used as post 1952 heavily influenced by Blithfield Reservoir.

2.6 Rating equations

Catchment is ungauged – no rating reviews undertaken or rating equations used

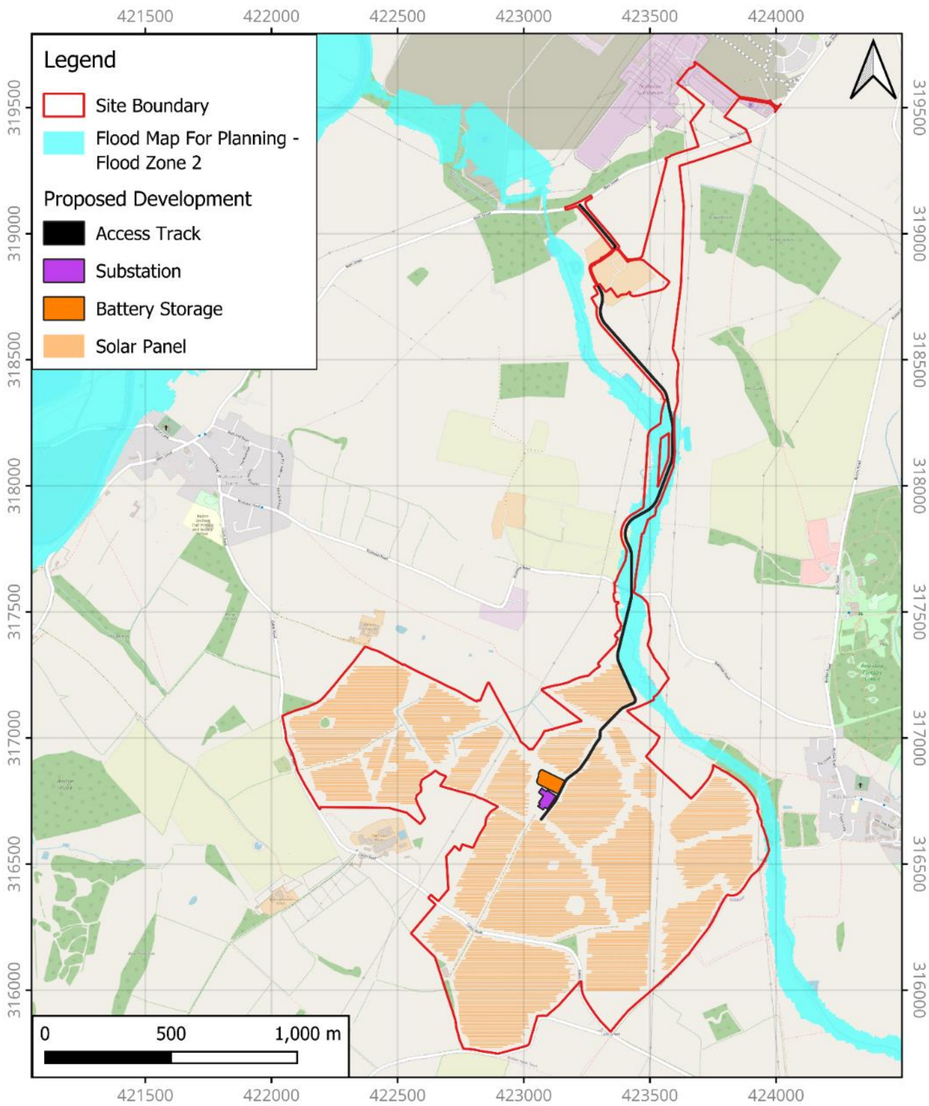
2.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Check flow gaugings	N/A			No gauges within study area
Historical flood data	N/A			No historical flood data available
Flow or river level data for events	N/A			No gauges within study area
Rainfall data for events	N/A			ReFH2 Calibration utility not being used – no calibration data
Potential evaporation data	N/A			ReFH2 Calibration utility not being used – no calibration data
Results from previous studies	N/A			No past studies for study area
Other data or information	N/A			None required for this simple assessment

2.8 Hydrological understanding of catchment

Conceptual model	The main area of interest within the catchment is where the Ordinary watercourse and it's tributary flow through the Site ownership boundary, and therefore where proposed infrastructure on Site such as solar panels may be at risk of flooding. Figure 2 details the proposed development extents relative to the existing Flood Zone 2. It should be noted that the tributary to the ordinary watercourse (which passes to the west of the battery storage and sub-station) was not included as part of the national Flood Zone mapping. The valley containing the watercourses is typically well incised with limited overland flow paths or embankments artificially holding back flow. The cause of flooding is therefore most likely to be due to peak flows rather than volumes.

Figure 2 – Proposed development extents



Unusual catchment features	The catchment has no unusual features. It is gravity fed, essentially rural and has no major urban areas.

2.9 Initial choice of approach

Is FEH appropriate?	Both FEH and ReFH2 are applicable methods as the catchment is fairly standard with no unusual features.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	Initial choice of method is the ReFH2 approach. This is due to the simple analysis required for the study, and focus on larger return periods for updating the flood zone extents within the study area. The catchment is ungauged and therefore whilst the FEH ungauged approaches would be suitable, these cannot be supported with gauged data on the catchment. The ReFH2 approach will however be compared against a donor adjusted QMED value derived using the FEH Statistical method. Hydrograph shapes will be derived using ReFH2 with default parameters, and then scaled to ensure target peak flows are met at the flood estimate point. The catchment will be split into sub-catchments, however this is only for the purpose of distributing the derived hydrograph according to area weighting. Where a sub-catchment forms the upstream limit of a model this will be a direct inflow, and where the sub-catchment is a intervening catchment, it will be distributed linearly along the watercourse within that sub-catchment.

	The catchment overall is small without any distinctive changes in characteristics between sub-catchments – therefore the additional detail of deriving individual hydrographs and/or target peak flows is not thought to provide any additional benefit to the study (in particular due to the lack of any gauged data to support that level of detail). A single storm duration will be used, representative of the critical duration at the flood estimation point at the downstream limit of the Site.
Software to be used (with version numbers)	FEH Web Service[1] / ReFH2 (v4)

3 LOCATIONS WHERE FLOOD ESTIMATES REQUIRED

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space. Figure 1 shows the location of the Flood Estimation Point

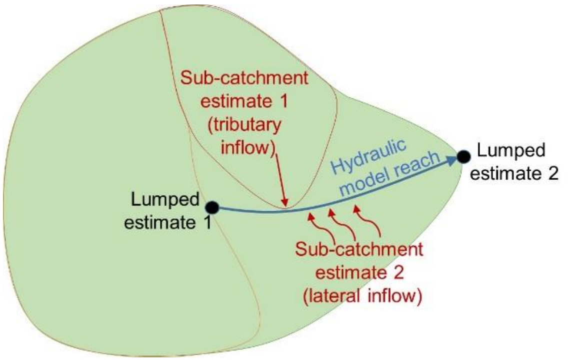
3.1 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
FEP 1	L	Ordinary Watercourse	Downstream limit of study area	422900	319100	9.97	9.69

Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required.

Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced.

The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.

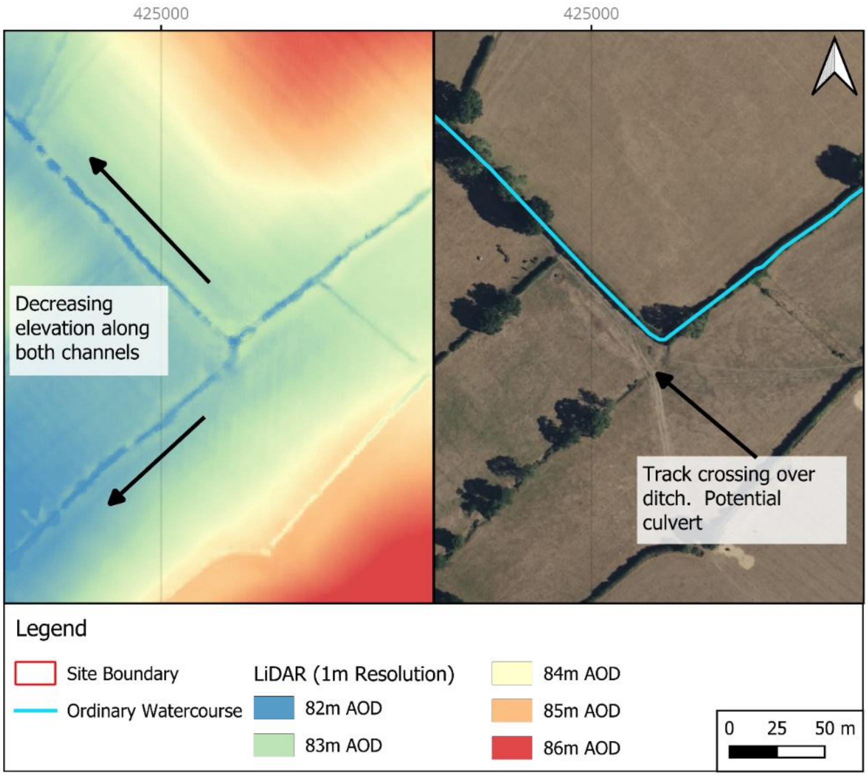


3.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 1990 Delete if not required	URBEXT 2000	FPEXT
FEP 1	1	0.3	0.455	4.9	28.9	641	N/A	0.021	0.0912

[1] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	The catchment boundary has been checked against LiDAR data by carrying out GIS watershed analysis (Figure 1). The resulting extent is broadly similar, although there are some areas of difference. In particular there is a difference where a stream heads south-west through the village of Coton-in-the-Elms. This stream appears to originate very close (or even connected) to the course of the Ordinary Watercourse modelled within this Study. Figure 3 shows a detail of the LiDAR and aerial imagery of the location. As a conservative measure, the full catchment to the east of this location has been assume to contribute to the modelled watercourse, with no flow lost to the neighbouring stream. This would be the case if either there is no connection between the two watercourses, or if there were a connection, the culvert were to be blocked. Whilst there is a change in area (of 3%) DPLBAR has not been updated. This is due to the catchment shape being substantially similar to the FEH catchment shape, and considerably different from a 'standard' tear-drop shaped catchment. It is therefore likely that the FEH DPLBAR is more representative than applying a generic equation based on overall area. **Figure 3 – Catchment map including Flow Estimation Points** 
Record how other catchment descriptors were checked and describe any changes.	No changes to other catchment descriptors were made. They were checked against data from the BGS (Bedrock and superficial geology) and Cranfield Soils data.
Source of URBEXT	URBEXT2000 – No significant urban development in the area, therefore no need to apply the URBAN50k method
Method for updating of URBEXT	CPRE formula from 2006 CEH report on URBEXT2000 Updated URBEXT value for 2024 is 0.0220

4 STATISTICAL METHOD

4.1 Application of Statistical method

What is the purpose of applying this method?	This approach has been used to provide a simple check on the ReFH2 derived QMED at FEP 1. As such the full flood frequency analysis of the statistical method has not been undertaken. The study is predominantly focussed on the larger events (1% and 0.1%) where the ReFH2 approach is considered to be most appropriate (due to limited long term nearby gauged records for smaller watercourses such as those within the study area).

4.2 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjust-ment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor			
						Weight	Weighted ave. adjustment		
FEP 1	1.57	DT	28002	27.05	0.87	N/A	N/A	1.2	1.12

Are the values of QMED spatially consistent?	N/A – Single catchment only
Method used for urban adjustment for subject and donor sites	WINFAP v4[2]

Parameters used for WINFAP v4 urban adjustment if applicable

Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010)**Error! Bookmark not defined.** in which PRUAF is c alculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016)[2].

4.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	The Blithe at Hamstall Ridware has been considered as a donor site based on its proximity to the target site (27km), location within the wider Trent catchment, and similarity in catchment descriptors such as BFIHOST19 (0.455 vs 0.481), FARL (1.00 vs 0.998), and SPRHOST (39.41 vs 38.16). The catchment is on the slightly larger side (162 km^2 vs 10 km^2), however there are limited similar gauged catchments of a more comparable size available nearby.

[2] Wallingford HydroSolutions (2016). WINFAP 4 Urban adjustment procedures.

4.4 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
28002	AM	No	17.5	29.42	0.59

4.5 Derivation of pooling groups

No pooling groups have been derived as the FEH statistical method has only been used as a check against the ReFH QMED value.

4.6 Derivation of flood growth curves at subject sites

No flood growth curves derived at subject sites using the FEH statistical method.

4.7 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)			
	2	30	100	1000
	Flood peak (m^3/s) for the following AEP (%) events			
	50	3.3	1	0.1
FEP 1	1.12	N/A	N/A	N/A

5 REVITALISED FLOOD HYDROGRAPH (REFH) METHOD

The ReFH method has not been applied for this study.

6 REVITALISED FLOOD HYDROGRAPH 2 (REFH2) METHOD

6.1 Application of ReFH2 method

What is the purpose of applying this method?	The ReFH2 method has been applied to produce lumped flow estimates at FEP 1 and to create an inflow hydrograph which will be distributed across the model inflows and intervening sub-catchments.

6.2 Catchment sub-divisions for ReFH2 model

Catchment is essentially rural – therefore no sub-division for urban areas undertaken

6.3 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp}	BL (hours)	BR
FEP 1	CD	6.446	N/A	373.122	N/A	47.162	2.179
Brief description of any flood event analysis carried out			N/A				
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

6.4 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
FEP 1	Rural	Winter	11

6.5 Design events for ReFH2 method: Sub-catchments and intervening areas

ReFH2 is not being applied for sub-catchments (A single ReFH2 hydrograph is being derived for FEP 1 and then distributed by area to the upstream sub-catchments. This hydrograph will then be scaled to meet the target flows at FEP 1)

6.6 Flood estimates from the ReFH2 method

Site code	Flood peak (m^3/s) for the following return periods (in years)			
	2	30	100	1000
	Flood peak (m^3/s) for the following AEP (%) events			
	50	3.3	1	0.1
FEP 1	1.62	3.55	4.97	7.93

7 DISCUSSION AND SUMMARY OF RESULTS

7.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years / 50% AEP			Return period 100 years / 1% AEP		
	ReFH2	FEH	ReFH2 / FEH	ReFH2	FEH	ReFH2 / FEH
FEP 1	1.62	1.12	1.45	4.97	N/A	N/A

7.2 Final choice of method

Choice of method and reasons	The final choice of method has been the ReFH2 approach. This provides a conservative estimate of QMED (closer to that derived from catchment descriptors, and almost 50% more than the FEH Donor adjusted QMED estimate). The ReFH2 method also has the benefit of potentially being more reliable at higher return periods such as the 1% and 0.1% AEP which are the principal purpose of this study where the FEH Statistical method is at its limit based on data record length.
How will the flows be applied to a hydraulic model?	The flows will be applied to the hydraulic model by proportioning the hydrograph derived at FEP 1 to sub-catchments based on an area weighting. These will be applied as direct inflows at the upstream limit of the Ordinary Watercourse and its tributary, and as a distributed inflow across intervening catchments. The input hydrograph will be scaled to ensure that the derived target peak flows at FEP 1 are met within 1% - therefore the final applied hydrograph may differ in magnitude than the hydrographs presented in section 7.6.

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumption for this study is that ReFH2 is a suitable hydrological model for deriving both target flow estimates and hydrographs, particularly given the lack of gauged data to verify this against.
Discuss any particular limitations	The use of any hydrological model or analysis for return periods such as the 0.1% AEP should be treated with caution and as a 'best estimate'. The methods

	adopted however are suitable for use within the catchment.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	Uncertainty bounds are difficult to establish for the ReFH2 methodology for ungauged catchments, however based on the discussion on confidence intervals for ReFH/ReFH2 (page 107 of LIT 11832 – Flood Estimation guidelines) the 95% upper and lower bounds presented in the table for design flows at ungauged sites based on the FEH method have been adopted. This provides the following upper and lower bounds: 50% AEP: 0.85 – 3.73 m^3/s 33.3% AEP: 1.60 – 7.82 m^3/s 1% AEP: 2.24 – 11.08 m^3/s 0.1%AEP: 3.41 – 18.49 m^3/s
Comment on the suitability of the results for future studies	The analysis undertaken in this study is for the particular purpose of improving the flood zones and therefore informing development layout for a proposed solar farm, considered 'Essential Infrastructure' under the NPPF classification. It should not be used for defining flood risk for Highly Vulnerable, More Vulnerable or Less Vulnerable development in the area which may contain residential properties or be accessed frequently by members of the public.
Give any other comments on the study	If the proposed study is to be taken and used for assessment of more vulnerable developments, then a full hydrological assessment including application of the full FEH Statistical method would be recommended.

7.4 Checks

Are the results consistent, for example at confluences?	No confluences within the study area, and no other data to check outputs against.
What do the results imply regarding the return periods / frequency of floods during the period of record?	No flow gauging data available to make comparisons
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	1% AEP growth factor is 3.07 – this is a very typical value.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The 0.1% / 1% AEP ratio is 1.60 – this is within a typical expected range.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No previous studies to compare against. Hydraulic modelling report will briefly discuss a comparison of new flood zone extents against existing extents.
Are the results compatible with the longer-term flood history?	No long-term flood history available
Describe any other checks on the results	The hydraulic model results will be sense-checked against existing flood zone extents to ensure that results are comparable (whilst acknowledging that this study is intended to update the flood zones, hence a direct match is not expected)

7.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)			
	2	30	100	1000
	Flood peak (m^3/s) for the following AEP (%) events			
	50	3.3	1	0.1
FEP 1	1.6	3.6	5.0	7.9

7.6 Uncertainty bounds

This table reports the flows derived from the uncertainty analysis detailed in Section 7.3. The 'true' value is more likely to be near the estimate reported in Section 7.5 than the bounds. However, it is possible that the 'true' value could still lie outside these bounds.

Site code	Flood peak (m^3/s) for the following return periods (in years)							
	2		30		100		1,000	
	Flood peak (m^3/s) for the following AEP (%) events							
	50		3.33		1		0.1	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
FEP 1	0.85	3.73	1.60	7.82	2.24	11.08	3.41	18.49

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hydrographs are presented in the table and graph below. Note that the 1% AEP + 51% CC curve is indistinguishable from the 0.1% AEP event with peak flows differing by 0.01 m^3/s. It is therefore proposed to only the 0.1% AEP event and to use those results as representative of both events.

Time (hrs)	Flow (m^3/s) for the following AEP (%) events					
	50% AEP	3.33% AEP	100% AEP	1% AEP + 30% CC	1% AEP + 51% CC	0.1% AEP
0	0.27	0.27	0.27	0.27	0.27	0.27
1	0.27	0.27	0.27	0.27	0.28	0.28
2	0.27	0.28	0.29	0.30	0.31	0.31
3	0.29	0.32	0.35	0.38	0.40	0.40
4	0.32	0.40	0.46	0.53	0.57	0.57
5	0.40	0.56	0.67	0.81	0.91	0.91
6	0.52	0.82	1.05	1.31	1.50	1.50
7	0.70	1.20	1.59	2.04	2.37	2.37
8	0.91	1.66	2.25	2.93	3.44	3.44
9	1.14	2.16	2.96	3.90	4.60	4.60
10	1.37	2.65	3.66	4.87	5.76	5.76
11	1.57	3.09	4.30	5.74	6.82	6.81
12	1.72	3.41	4.76	6.38	7.59	7.59
13	1.78	3.55	4.97	6.67	7.94	7.93
14	1.77	3.55	4.96	6.65	7.92	7.91
15	1.72	3.44	4.80	6.44	7.65	7.65

Time (hrs)	Flow (m^3/s) for the following AEP (%) events					
	50% AEP	3.33% AEP	100% AEP	1% AEP + 30% CC	1% AEP + 51% CC	0.1% AEP
16	1.65	3.27	4.56	6.09	7.22	7.22
17	1.55	3.07	4.26	5.67	6.71	6.71
18	1.46	2.86	3.95	5.23	6.18	6.18
19	1.37	2.67	3.67	4.84	5.70	5.70
20	1.30	2.50	3.43	4.50	5.28	5.28
21	1.23	2.35	3.21	4.20	4.91	4.91
22	1.16	2.21	3.00	3.92	4.57	4.57
23	1.10	2.07	2.81	3.64	4.24	4.24
24	1.03	1.94	2.61	3.37	3.91	3.91
25	0.97	1.80	2.41	3.10	3.58	3.58
26	0.90	1.66	2.21	2.83	3.26	3.26
27	0.84	1.53	2.02	2.57	2.94	2.94
28	0.78	1.40	1.84	2.33	2.65	2.65
29	0.73	1.30	1.70	2.13	2.42	2.42
30	0.70	1.23	1.60	1.99	2.25	2.25
31	0.67	1.18	1.52	1.89	2.13	2.13
32	0.65	1.14	1.47	1.82	2.04	2.04
33	0.63	1.11	1.43	1.76	1.98	1.98
34	0.62	1.08	1.39	1.72	1.94	1.93
35	0.61	1.06	1.36	1.68	1.89	1.89
36	0.59	1.04	1.34	1.65	1.85	1.85
37	0.58	1.01	1.31	1.61	1.82	1.82
38	0.57	0.99	1.28	1.58	1.78	1.78
39	0.56	0.97	1.25	1.55	1.74	1.74
40	0.55	0.95	1.23	1.51	1.70	1.70
41	0.53	0.93	1.20	1.48	1.67	1.67
42	0.52	0.91	1.18	1.45	1.63	1.63
43	0.51	0.89	1.15	1.42	1.60	1.60
44	0.50	0.87	1.13	1.39	1.57	1.57
45	0.49	0.86	1.10	1.36	1.53	1.53
46	0.48	0.84	1.08	1.33	1.50	1.50
47	0.47	0.82	1.06	1.31	1.47	1.47
48	0.46	0.80	1.04	1.28	1.44	1.44

Appendix D Model Chainage Table

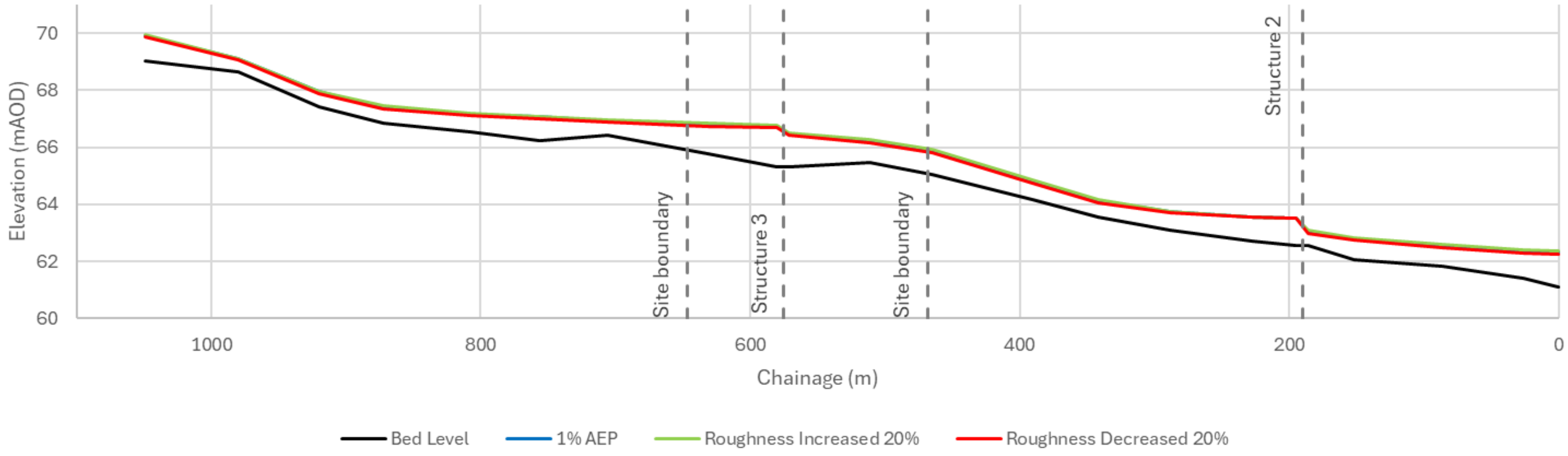
River Reach	Surveyed Section	Section Type	Chainage	Reach length (to next cross-section)
Ordinary Watercourse - Upper	45	XS	4354	94
	44	XS-copy	4260	19
	Structure 5	Culverts	4250	
	44	XS	4241	91
	43	XS	4150	111
	42	XS	4039	88
	41	XS	3951	90
	40	XS	3861	97
	39	XS	3764	82
	38	XS	3682	96
	37	XS	3586	28
	36	XS - Copy	3558	18
	Structure 4	Bridge	3548	
	36	XS	3540	59
	9	XS	3481	52
	8	XS	3429	64
	7	XS	3365	63
	6	XS	3302	56
	5	XS - copy	3246	16
	Structure 1	Bridge	3236	
	5	XS	3230	28
	4	XS	3202	48
	3	XS	3154	24
	Proposed design drawings	Proposed Crossing	3140	
	3	XS - copy	3130	8
	3	XS - copy	3122	0
Ordinary Watercourse - Lower	2	XS - copy	3122	23
	2	XS	3099	55
	1	XS	3044	124
	29	XS - copy	2920	20
	Area 4 - topo Survey	Culvert (Modified for Proposed Crossing)	2915	
	29	XS	2900	130
	30	XS	2770	153
	31	XS	2617	77

River Reach	Surveyed Section	Section Type	Chainage	Reach length (to next cross-section)
	31	XS – copy – shifted down 0.5m	2540	20
	Proposed design drawings	Proposed Crossing	2530	
	32	XS – copy – shifted up 0.38m	2520	58
	32	XS	2462	118
	33	XS	2344	124
	34	XS	2220	96
	35	XS	2124	69
	N/A	LiDAR	2055	10
	No access – estimated data	Bridge	2050	
	N/A	LiDAR	2045	125
	N/A	LiDAR	1920	100
	N/A	LiDAR	1820	100
	N/A	LiDAR	1720	100
	N/A	LiDAR	1620	100
	N/A	LiDAR	1520	100
	N/A	LiDAR	1420	100
	N/A	LiDAR	1320	100
	N/A	LiDAR	1220	100
	N/A	LiDAR	1120	80
	N/A	LiDAR	1040	40
	No access – estimated data	Bridge	1035	
	N/A	LiDAR	1000	100
	N/A	LiDAR	900	100
	N/A	LiDAR	800	100
	N/A	LiDAR	700	100
	N/A	LiDAR	600	100
	N/A	LiDAR	500	100
	N/A	LiDAR	400	100
	N/A	LiDAR	300	100
	N/A	LiDAR	200	100
	N/A	LiDAR	100	100

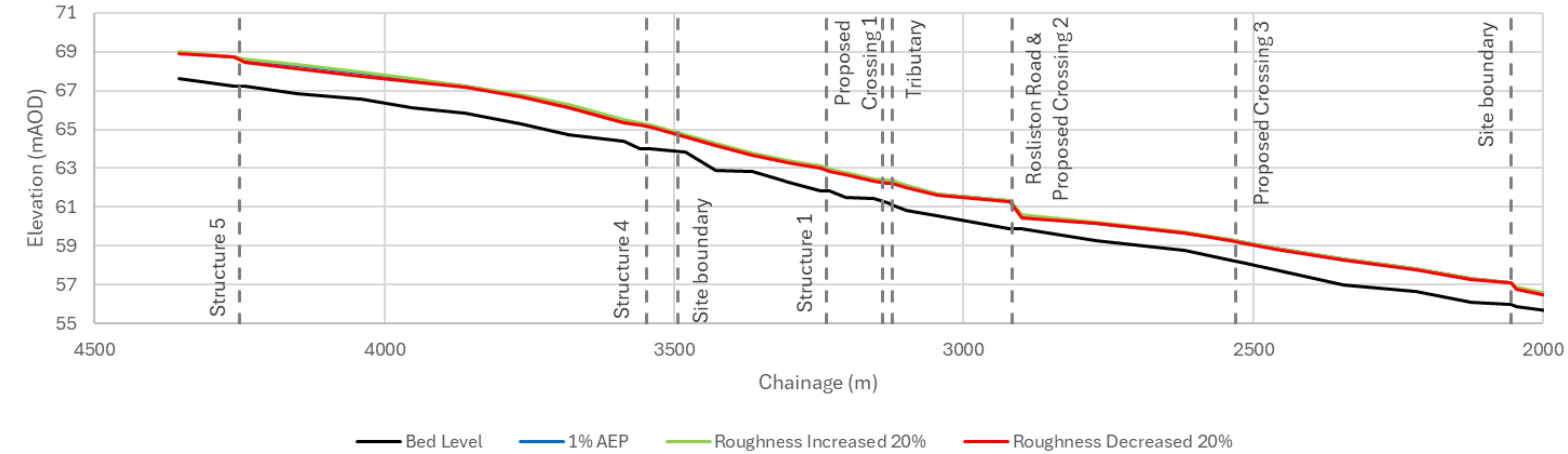
River Reach	Surveyed Section	Section Type	Chainage	Reach length (to next cross-section)
	N/A	LiDAR	0	0
Tributary	28	XS	1049	69
	27	XS	980	60
	26	XS	920	48
	25	XS	872	65
	24	XS	807	51
	23	XS	756	50
	22	XS	706	75
	21	XS	631	51
	20	XS - copy	580	9
	Structure 3	Culvert	575	
	20	XS	571	60
	19	XS	511	46
	18	XS	465	75
	17	XS	390	48
	16	XS	342	54
	15	XS	288	61
	14	XS	227	32
	13	XS - copy	195	9
	Structure 2	Structure	190	
	13	XS	186	34
	12	XS	152	66
	11	XS	86	60
	10	XS	26	26
	10	XS - copy	0	0

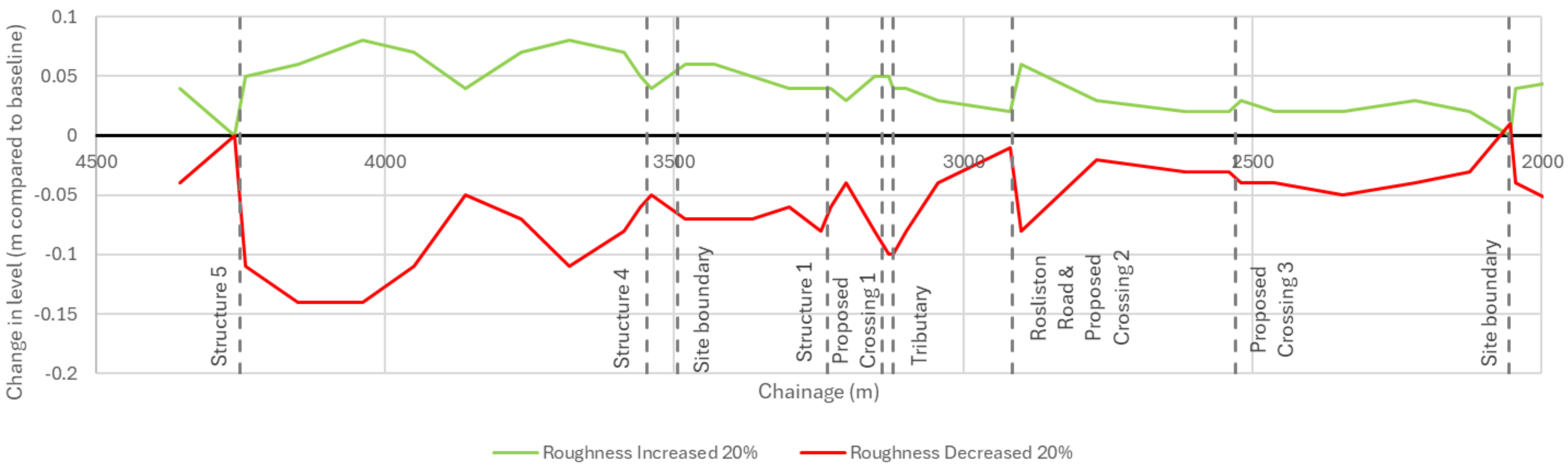
Appendix E Sensitivity Outputs

Mannings Roughness – Comparison against baseline (Tributary)

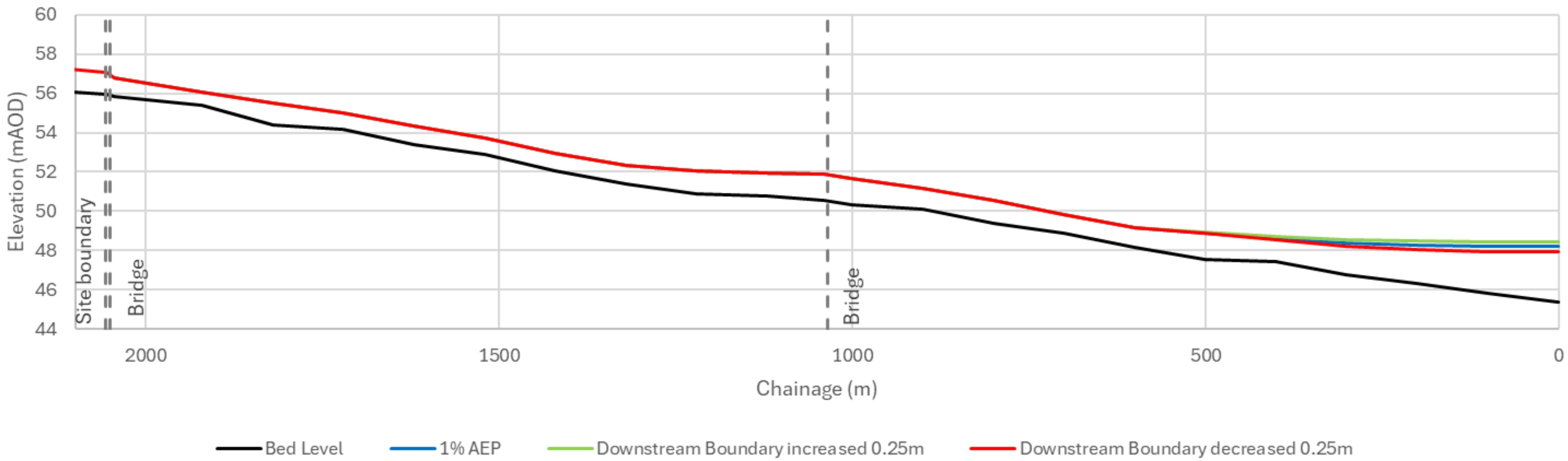


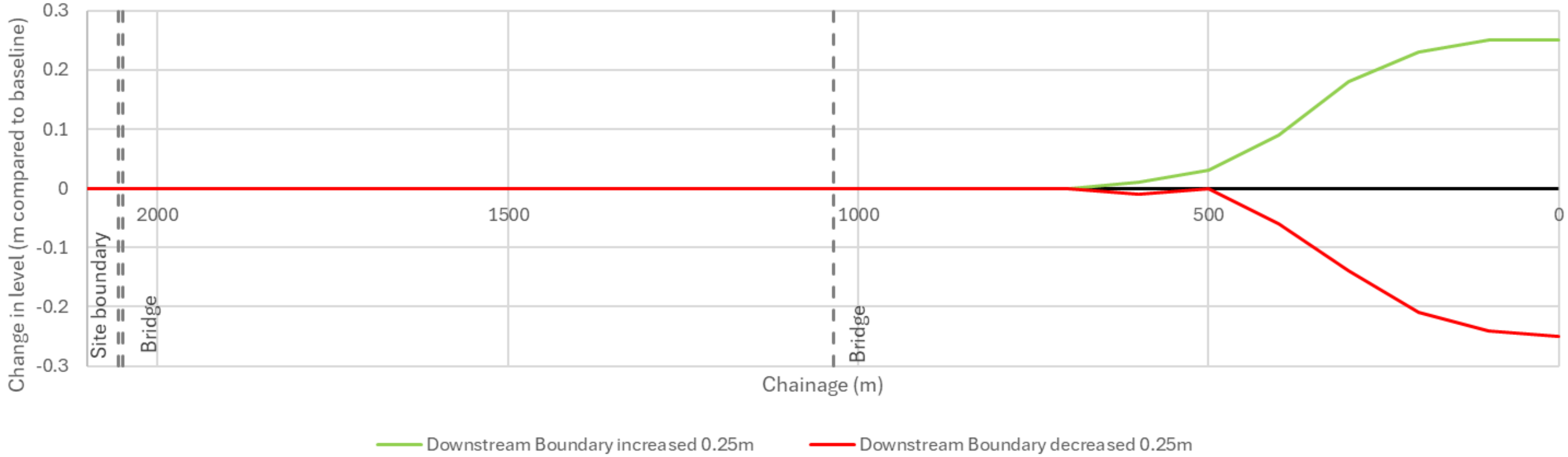
Mannings Roughness – Comparison against baseline (Ordinary Watercourse)



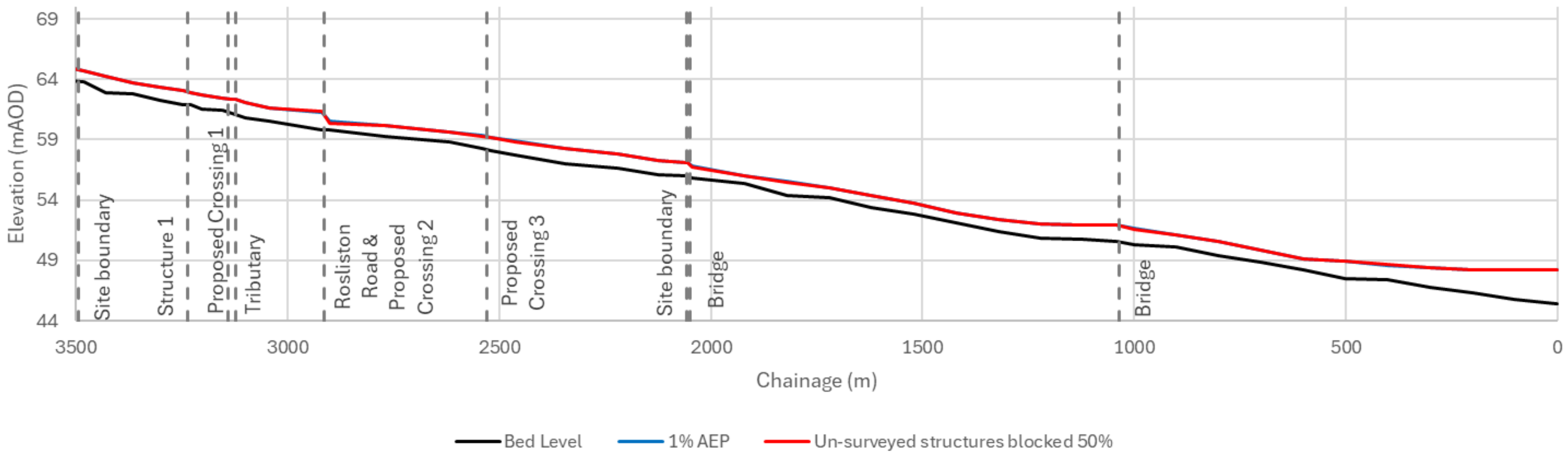


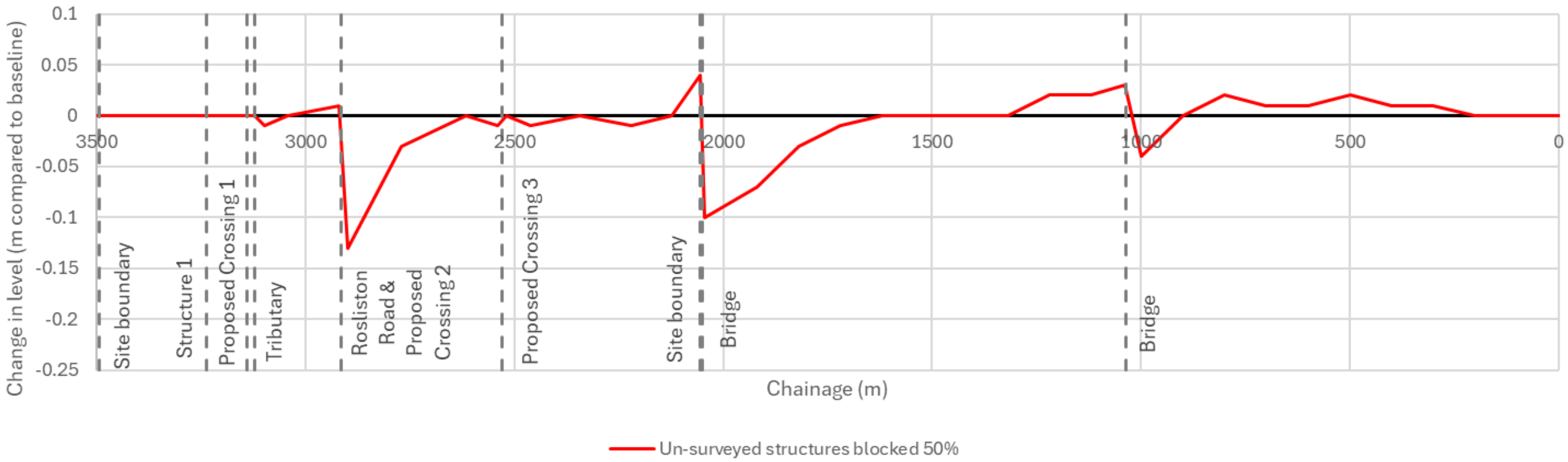
Downstream Boundary – Comparison against baseline (Ordinary Watercourse up to where no impact seen)



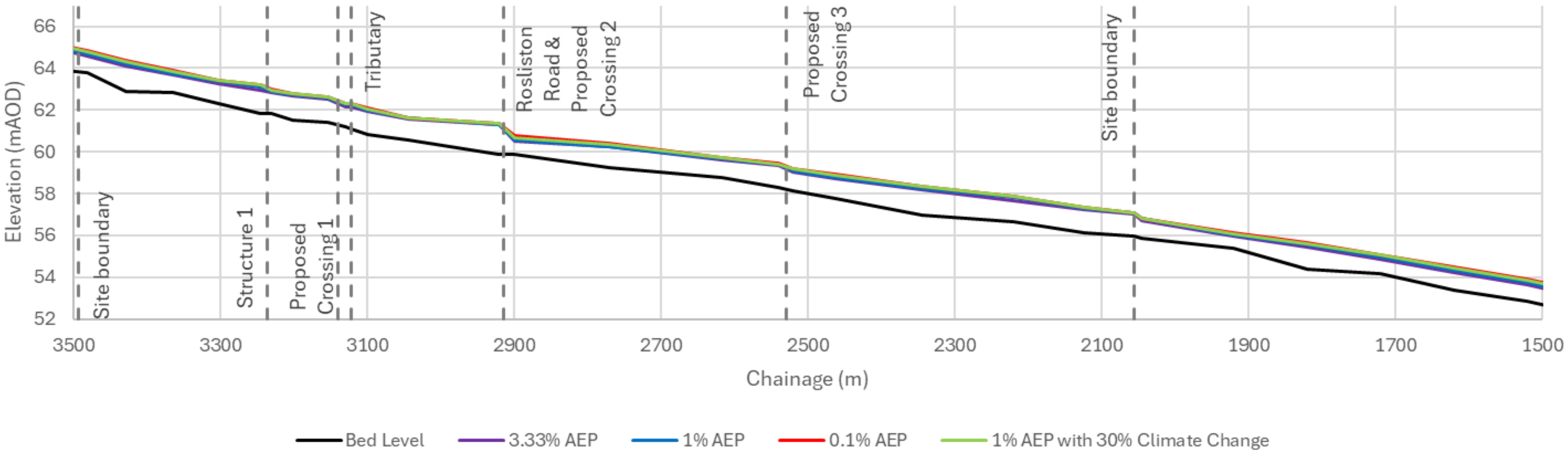


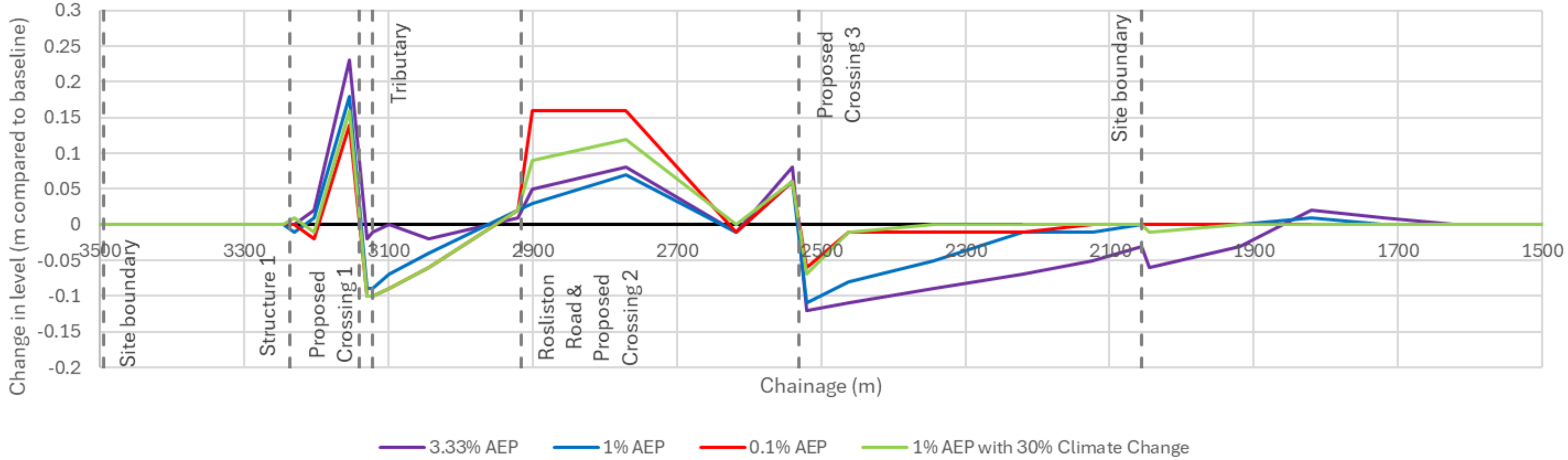
Structure Blockage – Comparison against baseline (Ordinary Watercourse up to where no impact seen)



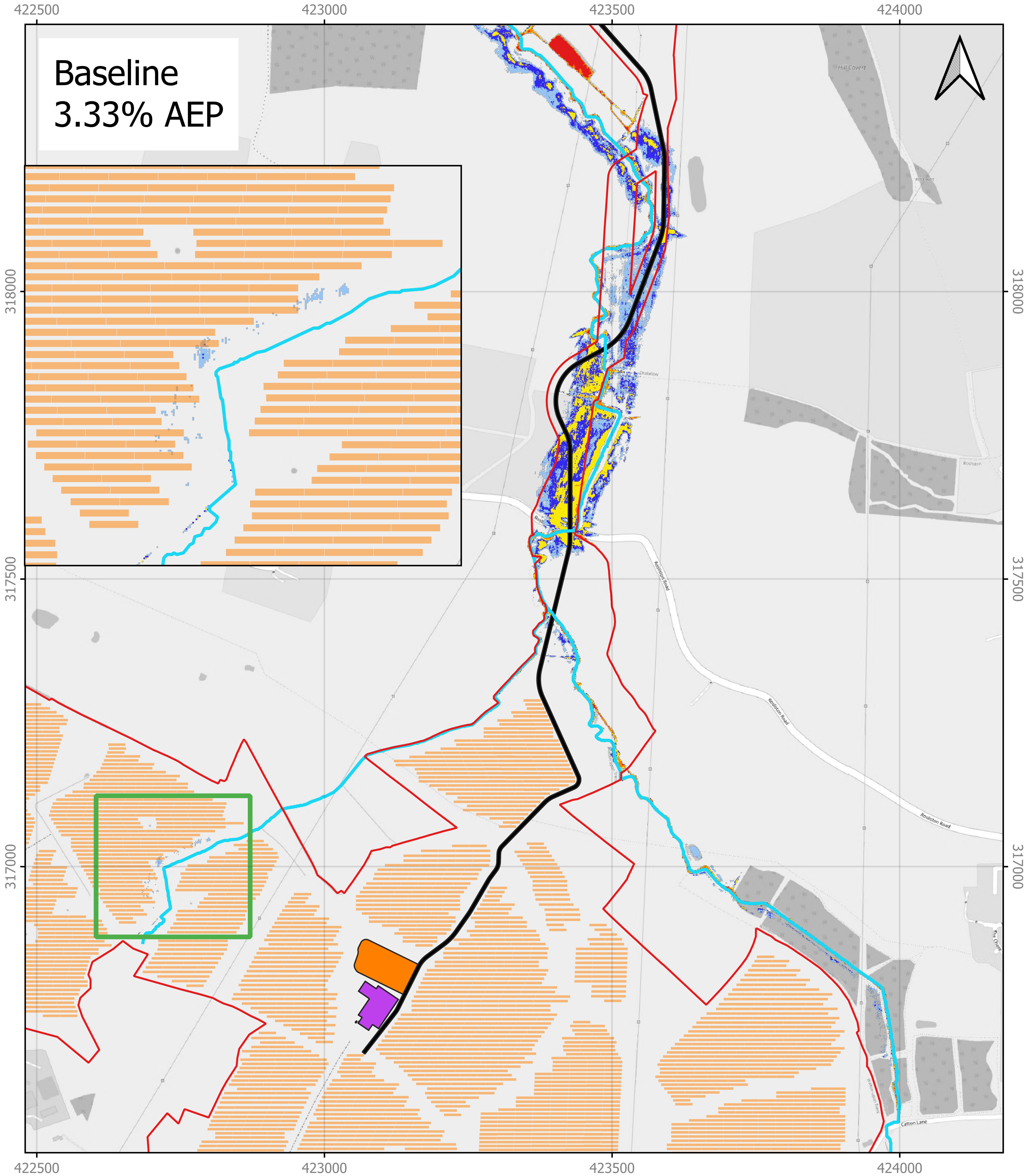


Proposed Development – Comparison against baseline of proposed crossings (Ordinary Watercourse up to where no impact seen)





Appendix F Flood maps



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Baseline 3.33% AEP
Drawn by	MJF
Checked by	JEM
Date	07/08/2024

0 200 400 m

Legend

Site Boundary

Proposed Development

- Access Track
- Substation
- Battery Storage
- Solar Panel

3.33% AEP Depth

- <= 0.05m
- 0.05 - 0.10m
- 0.10 - 0.25m
- 0.25 - 0.50m
- > 0.50m

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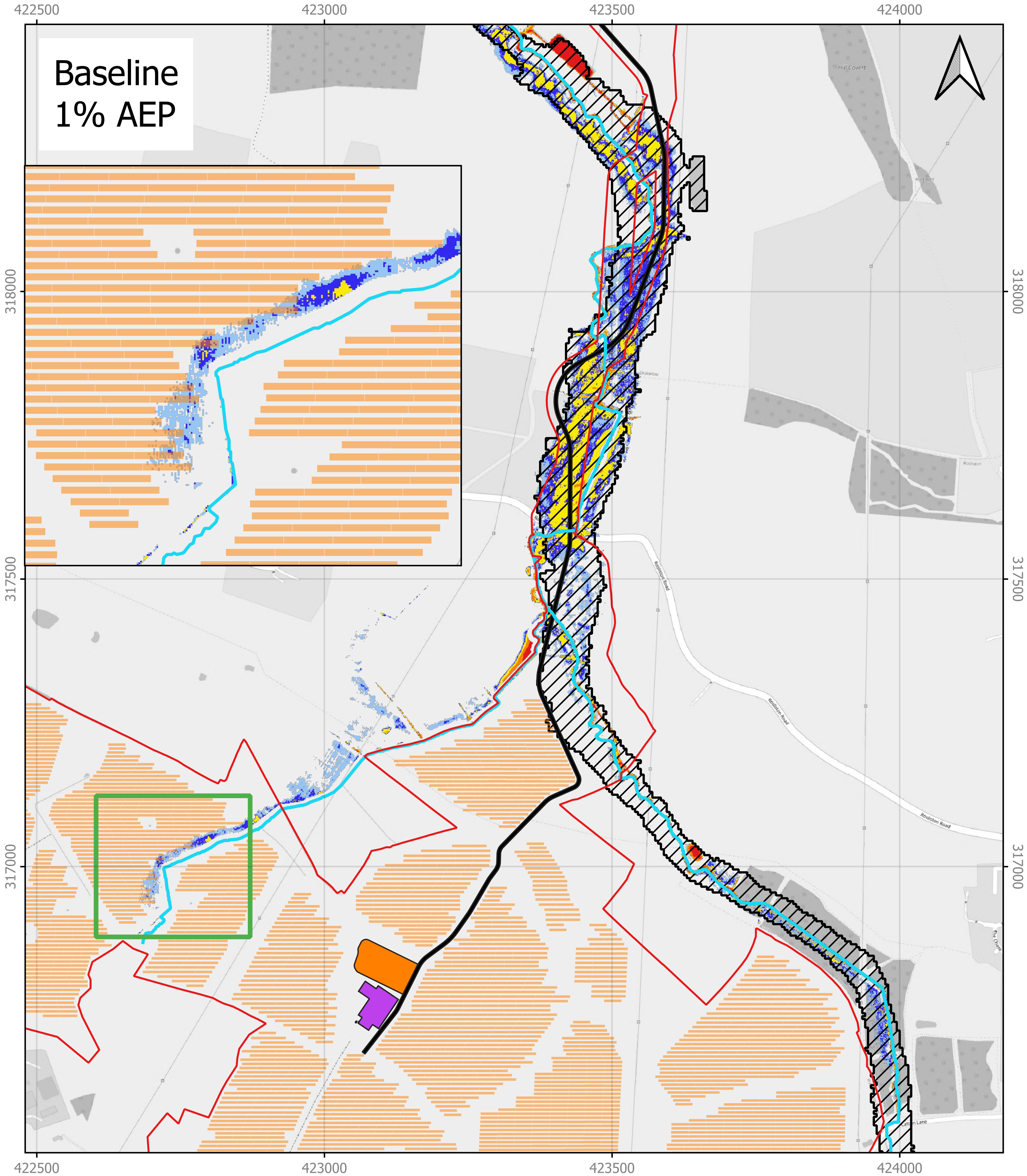


Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Baseline 1% AEP
Drawn by	MJF
Checked by	JEM
Date	07/08/2024

0 200 400 m

Legend

- Site Boundary
- Flood Zone 2

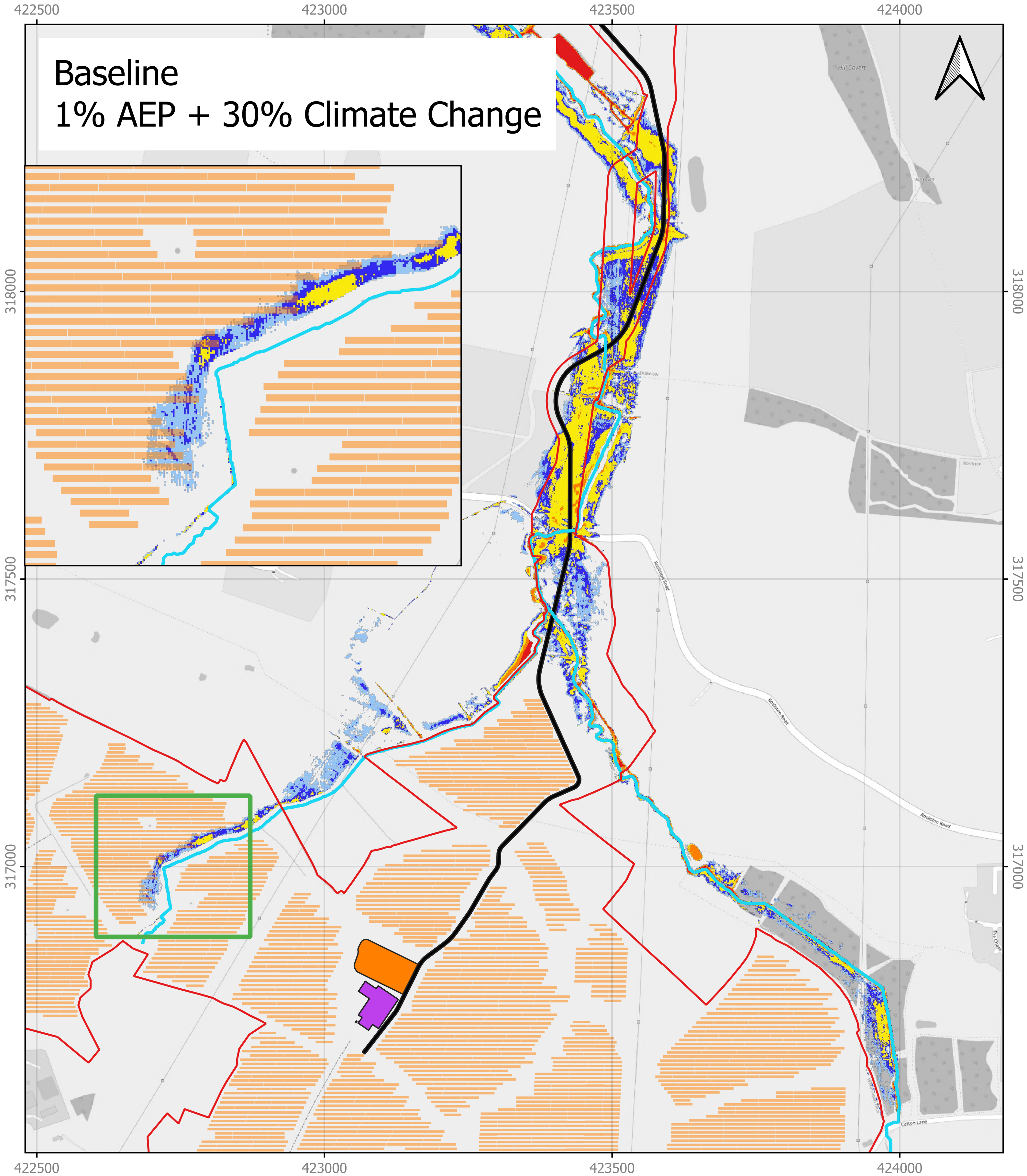
Proposed Development

- Access Track
- Substation
- Battery Storage
- Solar Panel

1% AEP Depth

- <= 0.05m
- 0.05 - 0.10m
- 0.10 - 0.25m
- 0.25 - 0.50m
- > 0.50m

AQUA TERRA
CONSULTING



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Baseline 1% AEP + 30% CC
Drawn by	MJF
Checked by	JEM
Date	07/08/2024

0 200 400 m

Legend

- Site Boundary

Proposed Development

- Access Track
- Substation
- Battery Storage
- Solar Panel

1% AEP + 30% CC Depth

- <= 0.05m
- 0.05 - 0.10m
- 0.10 - 0.25m
- 0.25 - 0.50m
- > 0.50m

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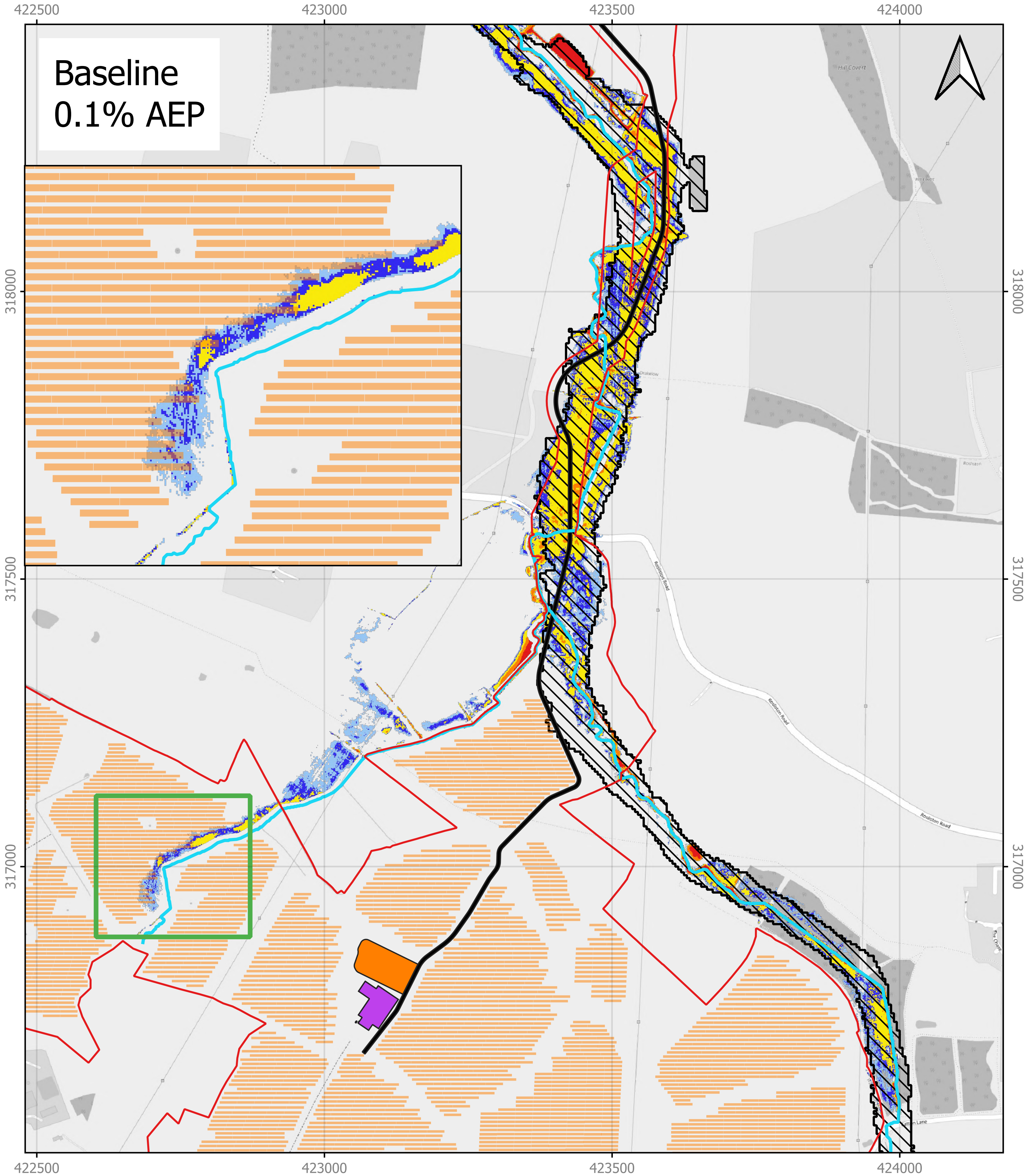

Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Baseline 0.1% AEP
Drawn by	MJF
Checked by	JEM
Date	07/08/2024

0 200 400 m

Legend

- Site Boundary
- Flood Zone 3

Proposed Development
- Access Track
- Substation
- Battery Storage
- Solar Panel

0.1% AEP Depth
- <= 0.05m
- 0.05 - 0.10m
- 0.10 - 0.25m
- 0.25 - 0.50m
- > 0.50m

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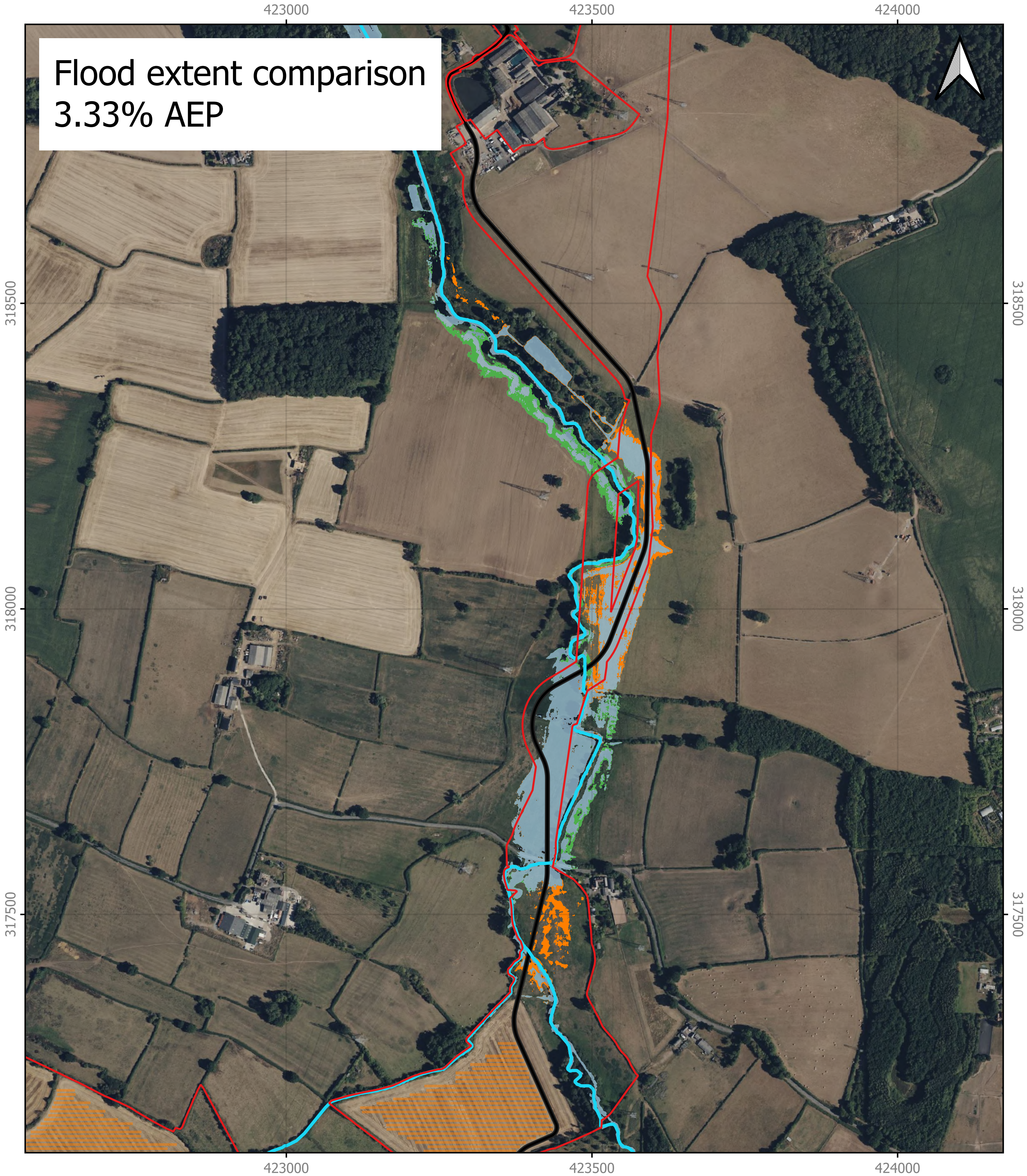
Flood extent comparison 3.33% AEP



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Flood Extents 3.33% AEP
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

0 200 400 m

Legend

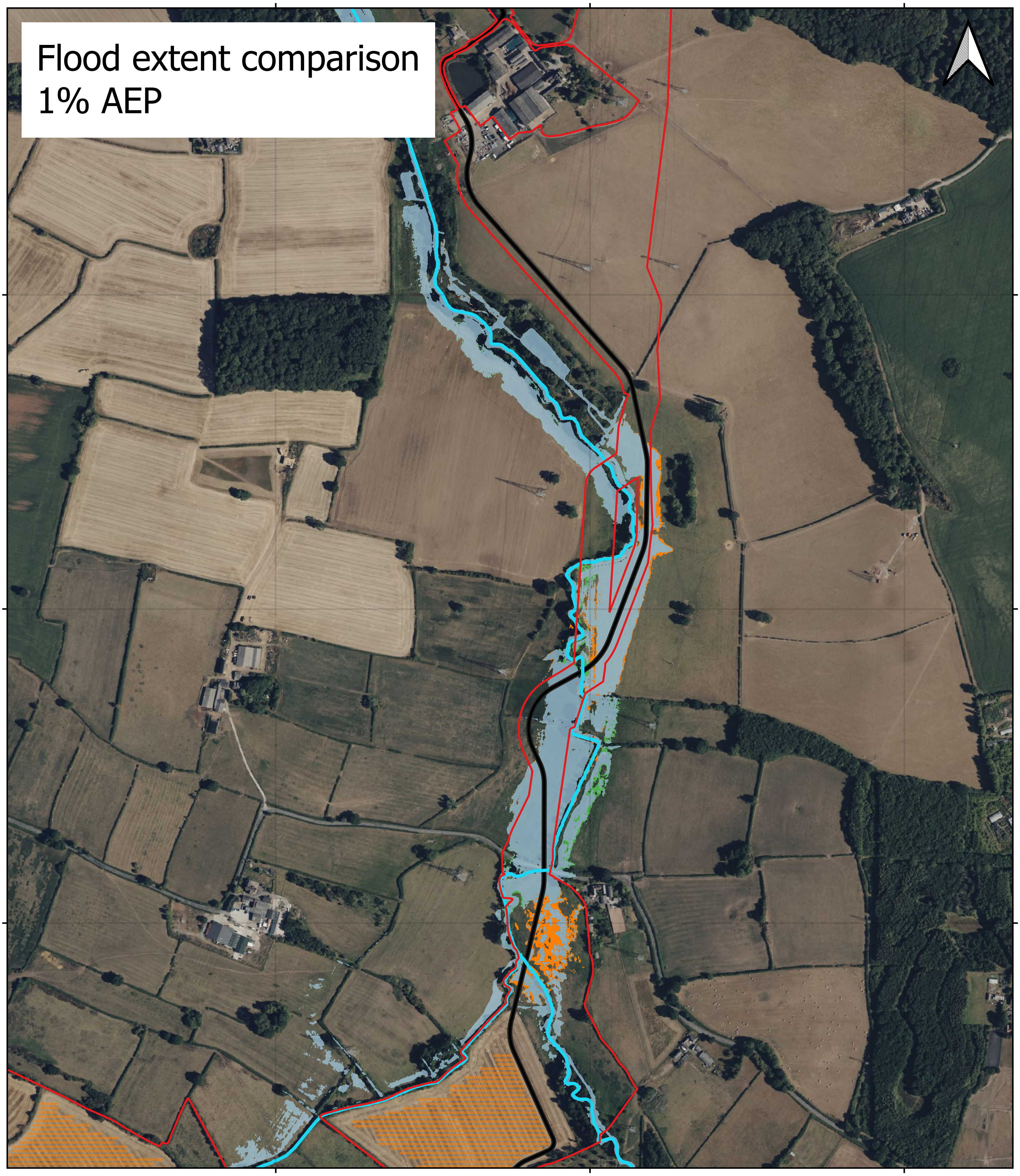
Site Boundary

Proposed Development
- Access Track
- Substation
- Battery Storage
- Solar Panel

Modelled Extents 3.33% AEP
- No change to flood extent
- Added to flood extent
- Removed from Flood extent

AQUA TERRA CONSULTING

Flood extent comparison 1% AEP



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Flood Extents 1% AEP
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

0 200 400 m

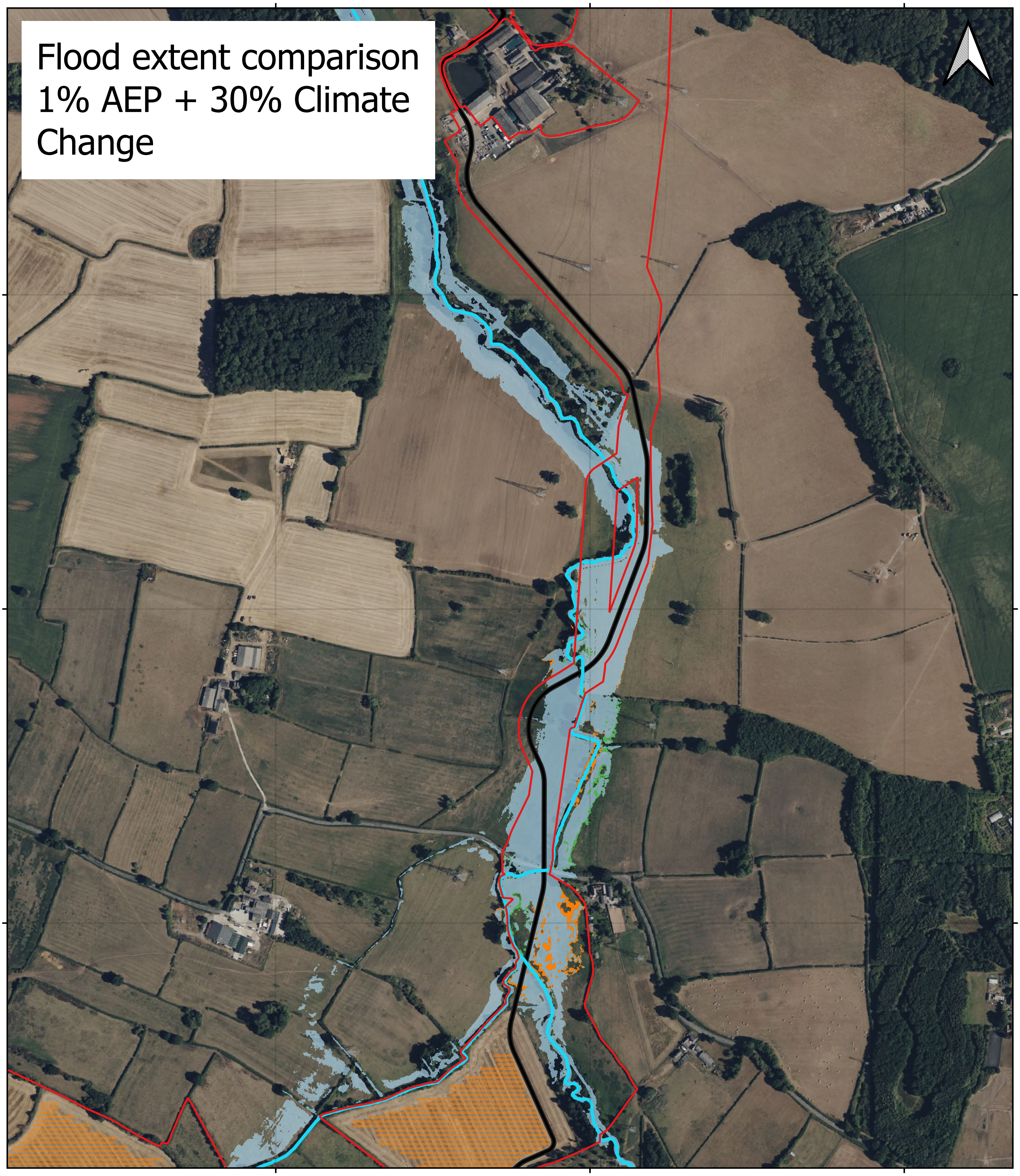
Legend

Site Boundary

Proposed Development
- Access Track
- Substation
- Battery Storage
- Solar Panel

Modelled Extents 1% AEP
- No change to flood extent
- Added to flood extent
- Removed from Flood extent

AQUA TERRA CONSULTING



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Flood Extents 1% AEP+30% CC
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

0 200 400 m

Legend

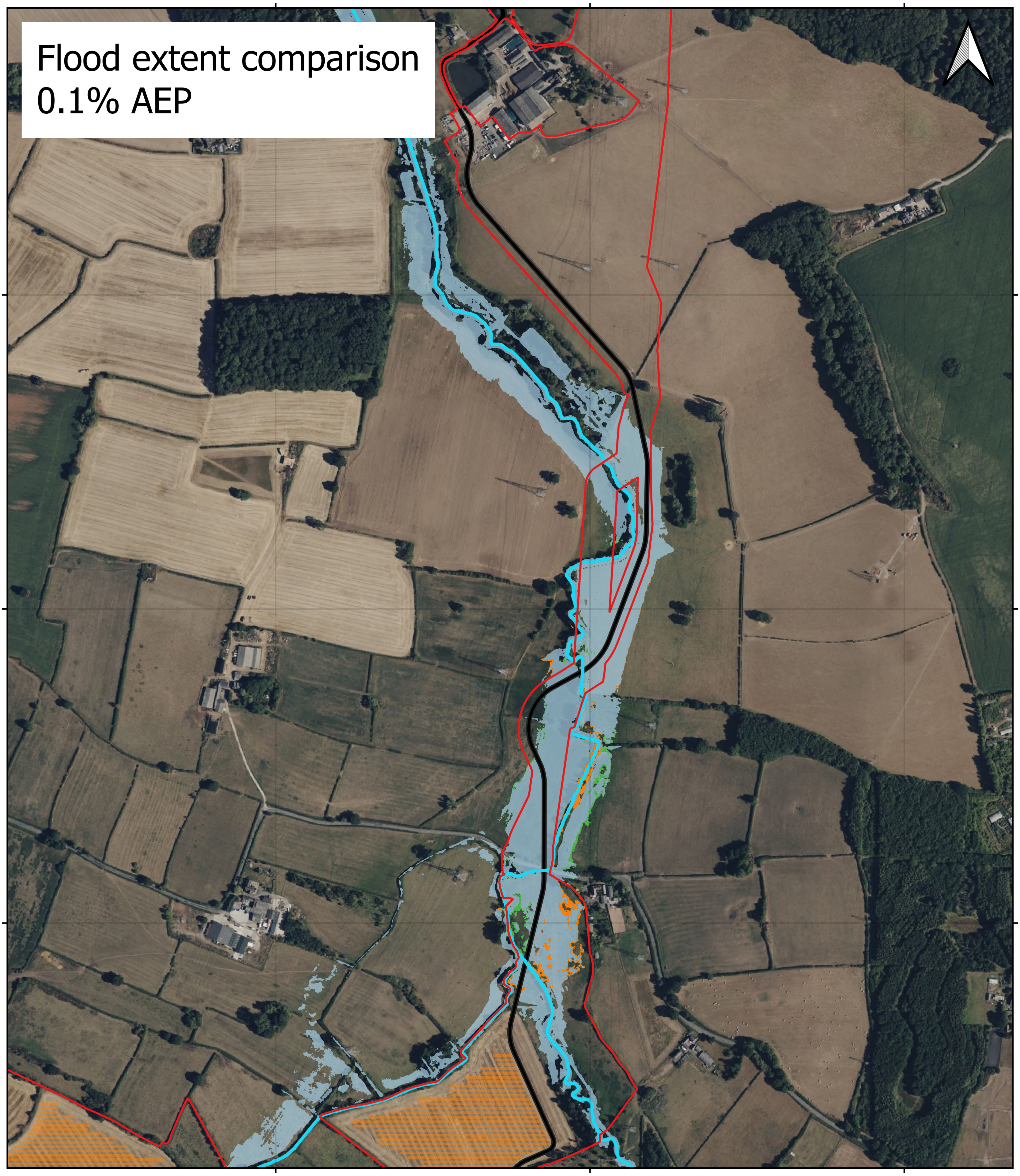
Site Boundary

Proposed Development
- Access Track
- Substation
- Battery Storage
- Solar Panel

Modelled Extents
1% AEP + 30% CC
- No change to flood extent
- Added to flood extent
- Removed from Flood extent

AQUA TERRA CONSULTING

Flood extent comparison 0.1% AEP



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Flood Extents 0.1% AEP
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

0 200 400 m

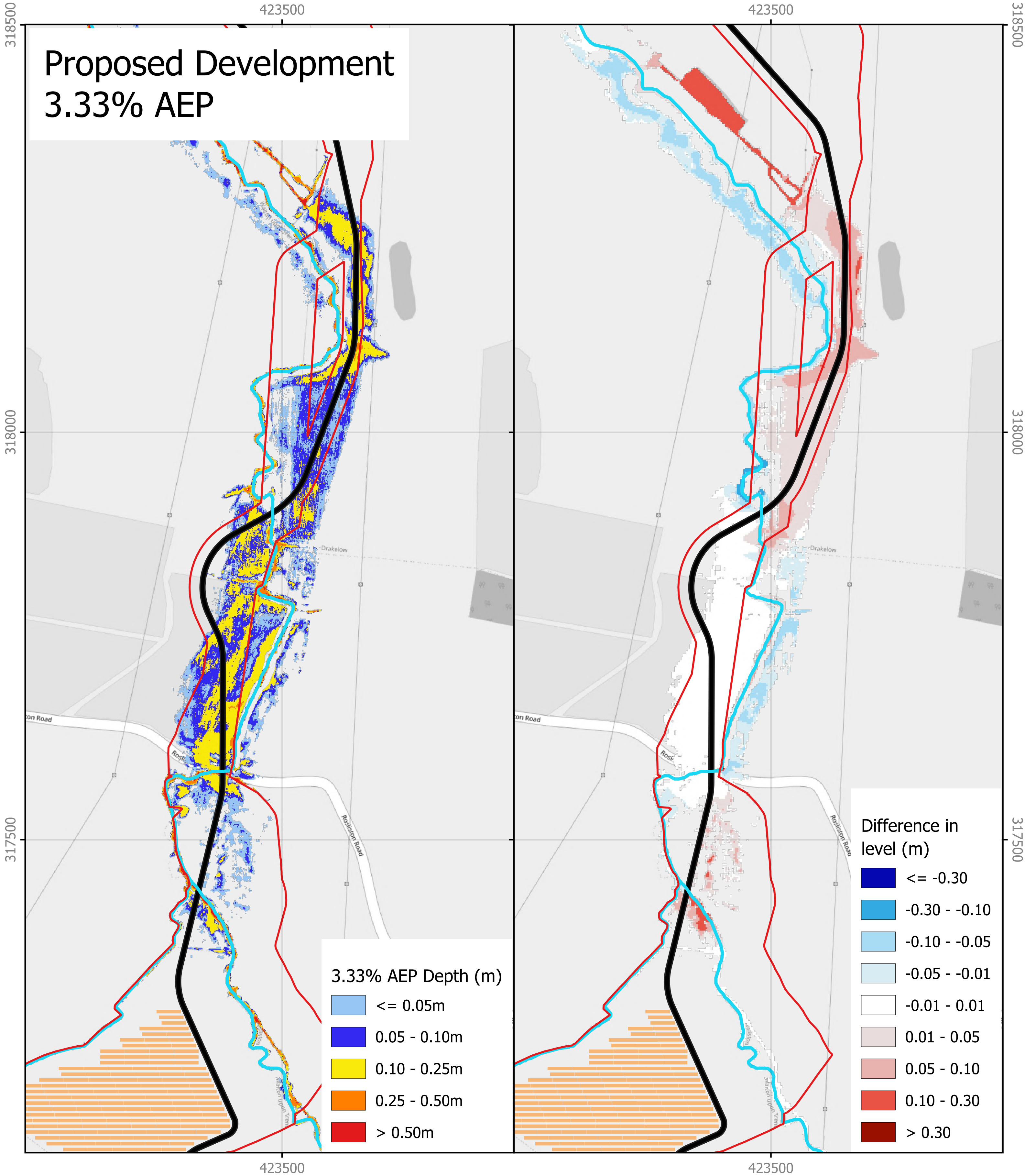
Legend

Site Boundary

Proposed Development
- Access Track
- Substation
- Battery Storage
- Solar Panel

Modelled Extents 0.1% AEP
- No change to flood extent
- Added to flood extent
- Removed from Flood extent

AQUA TERRA CONSULTING



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Proposed 3.33% AEP
Drawn by	MJF
Checked by	JEM
Date	24/10/2024

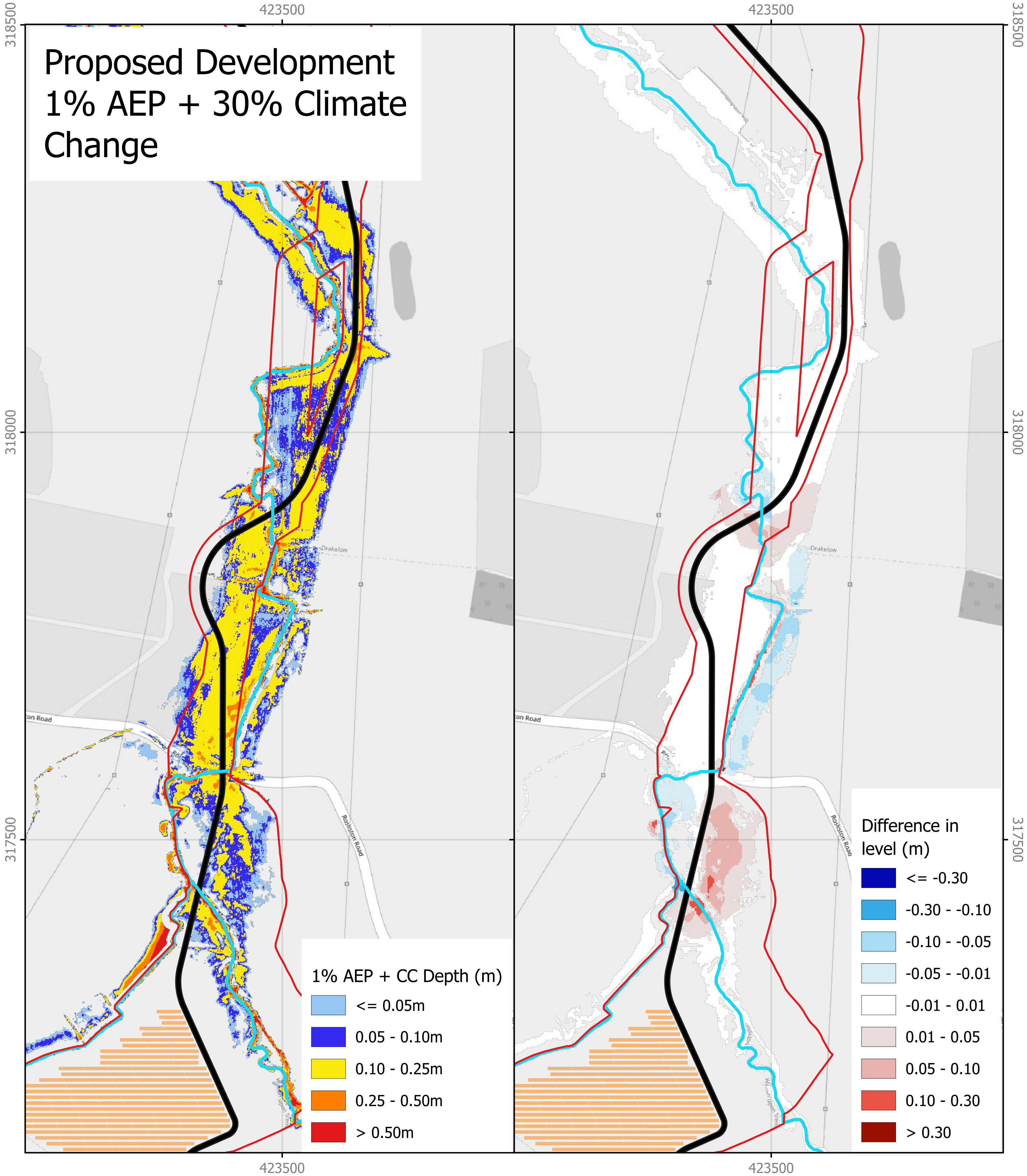
0 200 400 m

Legend

Site Boundary
Ordinary Watercourse

Proposed Development
Access Track
Substation
Battery Storage
Solar Panel

AQUA TERRA
CONSULTING



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Proposed 1% AEP + 30% CC
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

0 200 400 m

Legend

- Site Boundary
- Ordinary Watercourse

Proposed Development

- Access Track
- Substation
- Battery Storage
- Solar Panel

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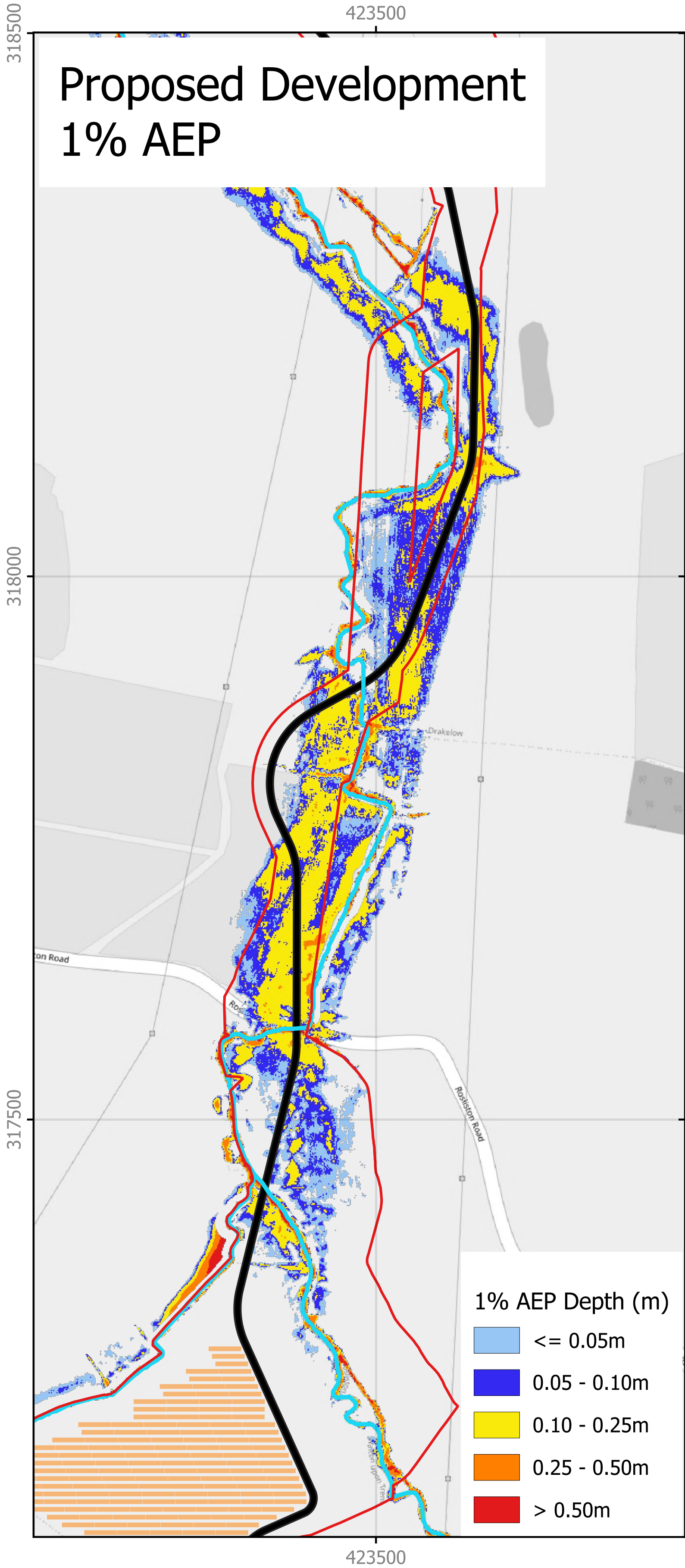
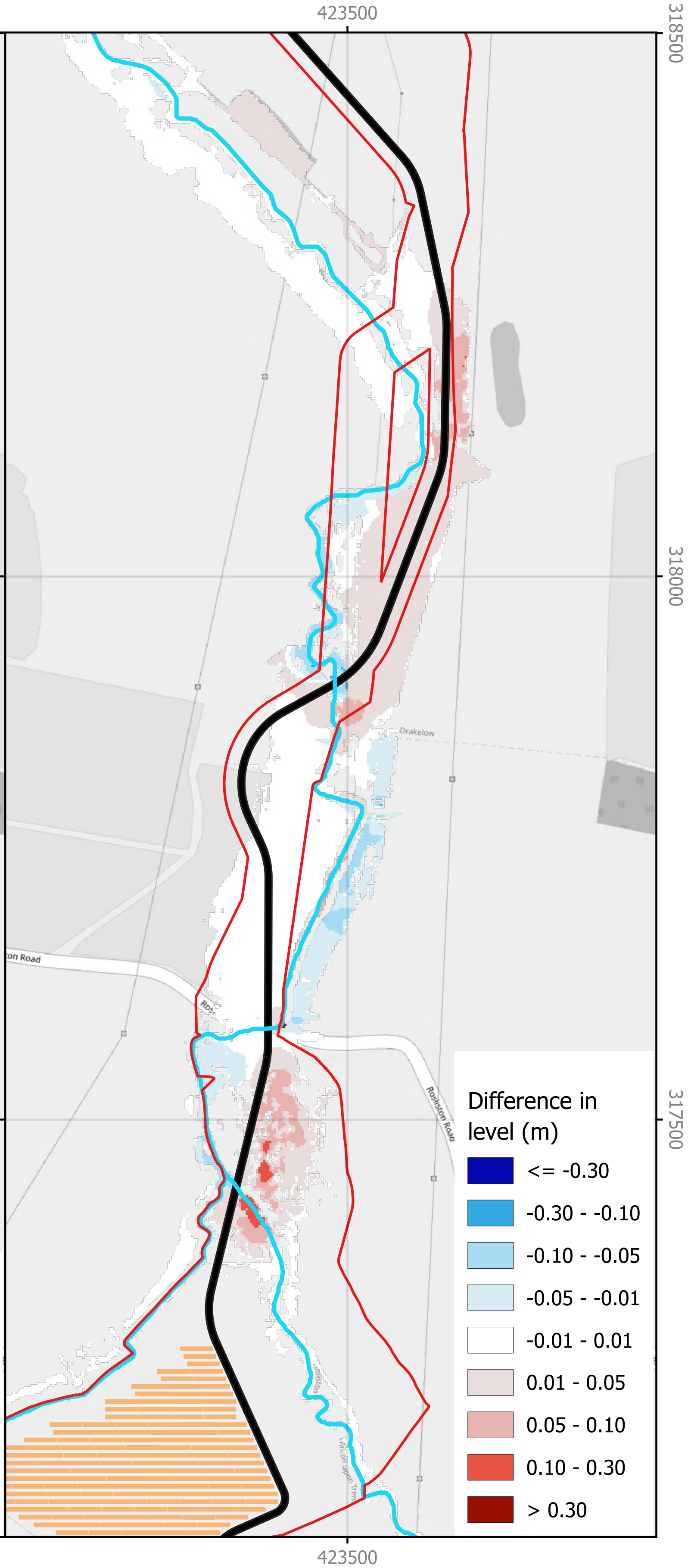
CONSULTING





Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Proposed 1% AEP
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

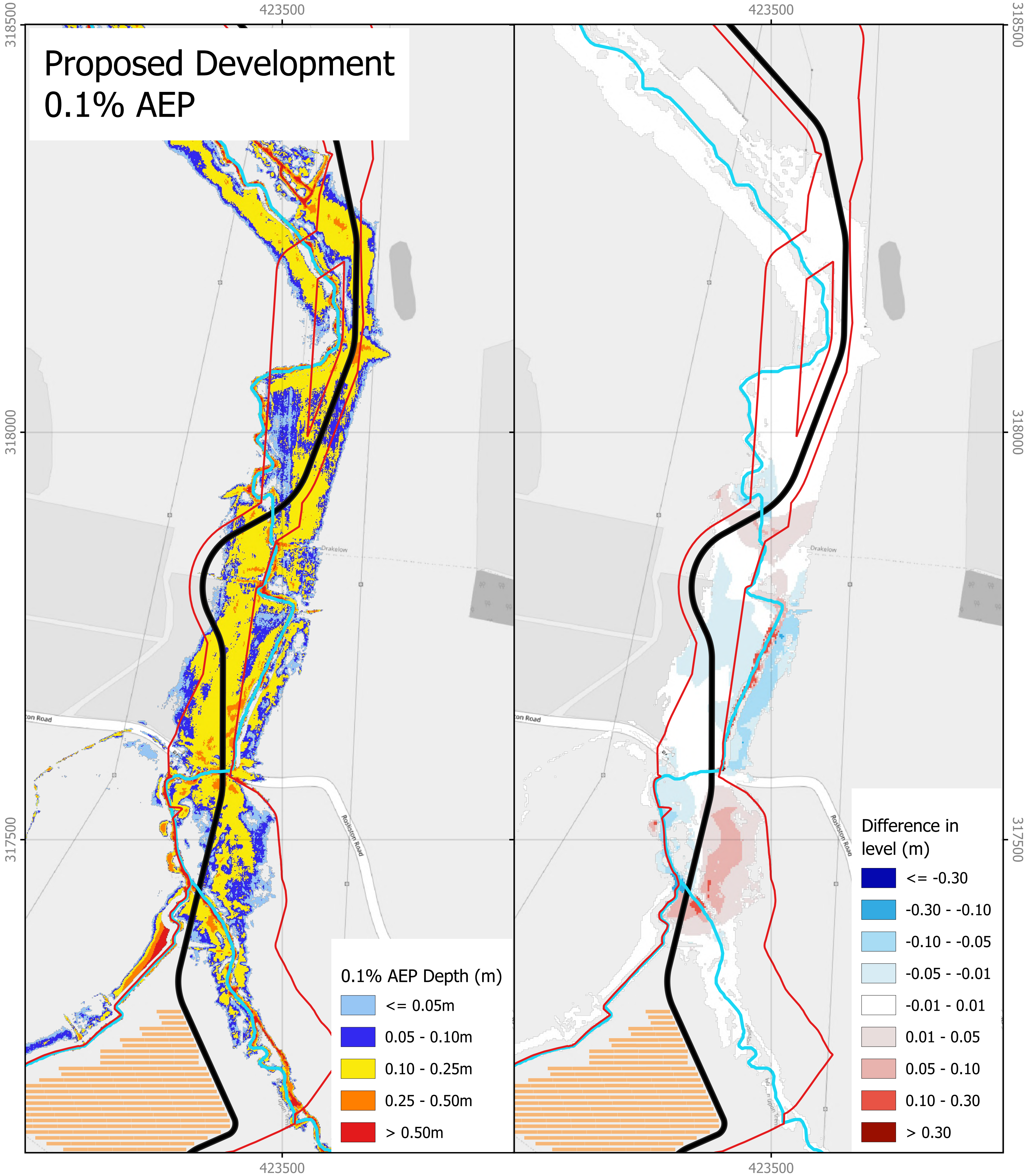
0 200 400 m

Legend

- Site Boundary
- Ordinary Watercourse

Proposed Development
- Access Track
- Substation
- Battery Storage
- Solar Panel

AQUA TERRA CONSULTING



Project	Oaklands Farm Solar Park
Project Number	P24022
Client	BayWa R.E
Title	Proposed 0.1% AEP
Drawn by	MJF
Checked by	JEM
Date	21/10/2024

0 200 400 m

Legend

- Site Boundary
- Ordinary Watercourse

Proposed Development

- Access Track
- Substation
- Battery Storage
- Solar Panel

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