

Flood Risk Assessment and Drainage Strategy

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Land at The Green, Great Cheverell, Wiltshire

Flood Risk Assessment and Drainage Strategy

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1 INTRODUCTION

Tumu Consulting Ltd have been appointed by Joel Figov and Annabell Mears to undertake a Flood Risk Assessment (FRA) and Drainage Strategy (DS) to accompany a Planning Application for the erection of a self-build dwelling with separate garage and associated works on Land at The Green, Great Cheverell, Wiltshire.

This site was subject to a previous Planning in Principle application (ref PL/2024/04957). This report addresses the various concerns raised by Wiltshire Council as LLFA (via AtkinsRealis), including the Team's meeting held on 7th February 2025 (attended by Barney Taylor).

1.1 Site Location

The site is an existing triangular shaped paddock, located on the northern outskirts of Great Cheverell, on the eastern side of The Green. It is bounded to the north by a double track railway. The nearest postcode to the site is SN10 5XN. It is proposed to locate the new dwelling in the south-west corner, adjacent the existing access gate.

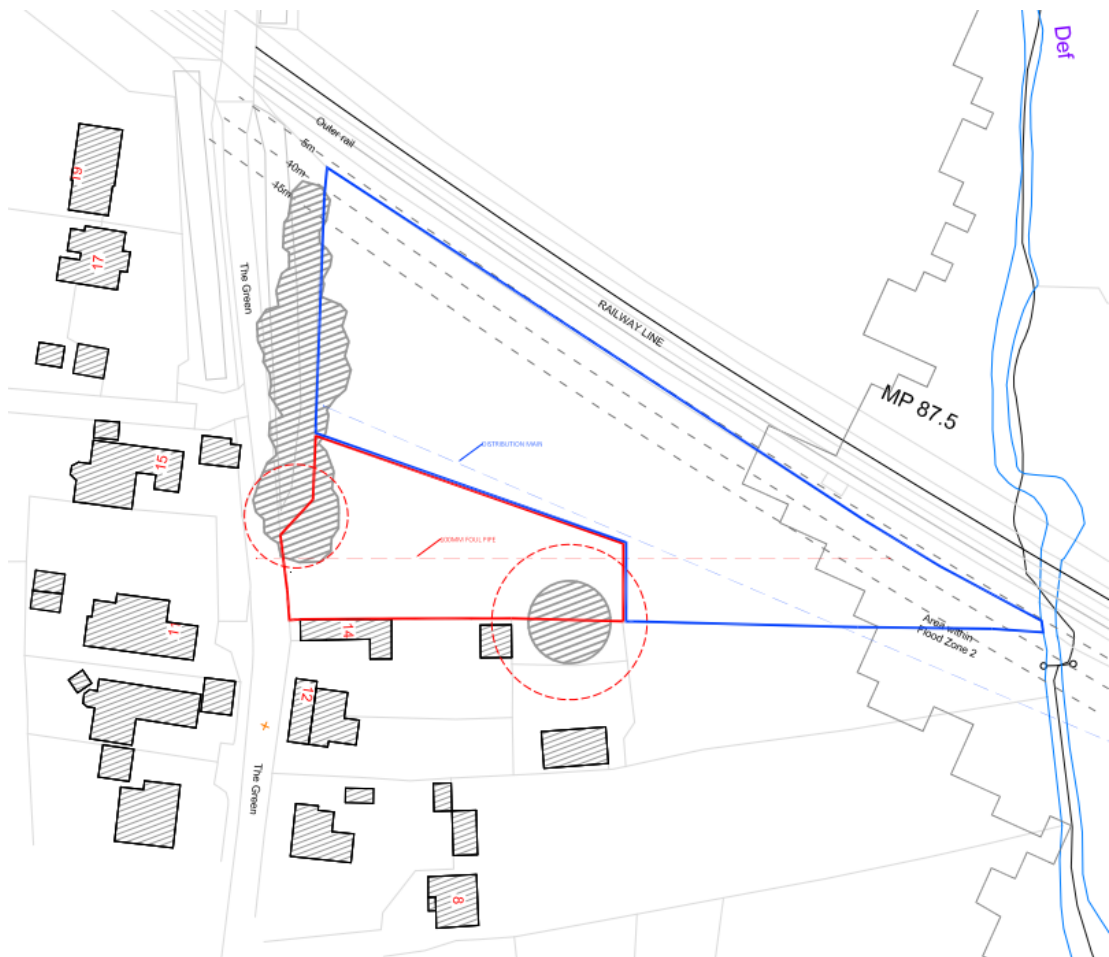


Figure 1-1 Site Location Plan

1.2 Existing Site Usage

The existing site is currently greenfield and used as a paddock. It is accessed via a gate off The Green to the west.

1.3 Topography

LIDAR data (Composite DTM, 2022, 1m) has been downloaded from the Environment Agency's National LIDAR Programme website, and processed using the QGIS software package to generate 0.25m interval contours for the site and its immediate surrounding area. The generated contours have been overlain on OpenStreetMap, as Figure 1-3a below.



Figure 1-3a Contour Mapping

A detailed topographical survey of the site was undertaken in January 2025, as Figure 1-3b below. The later confirms that the former was an accurate representation of the site.

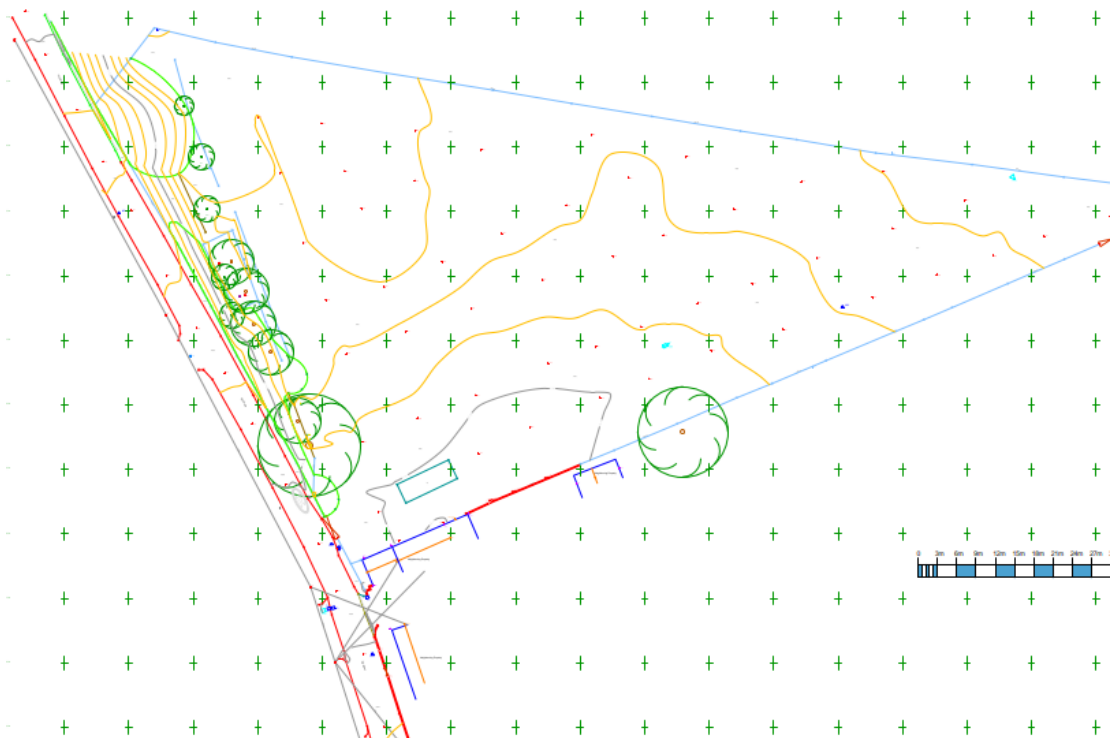


Figure 1-3b Topographical Survey Extract

Geology & Hydrogeology

According to the British Geological Survey (BGS) Viewer, the site is underlain with a bedrock geology of Gault Formation Mudstone overlain by superficial head deposits (clay, silt, sand and gravel). Whilst this geology continues away to the west, north and east, less than 100m to the south the higher land has Shaftesbury Sandstone bedrock geology with no superficial deposits.

Based on the geological conditions it is considered that the underlying natural soils are likely to be of low to negligible permeability.

The nearest watercourse is an unnamed stream immediately adjacent the site's eastern corner. This flows northwards, under the railway line, towards Semington Brook.

Wessex Water's Asset Mapping (see Figure 1-6 below) shows the site is crossed by both a 100mm diameter foul sewer (flowing west to east) and a 500mm diameter water main. There is also a surface water sewer running south to north along the western edge of The Green. This sewer increases in diameter from 225mm to 300mm at a chamber located opposite the site's entrance. A water main is also present along the eastern side of The Green.

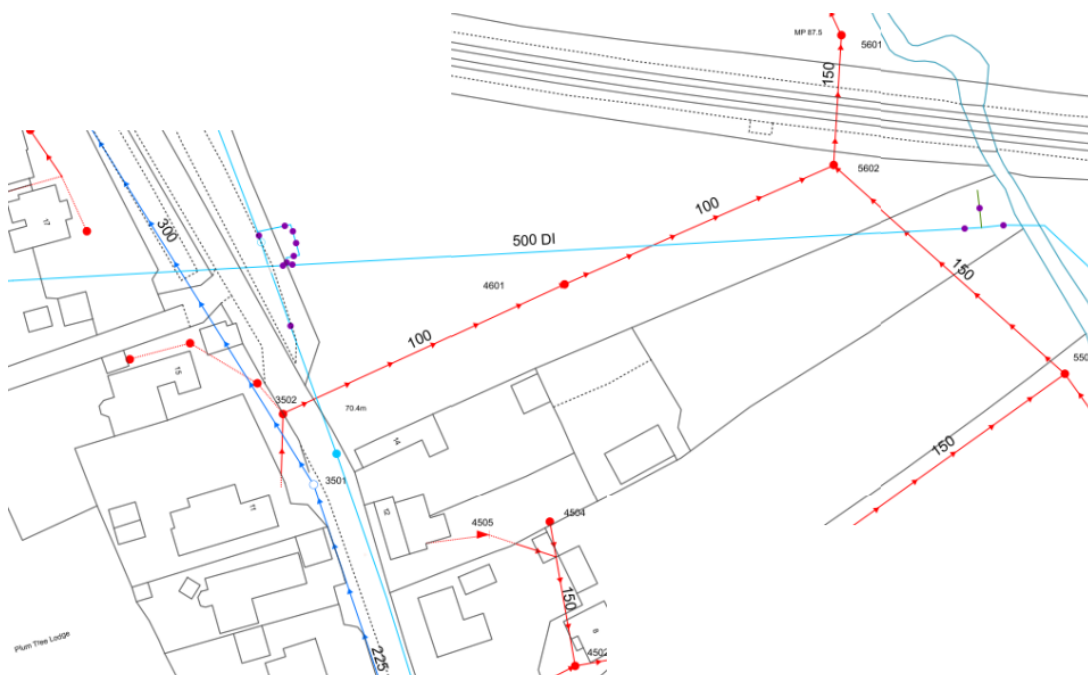


Figure 1-6a Wessex Water Asset Mapping

A sub-surface scan was undertaken in July 2025 by Solum Surveying Ltd (ref 14573) to establish the actual alignments of these Wessex Water assets.

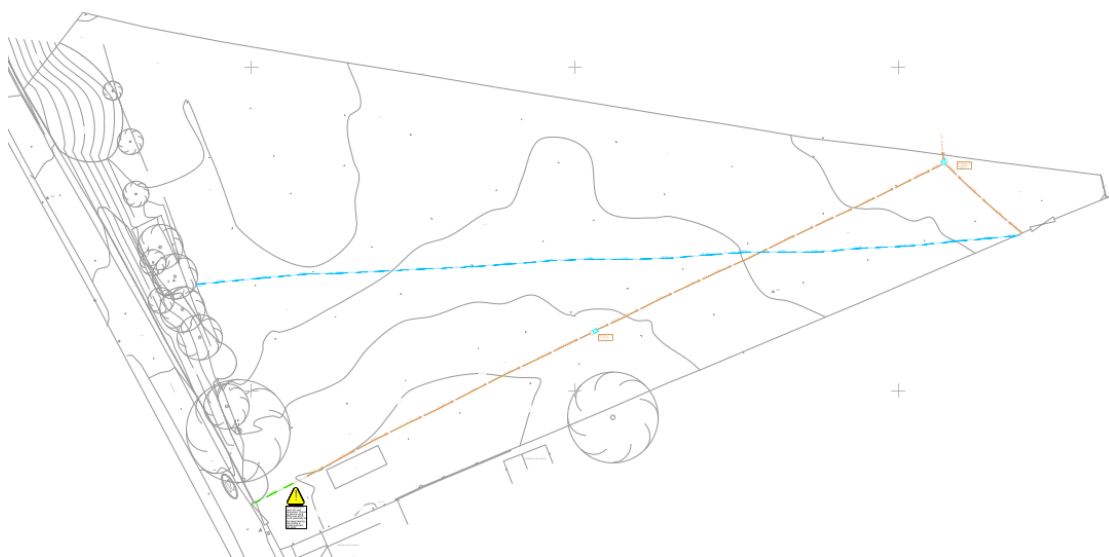


Figure 1-6b Sub-surface Scan of Water Main and Foul Sewer

2 DEVELOPMENT PROPOSALS

2.1 Description of the Proposed Works

It is proposed to construct a single dwelling in the southwest corner of the site which will utilise the existing paddock access. Frontage parking will be provided plus an access route to the remaining paddock area. See Figure 2-1 below.

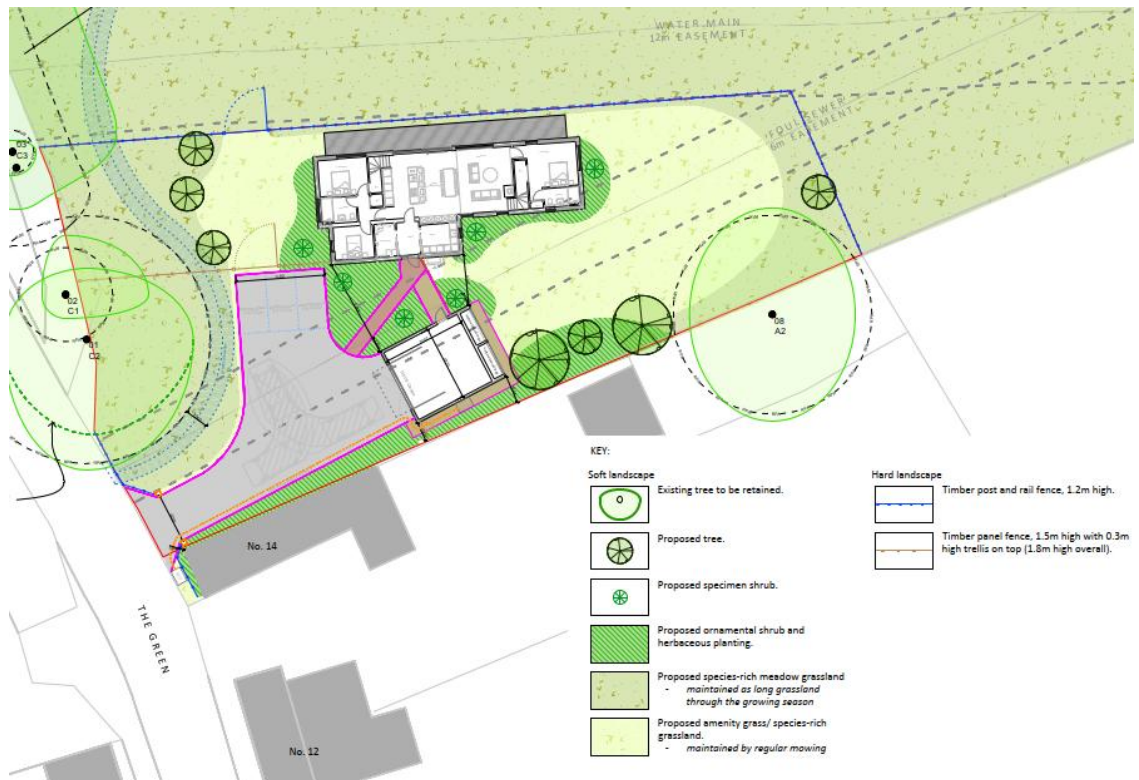


Figure 2-1 Development Proposals

2.2 Flood Risk Vulnerability Classification

In accordance with NPPF table D2 the development is classified 'more vulnerable':

More vulnerable

- Hospitals
- Residential institutions such as residential care homes, children's homes, social services homes, prisons and hostels.
- Buildings used for dwelling houses student halls of residence, drinking establishments, nightclubs and hotels.
- Non-residential uses for health services, nurseries and educational establishments.
- Landfill* and sites used for waste management facilities for hazardous waste.
- Sites used for holiday or short-let caravans and camping, subject to a specific warning and evacuation plan.

2.3 Design Life

As the nature of the development is a dwelling, the design life for flood risk will be 100 years.

3 ASSESSMENT OF FLOOD RISK

This section of the FRA assesses the flood risk from a variety of sources including the Government's 'Flood Map for Planning' and 'Long Term Flood Risk' maps.

3.1 Flood Map for Planning

The site is generally shown to be in Flood Zone 1 except for some incursions from Flood Zones 2 & 3. These however are into the area currently and proposed to remain as the paddock. Comparison with the contour mapping suggests the flood extent to the east correlates to around the 68.9m level whilst that to the north correlates to around 68.3m.

The area where the new buildings are proposed is fully in Flood Zone 1 and where the existing ground is a minimum of circa 69.50m. This is some 0.6m higher than the indicated Flood Zone 3 level.

The new buildings are thus at low risk of flooding from fluvial or tidal sources.

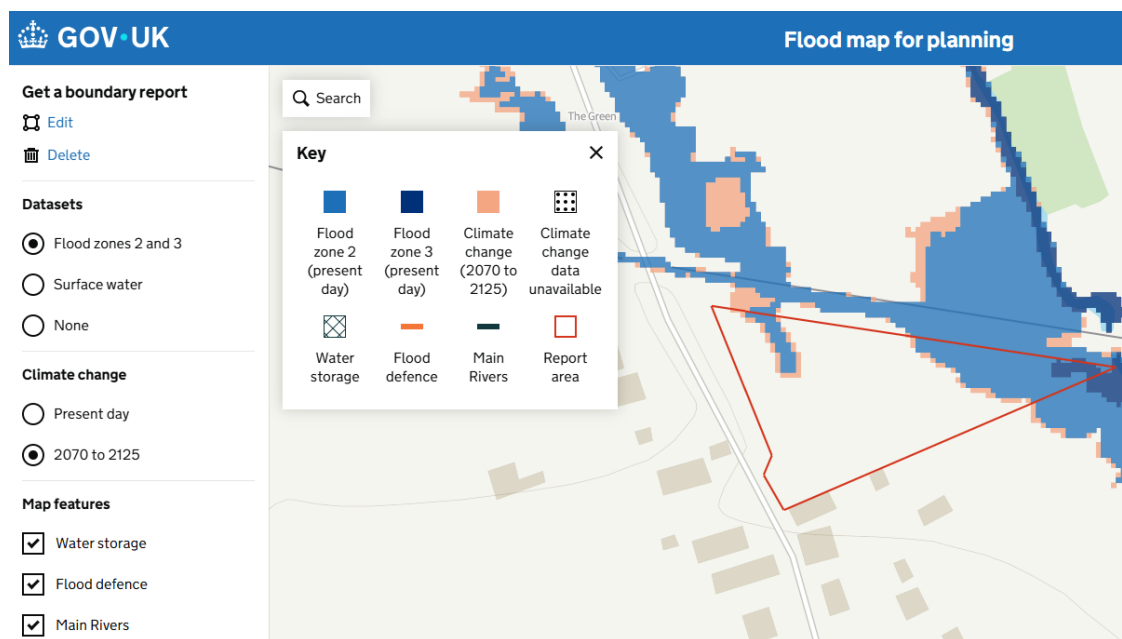


Figure 3-1 Flood Map for Planning

3.2 Surface Water Flood Risk

The UK Government's Long Term Flood Risk Mapping identifies the site to be impacted by areas at high risk (corresponding to a chance of flooding of >3.3% each year).

To the east, adjacent to the water course, is an area that correspond to the Flood Zone 3 extent. This has the character of being a bottleneck in the watercourse's capacity caused by its passage underneath the railway alignment, probably through an historic culvert.

To the west is a linear overland flow route. This follows the main road alignment emanating from Great Cheverell, on higher ground, to the southwest. At the gated entrance to the site it leaves

the road alignment and enters the paddock due to the road alignment rising up to cross over the railway line. Within the paddock it appears to pond before crossing the railway alignment and heading on to the north. The location of the ponding correlates to a low point shown on the contour mapping (see Figure 1-3) of circa 68.30m.

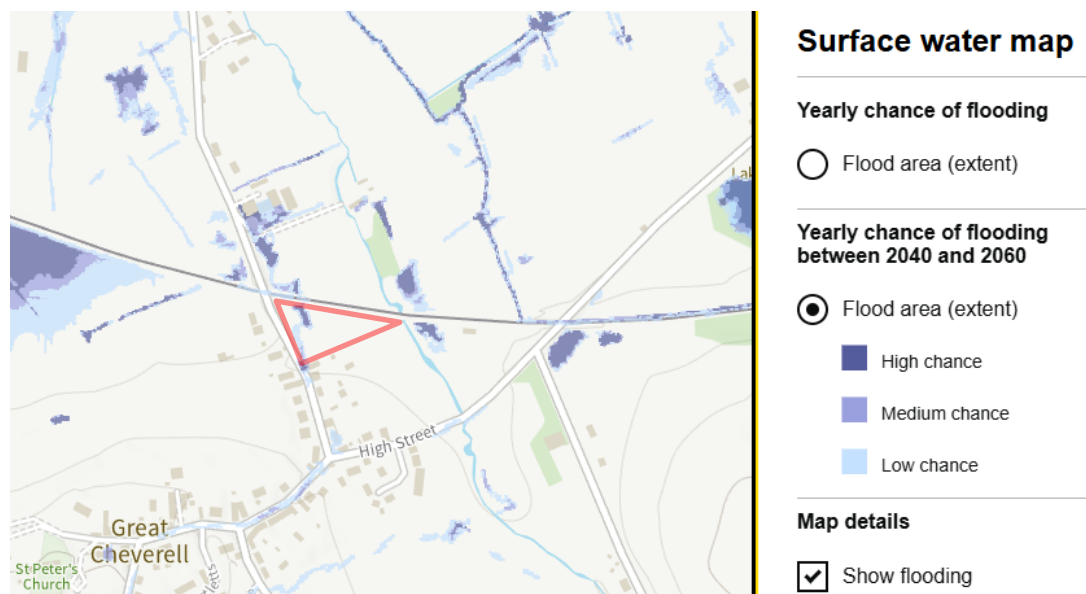


Figure 3-2 Surface Water Flood Risk Map

The site is therefore at a high risk of surface water flooding.

3.3 Groundwater Flood Risk

Groundwater flooding is caused when the underlying water table gets so high that it emerges above ground level. The EA long term flood risk data indicates that the risk of groundwater flooding is low for the site.

3.4 Sewers: Combined, Foul or Surface Water Flood Risk

Sewer flooding is often caused by a blockage, collapse or surcharging due to the volume of water entering it, which may be caused by groundwater ingress or surface water flows. There is a surface water sewer (see Figure 1-6a) following the alignment of The Green and this has a chamber (at an increase in pipe diameter) located opposite the access gate to the paddock. The topographical survey (see Figure 1-3b) shows that at this location there is a low spot in the road and surrounding area. This is the point at which the overland surface water flow (see Figure 3-2) crosses from the road into the paddock. There is therefore a risk that in the event of a capacity restriction in the surface water sewer downstream of the chamber opposite the site entrance that water could emerge (due to the low level of the chamber cover) onto the road. From here it would follow the overland surface water flow route previously identified as impacting the site. The site is therefore at risk of flooding from sewers.

3.5 Reservoirs, Canals & Other Artificial Sources

According to the governments long term flood risk maps the site is not at risk of flooding from reservoirs. There are no other artificial sources of flood risk in the vicinity of the site.

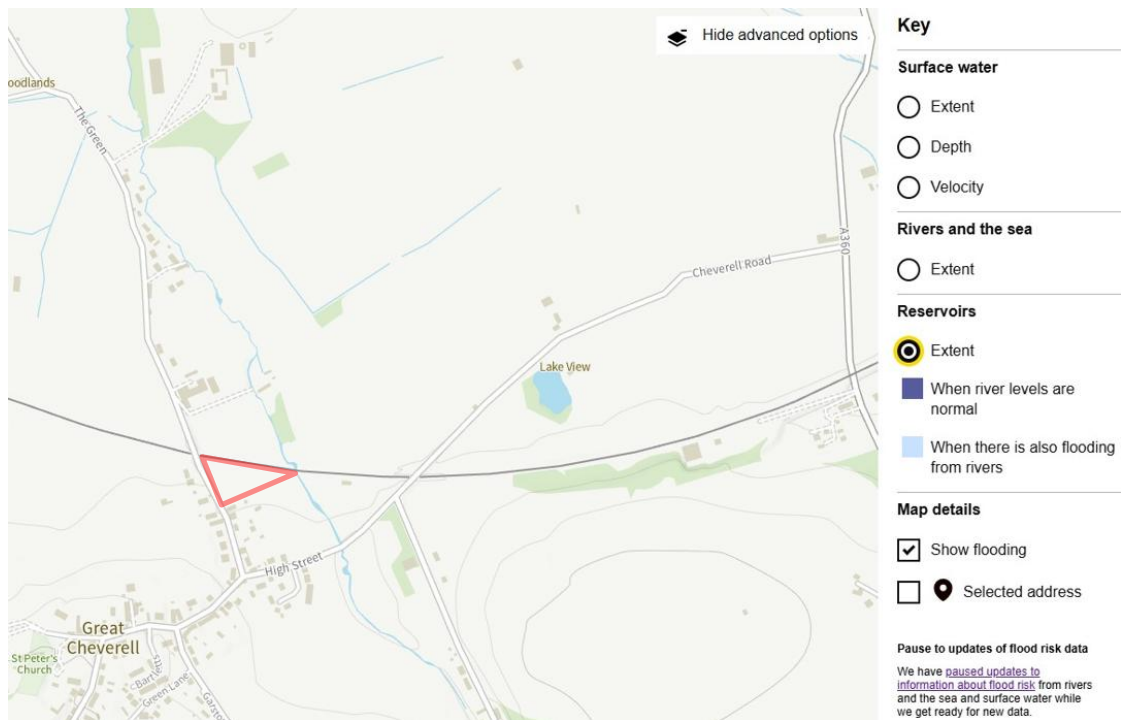


Figure 3-5 Reservoir Flood Risk Map

3.6 Historical Flooding

Photographic evidence of historical flooding events impacting the site has been provided by a local resident. These show overland surface water flowing down The Green in October 2023 (see Photo 3.6a below) and standing water at the site entrance and on the paddock in January 2020 (see Photo 3.6b below).

The form of this historic flooding matches the predicted overland surface water flooding routing and matches the low spots shown on the contour mapping.



Photo 3-6a – Overland flow along The Green just south of the paddock access – October 2023



Photo 3-6b – Standing water on the road and in the paddock – June 2020

3.7 Flood Risk Analysis.

From Sections 3.1 to 3.6 above the site has been determined to be at low risk from all sources except surface water and overland surface water flows. These however are coincidental events and corroborated by historical evidence.

From viewing [Google Streetview](#) it can be seen that the road alignment at the south west corner of the paddock forms a low spot due to the road rising northwards above natural ground level in order form an embankment and then bridge over the railway alignment. This low spot in the road naturally has gullies present; two along the west channel and two along the east (one of which gets impacted by verge growth). However, judging by the apparent Highway boundary demarcations only two of the gullies seem actual Highways (ie Local Authority) gullies, the others appearing to be in private land adjacent. The gullies also have varying grating styles suggesting they were constructed at different times.

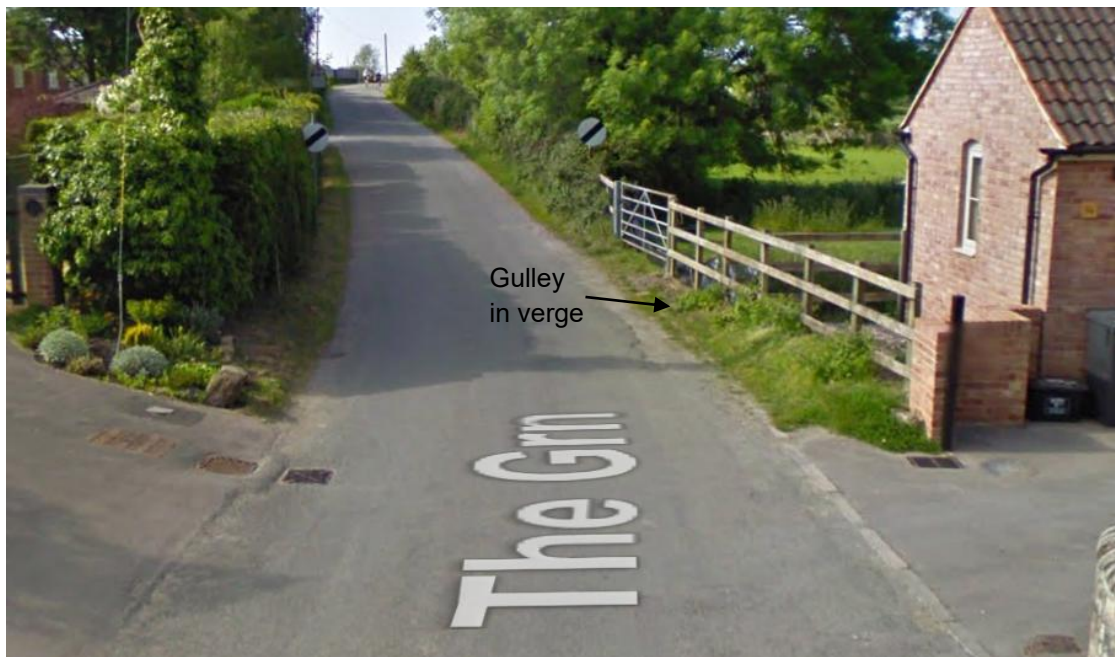


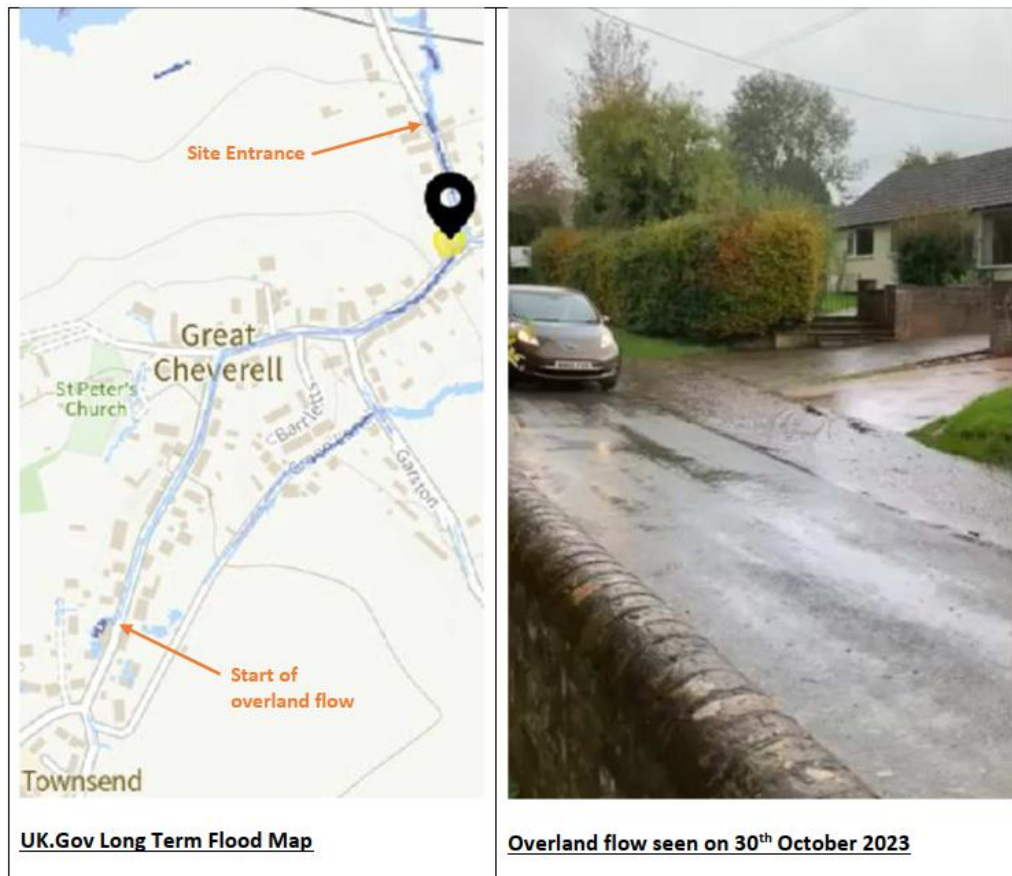
Figure 3-7 Google Streetview in 2009, showing gully locations and extents of adopted highway.

Given the presence of multiple gullies at this low spot the occurrence of historical flooding over them implies that the (Wessex Water) public surface water sewer to which they likely connect is ineffective at discharging higher intensity rainfall events. The cause of this incapacity is beyond the scope of this report but clearly it is a long-standing pre-existing condition.

Associated with the flooding of the road is the consequential but less common flooding of the paddock. From the contour mapping the paddock lies at increasingly lower levels with distance away from the low point in the road. Hence the sequence of flooding would be that standing water on the road forms first and that the paddock offers additional “flood storage” capacity once the road water level overtops the slightly raised verge line. The flooding of the road is thus not a consequence of flooding in the paddock, rather the paddock alleviates the flooding in the road.

These flooding events are all due to overland flows that are in themselves exceedance events, being caused by a lack of capacity in the existing drainage infrastructure located in the low spot of the public highway adjacent to the site access gate.

The exceedance overland flow approaches the site along the main road alignment from the south, as indicated on the UK.Gov's Long Term Flood Risk mapping and witnessed on site in October 2023. These show that surface water flow is channelled along the public highway network. This flow emanates from the southern part of the village of Great Cheverell, some 600m away and at a much higher elevation. The catchment area associated with this rainfall and drainage catchment is thus significant in size and includes public highway, farmland and dozens of private properties. Analysis of the hydrological performance of this catchment, including all the contributing drainage systems, is thus a task beyond the reach of this planning application due to the significant costs that would be expended on topographical and drainage network surveys. As most of these drainage networks belong to third parties it is likely impossible to obtain consents to access them.



Additionally, a detailed condition surveys of the highway and public surface water sewers upstream, adjacent and downstream of the site would be necessary in order to emulate the capacity failure within these Highway and Wessex Water assets that causes the overland flows to eventually breach from the road into the development site.

Without an accurate means of analysing the cause of the exceedance events it is not possible to determine what a 100 year plus climate change event represents, and this is a limitation on the scope that this site specific Flood Risk Assessment's and Drainage Strategy can consider.

It is therefore proposed to take the pragmatic approach of simply considering the mechanics of the events as they are known to unfold and ensure that the development proposals do not negatively impact them but that with some careful design also enable the depth of flooding at the site entrance to be reduced and pathway for overland flows into the northern low lying areas of the paddock formalised for the long term.

4 MITIGATION OF RISK

From an assessment of the site's topography the following mitigation proposals have been established.

4.1 Finished Floor Level

The finished floor level of the proposed dwelling should be set as high as reasonably achievable (given accessibility and drainage design constraints) but at an absolute minimum level of 70.1m. This will ensure it is at least 0.6m freeboard above the predicted Flood Zone 3 of the watercourse to the east, estimated to be around 68.9m and some 1.2m lower.

4.2 Flood Water Flow Path

A ditch/swale should be incorporated along the western edge of the proposed development. This will ensure a continuous low-lying route from the edge of the existing highway around to the paddock. The inlet levels will be such that it will only take water from the road in scenarios of standing water developing on the road and it should freely discharge into the existing low spots in the northern part of the paddock.

4.3 Drainage Separation

A back-to-back kerb, raised 25mm both sides, will be positioned along the interface between the driveway and existing road in order to ensure that under day-to-day rainfall events surface water run-off from the road and driveway are kept separate, with the road run-off directed to the existing road gullies and the driveway run-off captured into the developments proposed surface water SuDS system. A small zone just inside the site will be formed to permanently drain into the swale in order that in flood events the site's private drainage will not collect flood waters.

4.4 Flood Resilience Measures

The surfacing to the frontage driveway should be of an impermeable surfacing such that it is resistant to the detrimental effects of silt deposits that would result from the off-site overland surface water flows that will inevitably impact it's lower lying (western most) extremities.

4.5 Additional Flood Storage Volume

There are depressions within the paddock that are separated by a high spot of land. The west most depression matches the predicted and historic location for standing water on the site. Some localised regrading works will increase the available depression area for flood water storage plus create an overflow route towards the stream to the east. The space for doing this however is limited and constrained by the presence of the railway line and the water main, both of which have protective easements.

4.6 Mitigation Measures Design

In Appendix B to this report is Tumu drawing 19966-TUM-PL-XXX-0210 Flooding Mitigation Works. This drawing provides a co-ordinated detailed design of the above mitigation strategies to demonstrate that the individual mitigations are readily achievable and can be practically integrated within an overall site design.

4.7 Site Access and Egress

The development proposals for the site need to also facilitate safe and timely access and egress of the site by the occupants. This will be demonstrated by considering the sequence of stages associated to the onset of the Highway run-off exceedance surface water flooding event known to impact the area around the proposed site entrance.

Stage 1 – Normal Rainfall Events.

Under ordinary rainfall events surface water on the highway and surface water on the private driveway will be kept separated and directed into their individual separate drainage systems by the presence of the back-to-back bullnose kerbs along the site boundary across the driveway bellmouth. With the top of the highway gulley grating set 20mm below the tarmac road surface water will readily be accepted into the below ground Highway drainage system.

Access and egress to the site will be unimpacted.

Stage 2 – Onset of Exceedance Highway Runoff.

In the event that the flow of water off the Highway exceeds the capacity of the Highway drainage gullies then water depth at the road gulley will begin to increase. At this point it will overtop into the secondary gulley located on the site side of it, but still locally contained by the surrounding 25mm upstand kerbing. Initial exceedance flows will then be diverted via a pipe into the swale to the north and onwards into the depression in the paddock, rather than be allowed to stand as an increasing depth and extent of flooding in the road.

Access and egress to the site will be unimpacted. However, there will be a visual indication that an exceedance flood event is in progress and this will afford an opportunity for the occupants to choose to vacate the new dwelling unhindered.

Stage 3 – Persistence of Exceedance Highway Runoff.

The secondary gulley though has a limited discharge capacity. Should the exceedance event be of sufficient magnitude to reach this then the water level at the second gulley will begin to rise and eventually overtop the 25mm upstand. At this point the area of flooding will have begun to spread although it will initially only have a nominal depth of 25mm.

Access and egress to the site will be unimpacted as the areas away from the gullies will still be free from flooding. Particularly to the north of the gullies (and away from the arriving exceedance flows along the road to the south) surfaces will not yet be flooded and so safe access and egress in the direction of the road bridge over the railway will be freely achievable.

Stage 4 – Ongoing of Exceedance Highway Runoff.

The extent of flooding around the gullies will now start to spread out, into the road and over the driveway. However, the proposed levels form a depression around the gullies so the extent of the flooding area is controlled and will emanate out from gullies location. The proposed levels specifically though create a low-lying alignment in the direction of the headwall and swale. So, as the flood depth increases then water will spread along this alignment until it eventually overtops the flush driveway edging and discharges direct into the swale. At this point the depth of flood water will become maximised as the swale is at a much lower level and will have more than adequate capacity to carry the exceedance flows way and into the paddock.

Access and egress to the site at this time will be impacted due to the full driveway width now being under standing water. However, as the level difference between the top of the gullies and the low point along the northern edge of the bellmouth is only 45mm then the water depth is anticipated to only be circa 75mm maximum. Closer to the northern edge of the driveway the water depth will be much less and getting shallower over a width of only a couple of meters. Flow velocity will be noticeable but not excessive to being transversed by pedestrians, buggies or wheelchairs. Vehicular access and egress will be viable and safe.

In comparison to the existing scenario, where the verge to the road is some 200mm higher than the road surface, the depth of flooding in the road will be significantly reduced.

5 SURFACE WATER DRAINAGE STRATEGY

The surface water drainage strategy has been developed up in accordance with the relevant design standard and guidance, including but not limited to; CIRIA C753 The SuDS Manual, The Approved Documents: Building Regulations 2010 Part H: Drainage and Waste Disposal, and the Non-Statutory Technical Standards for SuDS (NTSS).

The NTSS states that for a previously developed site, the rate of discharge should never exceed the rate pre redevelopment. The NTSS also states that flooding should not occur to any part of the site in the 1 in 30 year event, and no flooding to buildings or utility plant should occur in the 1 in 100 year event.

The Wiltshire Council Drainage and Betterment Strategy (June 2024) has also been consulted when developing the drainage strategy, in order to conform with location specific requirements for the Wiltshire area.

5.1 Surface Water Discharge Location

In line with the surface water drainage discharge hierarchy, the site should aim to discharge in order of priority to: Soakaways/Infiltrating Drainage System, Watercourse, Public Surface Water Sewer then finally, Public Combined Sewer.

Infiltration of surface water on site is not considered a viable solution due to the underlying strata of the site.

The nearest watercourse is an unnamed stream which forms the ownership boundary of the site, to the east. This is the existing receptor for the site's greenfield and overland surface flows and is therefore the appropriate receptor of the developed site's surface water discharge.

5.2 Surface Water Discharge Rates

Existing greenfield runoff rates have been calculated using the FEH Method within the FLOW software package for the associated hardstanding developed area, of 0.047ha. The calculations can be found on page 2 of the calculations summary included in Appendix A of this report.

In order to provide betterments on the existing greenfield runoff rates, in accordance with the Wiltshire Council Drainage and Betterment Strategy (June 2024), a minimal allowable discharge rate of 0.1l/s will be adopted. This provides the betterments outlined in Table 3-1 below.

Rainfall Event	Greenfield Discharge Rate	Proposed Discharge Rate	Betterment
Q _{BAR}	0.1	0.1	0%
Q ₃₀	0.2	0.1	50%
Q ₁₀₀	0.3	0.1	66%

Table 3-1 Proposed Discharge Rate Betterment on Greenfield

As the site's usage is residential, a lifespan of 100 years should be considered. Therefore, when developed (approximately 2025), the end of life can be estimated as 2125. According to government guidance, a building with a lifespan between past 2100 should use a 'upper end' for a 2070s epoch when considering rainfall climate change allowances. For the Avon Bristol and North Somerset Streams Management Catchment area, the climate change allowance to consider should be 40% and 45%, for the 30 and 100 year storms respectively, as shown in Figure 4-1 below.

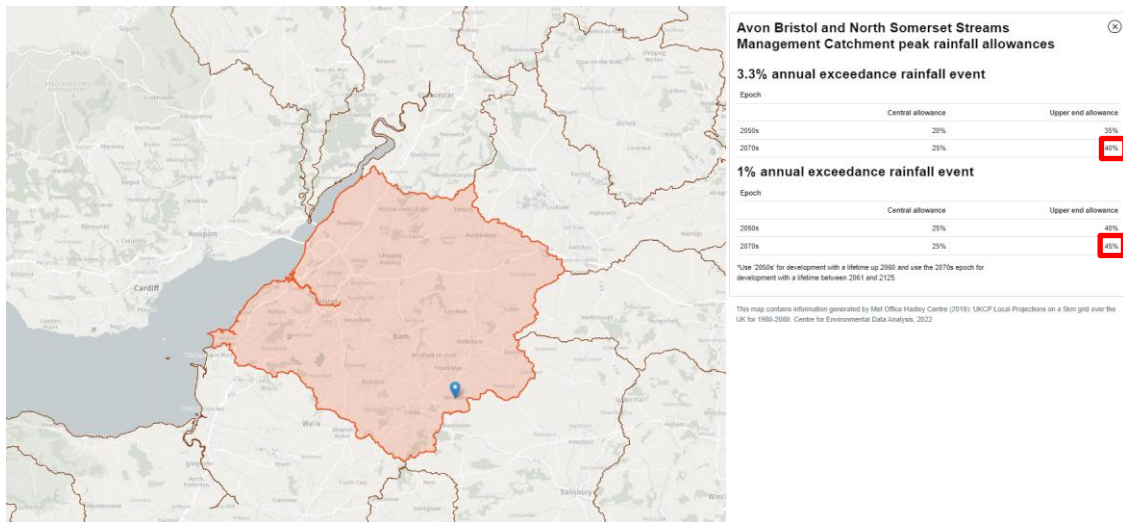


Figure 4-1 Avon Bristol and North Somerset Streams Management Catchment Rainfall Allowances

5.3 Drainage Layout

Flows as a result of pluvial sources will be conveyed from the roof of the dwelling to a surface water drainage network. The access and driveway will be drained from a channel drain and gully, located at the lowest elevations. Surface water will be conveyed to a geocellular attenuation tank and flow control device. This will restrict the outflow to 0.1 l/s, and store surcharging flows and higher intensity storms. Resulting discharge from the system will flow into a swale, which discharges to the eastern corner of the site. Due to well established and dense vegetation existing between the site boundary and the channel of the watercourse the swale has been configured to gently decrease in depth (whilst maintaining a continuous fall) until it peters out onto the surface. Due to the very low discharge rate of 0.1 l/s the discharged water will generally soak into the topsoil layer and be naturally conveyed to the watercourse, with only higher and extreme events potentially generating overland flows. Refer to Appendix B for the Proposed Drainage Layout.

5.4 Surface Water Calculations

The surface water drainage system has been modelled in FLOW to demonstrate the surface water drainage has been appropriately sized. Network parameters, in accordance with the Wiltshire Council Drainage and Betterment Strategy (June 2024) have been used:

- $C_v = 1.0$
- Urban Creep = 10%
- MADD Factor = 0
- Storage in Pipe Network = 0
- Modelled storms = 15 – 2880 min.

In order to achieve the betterments set out in Table 3-1, attenuation will be required to temporarily hold surface water before it can be discharged to the receptor. The surface water system is designed not to flood in the 30 year rainfall event with a 40% allowance for climate change and also in the 1 in 100 year event with a 45% climate change. Refer to Appendix C for the FLOW calculations summary printout.

5.5 Sustainable Drainage

In order to conform with CIRIA 753, The SuDS Manual, sustainable drainage techniques need to be provided in order to naturally treat anticipated pollutants associated with the developed land use. The Simple Index Approach has been implemented to the development. Table 26.2 of The SuDS Manual has been tabulated below, showing the pollutant hazard indices for the associated (worst case) land use.

Land Use	Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Individual property driveways, residential car parks & low traffic roads.	Low	0.5	0.4	0.4

Table 3-2 CIRIA 753 Table 26.2 Pollution Hazard Indices

In order to mitigate the pollution indices associated with the site, the mitigation indices are required to be equal or higher than the pollution hazard indices. Table 26.3 of The SuDS Manual has been tabulated below.

SuDS Component	Total Suspended Solids	Metals	Hydrocarbons
Swale	0.5	0.6	0.6

Table 3-3 CIRIA 753 Table 26.3 SuDS Mitigation Indices

As the mitigation indices are higher than the pollution hazard indices, the SuDS provided on site successfully mitigate against the anticipated pollution associated with the proposed land use. In addition to this, silt traps have been provided on chambers draining hardstanding areas.

As well as the water quality benefits the swale provides, it also provides visual amenity and biodiversity benefits. In less intense storms, the volume of surface water discharging to the watercourse can also be reduced through the process of evapotranspiration.

The proposed drainage also includes the provision of a rainwater harvesting tank which will be used to supply water within the new dwelling for non-potable uses (ie. W/C flushing, washing machine and dishwasher). This will reduce the discharge volume from the developed site.

5.6 Maintenance Arrangements

All surface water drainage pipework will be privately managed and maintained by occupier of the dwelling. Operation and maintenance will be undertaken in accordance with the recommendations outlined in the SuDS Manual (CIRIA C753), provided in tables 3-4, 3-5 and 3-6.

Maintenance schedule	Required action	Typical frequency
Regular Maintenance	Inspect surface structures, removing obstructions and silt as necessary. Check there is no physical damage.	Monthly
	Remove cover and inspect, ensuring water is flowing freely and that the exit route for water is unobstructed. Remove any debris and silt.	Annually
	Undertake inspection after leaf fall in autumn.	
	Jet the pipes leading from overflow structures and check water is going through the overflow	Annually
Remedial actions	Repair physical damage	As required

Table 5-1 Maintenance Requirements for Inlets, Outlets, Controls and Inspection Chambers

Maintenance Schedule	Required Action	Typical Frequency
Regular Maintenance	Inspect and identify any areas that are not operating correctly. If required, take remedial action.	Monthly for 3 months, then annually
	Remove debris from the catchment surface (where it may cause risks to performance)	Monthly
	For systems where rainfall infiltrates into the tank from above, check surface of filter for blockage by sediment, algae, or other matter; remove and replace surface infiltration medium as necessary	Annually
	Remove sediment from pre-treatment structures and/ or internal forebays	Annually, or as required
Remedial actions	Repair/rehabilitate inlets, outlet, overflows and vents	As required
Monitoring	Inspect/check all inlets, outlets, vents and overflows to ensure that they are in good condition and operating as designed	Annually
	Survey inside of tank for sediment build-up and remove if necessary	Every 5 years or as required

Table 5-2 Maintenance Requirements for Geocellular Attenuation

Maintenance schedule	Required action	Typical frequency
Regular Maintenance	Inspection of the tank for debris and sediment build-up, inlets/outlets/withdrawal devices, overflow areas, pumps, filters	Annually (and following poor performance)
	Cleaning of tank, inlets, outlets, gutters, withdrawal devices and roof drain filters of silts and other debris	Annually (and following poor performance)
Occasional Maintenance	Cleaning and/or replacement of any filters	Three monthly (or as required)
Remedial actions	Repair of overflow erosion damage or damage to tank	As required
	Pump repairs	As required

Table 5-3 Maintenance Requirements for Rainwater Harvesting Systems

5.7 Flood Exceedance Routes

The proposed buildings are located on the highest area within the site. In the event of exceedance flows occurring these will be directed away from the buildings and towards the paddock to the north and east where they will eventually either infiltrate or drain away into the eastern watercourse. Limited exceedance flows might be directed towards the west and the site entrance but these will be intercepted by the wider surface water exceedance flooding mitigation works.

Appended to this report is Tumu drawing 19966-TUM-PL-XXX-0201 Exceedance Routes. This shows the pre and post development overland flow paths.

6 FOUL WATER DRAINAGE STRATEGY

The site is crossed by an existing foul water sewer. Under the previous PiP application, the proposed dwelling would straddle this sewer and subsequently a “build over” agreement with Wessex Water had been secured. The building locations proposed in this planning application however no longer straddle the foul sewer but will be “built near” instead. Wessex Water have a set of six standard criteria that if met exempt the need for a formal “build near” agreement. The proposed site layout has been developed to meet these criteria and therefore a formal “build near” agreement is not required. This has been confirmed with Wessex Water.

Foul flows from the new dwelling will be directed into the existing public foul sewer via an existing chamber within the development boundary. This is shown on Tumu drawing 19966-TUM-PL-XXX-0200 Proposed Drainage Strategy, appended to this report.

CONCLUSION

- The site is at low risk of flooding from rivers, seas, groundwater, reservoirs and other artificial sources.
- The site is both predicted to be at future risk of, and historically proven to have suffered from, flooding associated with overland surface water flows and highway drainage related flooding.
- Water emanating from higher land to the south flows along the road alignment of The Green to a low spot adjacent the site entrance. Inadequate performance of the road gullies and/or insufficient capacity in the local surface water public sewer results in standing water across the road. Eventually the depth of this water breaches the level of the slightly raised verge line and water flows onto the paddock where it is temporarily stored in localised depressions adjacent the railway line.
- The proposal to construct a single dwelling on the paddock will ensure as a minimum that the paddock can continue to function as an overflow flood storage provider but additionally offer an opportunity for betterment through increasing the paddock storage volume available.
- Mitigations measures proposed include a minimum ground floor level for the dwelling, levels detailing at the site entrance to control and limit flood levels, provision of a formalised flood water flow path via the inclusion of a swale along the western boundary and works to increase the available flood water storage volume within the paddock.
- The mitigation measures will also allow safe access and egress to and from the proposed dwelling during the anticipated highway run-off flood event for both pedestrians and vehicles, by limiting the depth of the flood waters and providing clear flow paths. Flooding depth at the site entrance will be significantly reduced compared to that which would presently occur.
- The incorporation of the additional gulley at the site entrance will also benefit the adjacent residences by reducing the frequency of the localised flooding in the road and by limiting the depth of all flooding events that occur.
- Infiltration of surface water isn't feasible due to the ground conditions of the site. It is therefore proposed to discharge into the watercourse to the southeast of the site, which is the site's natural receptor.
- The redeveloped site's discharge rate will be limited to a maximum of 0.1 l/s. This results in betterments over the existing greenfield discharge rates by up to 66%. Attenuation in the form of geocellular crates will be provided as attenuation for the heavier rainfall events with climate change allowances (40% and 45%, for the 30 and 100 years respectively).
- A rainwater harvesting tank has been included which will be used to supply water within the new dwelling for non-potable uses. This will reduce the discharge volume from the developed site.
- The proposed foul water drainage will discharge to the foul water sewer that crosses the site. Wessex Water's standard criteria for "build near" are met by the site's proposals.
- It is concluded that the development can be undertaken in a sustainable manner without increasing the flood risk to the proposed development or to existing properties/systems, and comply with both national and local standards.

APPENDIX A

Calculations

Design Settings

Rainfall Methodology	FEH-22	Minimum Velocity (m/s)	1.00
Return Period (years)	100	Connection Type	Level Soffits
Additional Flow (%)	0	Minimum Backdrop Height (m)	0.200
CV	0.750	Preferred Cover Depth (m)	1.200
Time of Entry (mins)	2.00	Include Intermediate Ground	✓
Maximum Time of Concentration (mins)	30.00	Enforce best practice design rules	x
Maximum Rainfall (mm/hr)	50.0		

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Width (mm)	Easting (m)	Northing (m)	Depth (m)
S1	0.008	2.00	70.050	450	450	398417.034	154602.687	0.530
S2	0.002	2.00	69.850	450	450	398418.932	154598.660	0.430
S4	0.002	2.00	70.050	450	450	398430.184	154599.528	0.750
S3	0.008	2.00	70.050	450	450	398440.696	154604.707	0.530
S7	0.023	2.00	70.020	450	450	398415.182	154583.819	1.020
S5	0.007	2.00	70.050	450	450	398432.404	154595.055	0.800
S8			70.020	450		398438.905	154595.693	1.320
S6			70.030	450	450	398437.532	154597.981	0.980
S9			69.970	1200		398449.658	154603.982	1.290
S10			69.100	450	450	398482.582	154615.725	0.770

Links (Input)

Name	US Node	DS Node	Length (m)	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	Link Type	T of C (mins)
1.000	S1	S2	4.452	69.520	69.470	0.050	89.0	100	Circular	2.09
1.001	S2	S4	11.285	69.420	69.300	0.120	94.0	150	Circular	2.27
2.000	S3	S4	11.719	69.520	69.350	0.170	68.9	100	Circular	2.21
1.002	S4	S5	4.994	69.300	69.250	0.050	99.9	150	Circular	2.36
1.003	S5	S6	5.904	69.250	69.050	0.200	29.5	150	Circular	2.41
1.004	S6	S8	2.668	69.050	68.700	0.350	7.6	150	Circular	2.42
1.005	S8	S9	13.577	68.800	68.720	0.080	169.7	150	Circular	2.80
3.000	S7	S8	26.529	69.000	68.800	0.200	132.6	150	Circular	2.51
1.006	S9	S10	34.956	68.680	68.330	0.350	99.9	100	Circular	3.56

Simulation Settings

Rainfall Methodology	FEH-22	Drain Down Time (mins)	240	30 year (l/s)	0.2
Rainfall Events	Singular	Additional Storage (m³/ha)	20.0	100 year (l/s)	0.3
Summer CV	0.750	Starting Level (m)		Check Discharge Volume	x
Analysis Speed	Normal	Check Discharge Rate(s)	✓		
Skip Steady State	x	2 year (l/s)	0.1		

Storm Durations

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

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
2	0	0	0
30	40	0	0
100	45	0	0

Pre-development Discharge Rate

Site Makeup	Greenfield	Growth Factor 30 year	1.95
Greenfield Method	FEH	Growth Factor 100 year	2.48
Positively Drained Area (ha)	0.047	Betterment (%)	0
SAAR (mm)	713	QMed	0.1
Host	1	QBar	0.1
BFIHost	0.622	Q 2 year (l/s)	0.1
Region	1	Q 30 year (l/s)	0.2
QBar/QMed conversion factor	1.111	Q 100 year (l/s)	0.3
Growth Factor 2 year	0.90		

Node S9 Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	x	Sump Available	✓
Invert Level (m)	68.680	Product Number	CTL-SHE-0014-1000-0750-1000
Design Depth (m)	0.750	Min Outlet Diameter (m)	0.075
Design Flow (l/s)	0.1	Min Node Diameter (mm)	1200

Node S9 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	68.700
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	46.0	46.0	0.750	46.0	64.0	0.751	0.0	64.0

Results for 2 year Critical Storm Duration. Lowest mass balance: 97.56%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	9	69.552	0.032	1.3	0.0162	0.0000	OK
15 minute summer	S2	9	69.450	0.030	1.6	0.0089	0.0000	OK
15 minute summer	S4	9	69.346	0.046	3.2	0.0118	0.0000	OK
15 minute summer	S3	9	69.549	0.029	1.3	0.0147	0.0000	OK
15 minute summer	S7	9	69.051	0.051	3.8	0.0330	0.0000	OK
15 minute summer	S5	9	69.291	0.041	4.4	0.0157	0.0000	OK
15 minute summer	S8	9	68.885	0.185	8.2	0.0295	0.0000	OK
15 minute summer	S6	9	69.076	0.026	4.4	0.0054	0.0000	OK
480 minute summer	S9	488	68.851	0.171	1.4	6.7937	0.0000	SURCHARGED
480 minute summer	S10	488	68.337	0.007	0.1	0.0000	0.0000	OK

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute summer	S1	1.000	S2	1.3	0.619	0.203	0.0094	
15 minute summer	S2	1.001	S4	1.6	0.457	0.087	0.0399	
15 minute summer	S4	1.002	S5	3.2	0.747	0.179	0.0213	
15 minute summer	S3	2.000	S4	1.3	0.693	0.178	0.0219	
15 minute summer	S7	3.000	S8	3.8	0.528	0.246	0.2060	
15 minute summer	S5	1.003	S6	4.4	1.467	0.133	0.0178	
15 minute summer	S8	1.005	S9	7.6	0.773	0.559	0.1346	
15 minute summer	S6	1.004	S8	4.4	0.369	0.067	0.0263	
480 minute summer	S9	1.006	S10	0.1	0.240	0.009	0.0081	2.0

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 98.53%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	9	69.594	0.074	5.0	0.0372	0.0000	OK
15 minute summer	S2	9	69.480	0.060	6.2	0.0178	0.0000	OK
15 minute summer	S4	9	69.403	0.103	12.4	0.0263	0.0000	OK
15 minute summer	S3	9	69.584	0.064	5.0	0.0323	0.0000	OK
15 minute summer	S7	9	69.338	0.338	14.3	0.2205	0.0000	SURCHARGED
15 minute summer	S5	9	69.335	0.085	16.7	0.0322	0.0000	OK
480 minute summer	S8	480	69.246	0.546	3.6	0.0868	0.0000	SURCHARGED
15 minute summer	S6	10	69.252	0.202	16.8	0.0409	0.0000	SURCHARGED
480 minute summer	S9	480	69.246	0.566	3.5	24.5208	0.0000	SURCHARGED
480 minute summer	S10	464	68.339	0.009	0.1	0.0000	0.0000	OK

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute summer	S1	1.000	S2	5.0	0.853	0.781	0.0261	
15 minute summer	S2	1.001	S4	6.2	0.636	0.339	0.1096	
15 minute summer	S4	1.002	S5	12.4	1.069	0.698	0.0579	
15 minute summer	S3	2.000	S4	5.0	0.973	0.686	0.0602	
15 minute summer	S7	3.000	S8	12.2	0.695	0.796	0.4670	
15 minute summer	S5	1.003	S6	16.8	1.702	0.510	0.0824	
15 minute summer	S8	1.005	S9	26.2	1.487	1.928	0.2357	
15 minute summer	S6	1.004	S8	14.8	0.902	0.228	0.0470	
480 minute summer	S9	1.006	S10	0.1	0.279	0.015	0.0112	3.1

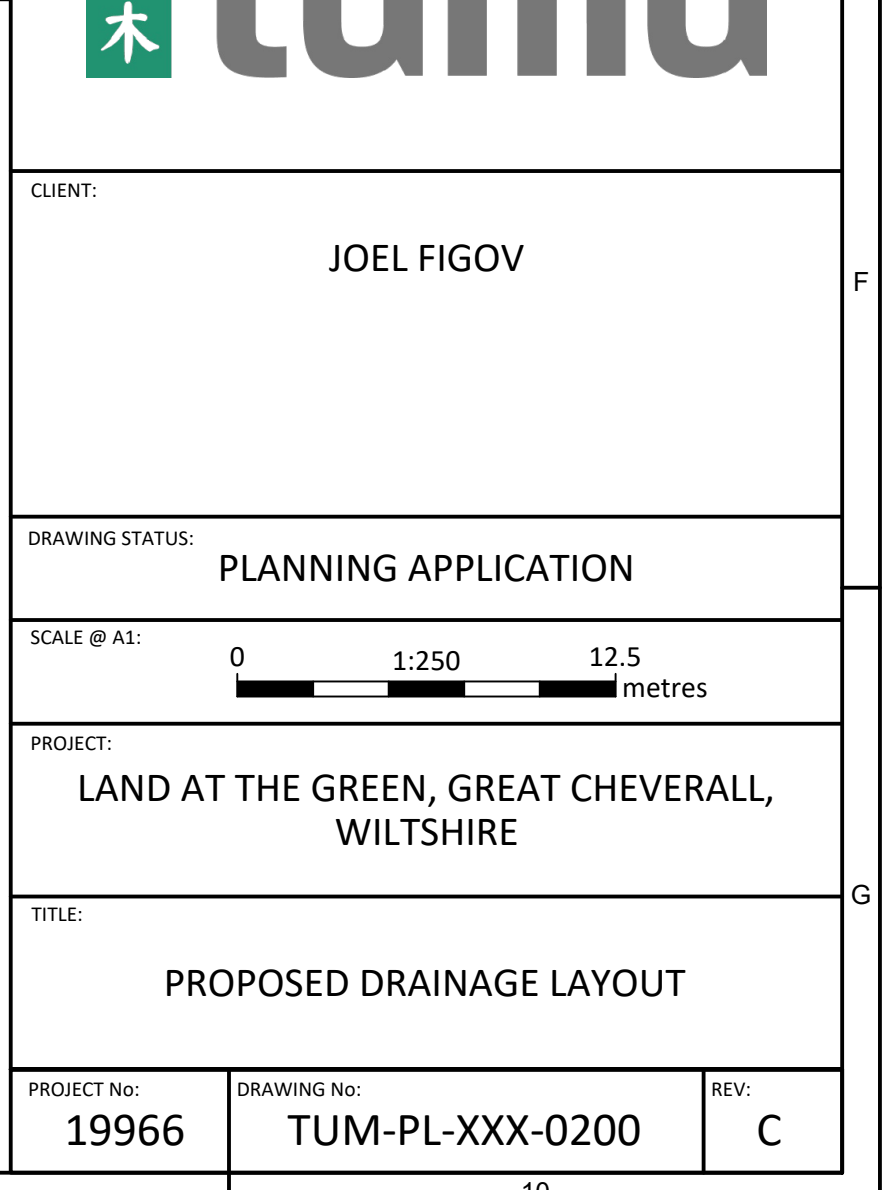
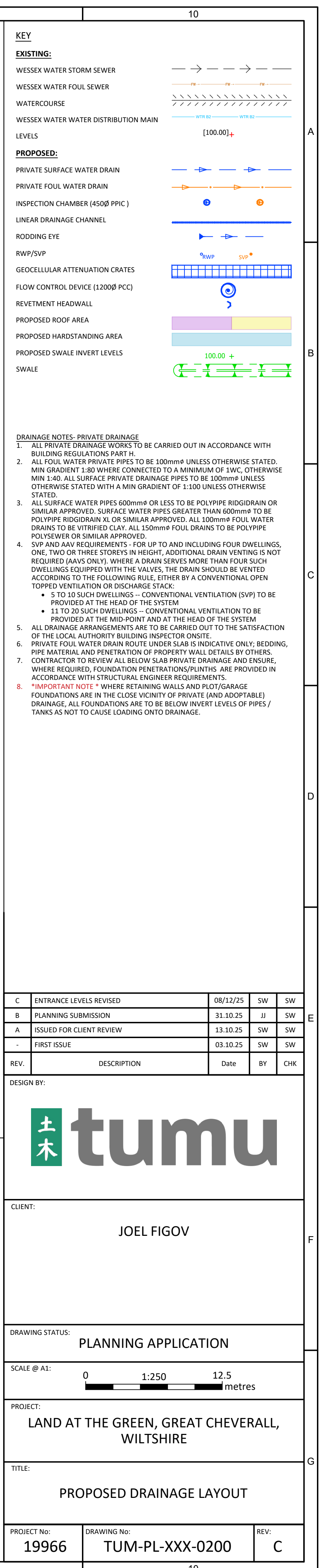
Results for 100 year +45% CC Critical Storm Duration. Lowest mass balance: 97.77%

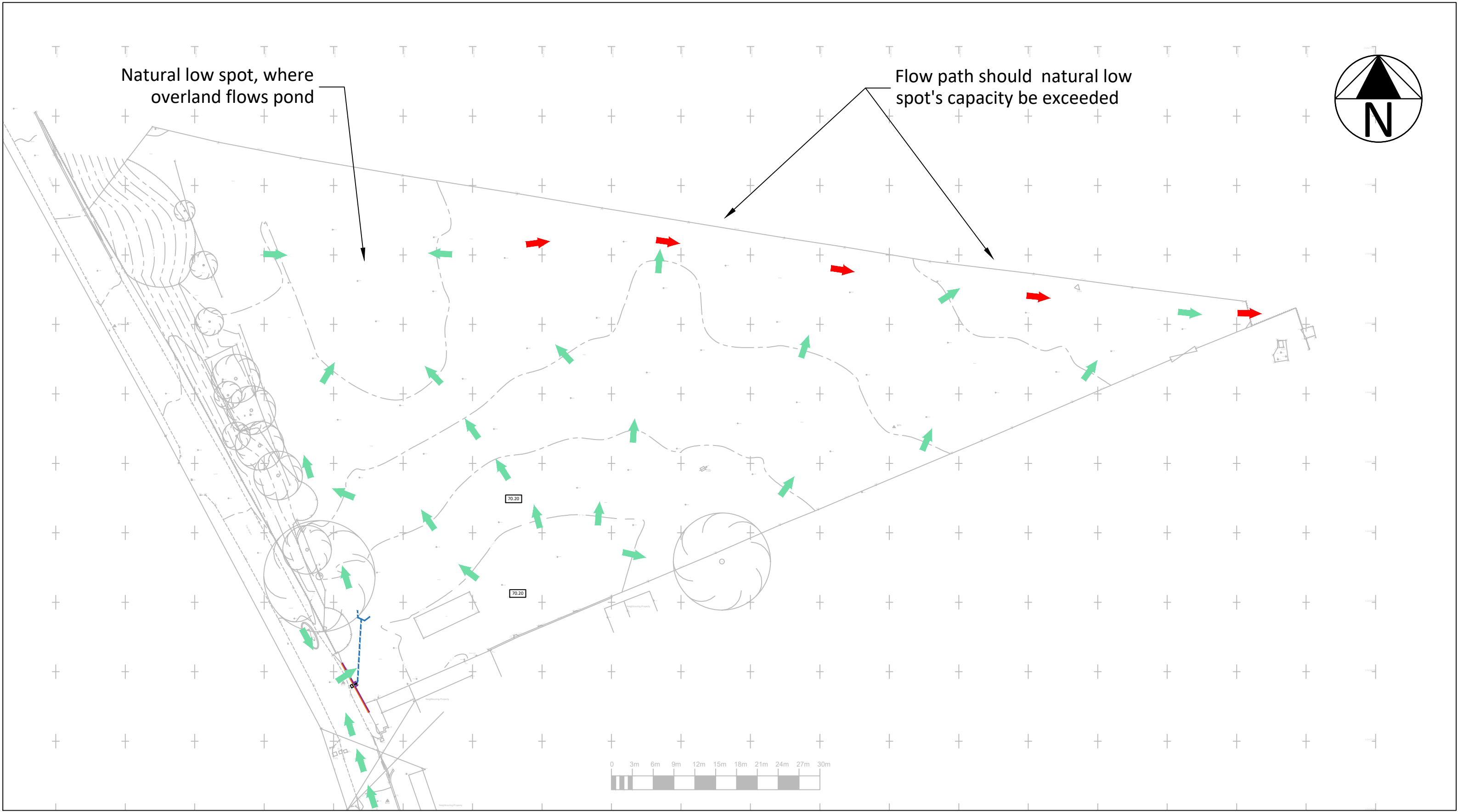
Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	9	69.622	0.102	6.5	0.0516	0.0000	SURCHARGED
720 minute summer	S2	720	69.622	0.202	0.7	0.0598	0.0000	FLOOD RISK
720 minute summer	S4	720	69.622	0.322	1.4	0.0824	0.0000	SURCHARGED
720 minute summer	S3	720	69.622	0.102	0.6	0.0514	0.0000	SURCHARGED
720 minute summer	S7	720	69.622	0.622	1.6	0.4062	0.0000	SURCHARGED
720 minute summer	S5	720	69.622	0.372	1.9	0.1406	0.0000	SURCHARGED
720 minute summer	S8	720	69.622	0.922	3.3	0.1466	0.0000	SURCHARGED
720 minute summer	S6	720	69.622	0.572	1.9	0.1161	0.0000	SURCHARGED
720 minute summer	S9	720	69.622	0.942	3.3	33.8842	0.0000	SURCHARGED
720 minute summer	S10	720	68.339	0.009	0.1	0.0000	0.0000	OK

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute summer	S1	1.000	S2	6.4	0.862	0.998	0.0326	
15 minute summer	S2	1.001	S4	7.4	0.660	0.401	0.1808	
15 minute summer	S4	1.002	S5	13.8	1.059	0.775	0.0879	
15 minute summer	S3	2.000	S4	6.2	0.988	0.847	0.0846	
15 minute summer	S7	3.000	S8	16.0	0.909	1.040	0.4670	
15 minute summer	S5	1.003	S6	18.7	1.822	0.570	0.1039	
15 minute summer	S8	1.005	S9	31.0	1.761	2.283	0.2390	
15 minute summer	S6	1.004	S8	16.7	0.951	0.258	0.0470	
720 minute summer	S9	1.006	S10	0.1	0.297	0.018	0.0130	4.7

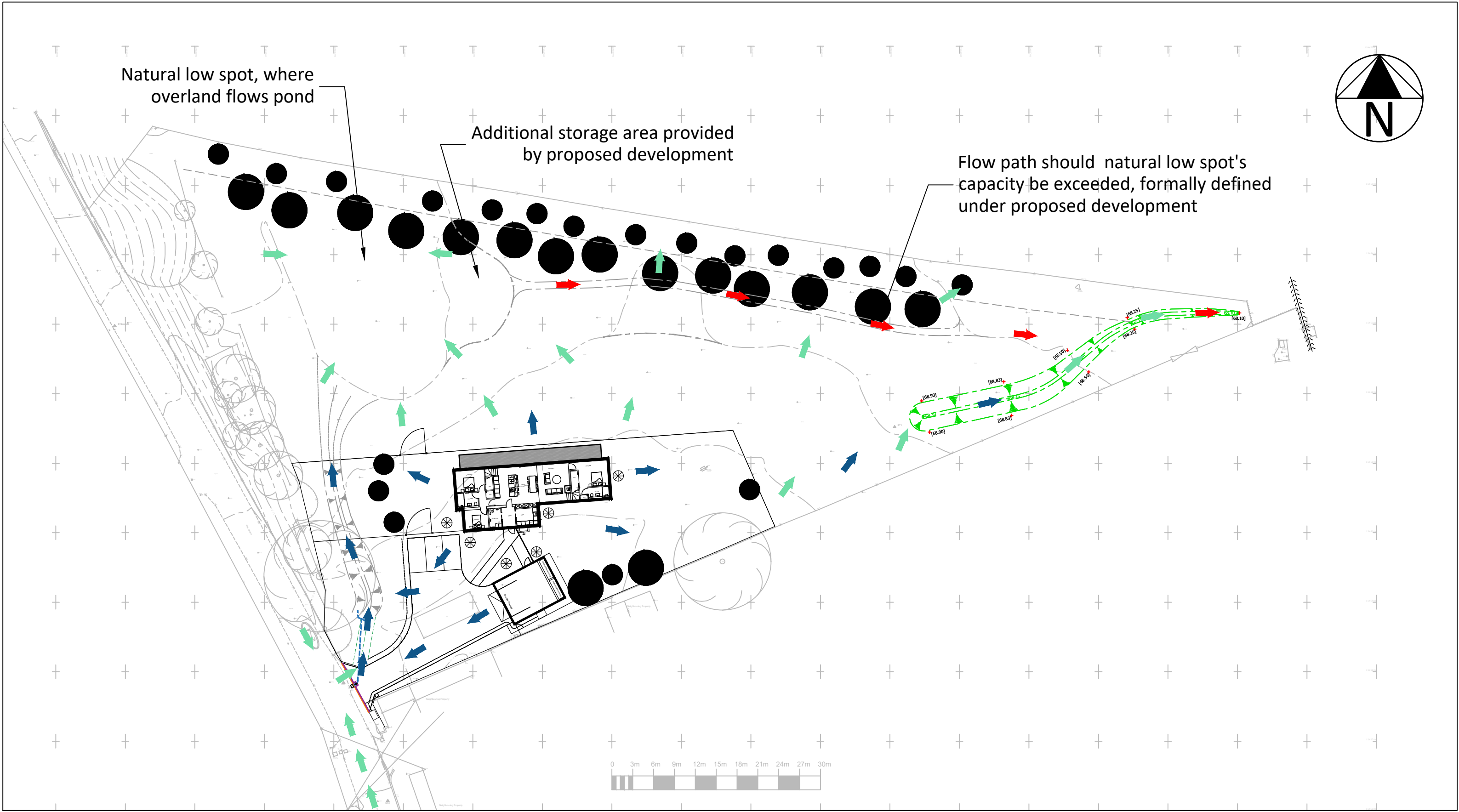
APPENDIX B

Drawings





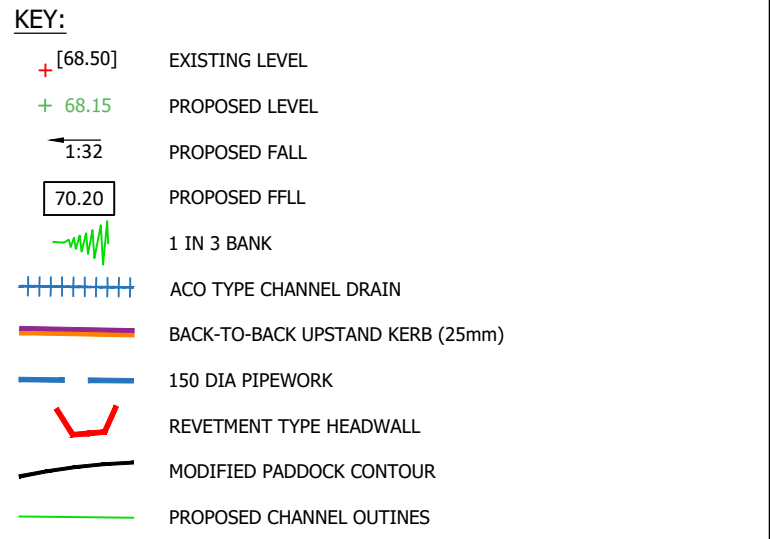
EXISTING EXCEEDANCE/OVERLAND FLOW ROUTES



PROPOSED EXCEEDANCE/OVERLAND FLOW ROUTES

- KEY
- NATURAL OVERLAND FLOW ROUTES
 - NATURAL LOW SPOT EXCEEDANCE ROUTES
 - PROPOSED DRAINAGE EXCEEDANCE ROUTES

A	PLANNING APPLICATION	31.10.25	JJ	SW
REV.	DESCRIPTION	Date	BY	CHK
DESIGN BY:				
CLIENT:				
JOEL FIGOV				
DRAWING STATUS:				
PLANNING APPLICATION				
SCALE @ A1:				
PROJECT:				
LAND AT THE GREEN, GREAT CHEVERALL, WILTSHIRE				
TITLE:				
EXCEEDANCE ROUTES				
PROJECT No:	DRAWING No:	REV:		
19966	TUM-PL-XXX-0201	A		



C	ENTRANCE LEVELS REVISED	08/12/25	SW	SW
B	PLANNING SUBMISSION	31/10/25	JJ	SW
A	ISSUED FOR REVIEW	13/10/25	JJ	SW
-	FIRST ISSUE - WORK IN PROGRESS	22/09/25	SW	SW
REV.	DESCRIPTION	Date	BY	CHK

DESIGN BY:



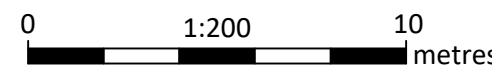
CLIENT:

JOEL FIGOV

DRAWING STATUS:

PLANNING APPLICATION

SCALE @ A1:



PROJECT:

LAND AT THE GREEN, GREAT CHEVERALL,
WILTSHIRE

TITLE:

SITE LEVELS AND FLOOD MITIGATION WORKS

PROJECT No:
19966

WING No: TUM-PI-XXX-0270

KEY: