



David Lloyd Clubs Chippenham

Flood Risk Assessment and SUDS Strategy

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Project Details

Project	David Lloyd Club Chippenham
Location	Southpoint Business Park, Chippenham

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EXECUTIVE SUMMARY

This report has been prepared for David Lloyd Leisure (DLL) and assesses the level of flood risk to and from the proposed development, as well as describing the sustainable drainage strategy and any maintenance and management requirements for the proposed development based on the latest national and local planning requirements and guidance.

- Site is almost entirely in EA Flood Zone 3, High probability, land assessed as having a 1% (1 in 100) or more chance of flooding from rivers and 0.5% (1 in 200) or more chance of flooding from the sea.
- Surface water runoff from the site is to be discharged via a 525mm spur provided by Stantec. The spur and onward drain will route to the adjacent existing surface water drainage system within the newly built access road serving the wider development. Maximum flow rate of discharge from the site is limited to 1.5 l/s/HA with a calculated runoff rate of 4.99l/s, less than the calculated QBAR greenfield runoff rate for the site.
- Surface water runoff controlled via SuDS features for all events up to and including the 1 in 100-year storm + 25% climate change, in line with National and Local Planning Policy Requirements.
- Use of SuDS including porous subbases to the external playing courts with dedicated flow controls, and attenuation basin to provide surface water conveyance, attenuation, water quality and biodiversity benefits.
- The quality of the surface water discharge is in line with the recommendations in The SuDS Manual (CIRIA C753).
- Supporting calculations incorporating all SuDS features and piped drainage network included as part of this strategy report.
- Maintenance regimes proposed for all drainage features to be carried out by a maintenance company or onsite staff.
- Foul drainage will discharge via a 150mm spur to be provided by Stantec. The spur will route to the existing foul water drainage system within the newly built estate road serving the wider development. The maximum flow rate from the site will not exceed 4.12 l/s.
- Overall, the report demonstrates that the drainage design has been prepared to meet the requirements of both national and local guidance. It also provides a betterment on greenfield run off rates in severe storms by limiting peak rates of discharge to a value less than Qbar rate for all storms up to and including the 1 in 100-year event plus a 25% allowance for climate change.

1 INTRODUCTION

1.1 Appointment

Willis Hazell Engineers Limited (Willis Hazell) has been appointed by David Lloyd Leisure (DLL) for civil and structural engineering services on their proposed new leisure club in Chippenham. The appointment includes preparation of this Flood Risk Assessment and SuDS Strategy, in support of a Full Planning Application for the proposed development.

1.2 Description of the Project

The project is for a new Leisure Club, to be located on a greenfield site approximately 3.33ha in size on land north of West Cepen Way A350, Southpoint Business Park, Chippenham. Chippenham town centre is located approximately 2.8km to the northeast of the proposed development. The proposed development comprises of the main club building, including gym space and associated areas, indoor & outdoor swimming pools, tennis dome, external terrace, pool terrace, spa garden and relate space, children's play area, battle box, padel courts and additional padel social space, along with car parking & landscaping for the facility.

1.3 Purpose of Report

This report has been prepared for David Lloyd Leisure (DLL) and assesses the level of flood risk to, and resulting from the development, as well as describing the sustainable drainage strategy and any associated maintenance and management requirements for the proposed development.

The aim of this report is to define the following, to a suitable level of detail to obtain planning approval for the development:

- Site's characteristics (slope profile, soil geology, existing flood risk (from fluvial flooding and other potential sources of flooding).
- Calculated surface water runoff rates and required storage volumes.
- Where the runoff is to be discharged to.
- The sustainable drainage system (SuDS) features proposed.
- How the drainage system is going to be managed and maintained.

1.4 Limitations

This report has been produced exclusively for the use of DLL. Any other party seeking to rely on this report should seek written permission from Willis Hazell. This report has been produced for the proposed Chippenham club as currently shown on architect's drawings. If the proposals change then findings of this report should be reviewed accordingly.

In preparation of this statement, we have relied on information provided by third parties and as such, Willis Hazell cannot accept responsibility for the accuracy of the information received.

2 SITE DESCRIPTION

2.1 Site Location and Existing Site Usage

The site of the proposed David Lloyd leisure club development is located on a greenfield site approximately 3.33ha in size, on land north of West Cepen Way A350, Southpoint Business Park, Chippenham. Chippenham town centre is located approximately 2.8km to the northeast of the proposed development. The site location is indicated on **Figure 1** below, and a more detailed Site Location Plan can be found in **Appendix A**.

The site comprises an undeveloped parcel of land characterised by overgrown vegetation, shrubs and weeds. Small stockpiles of unknown materials are present sporadically across the site. A large railway embankment is present adjacent to the west of the site and overhead services along the bottom of the embankment. The site is bound by mature trees to the north and northwest and West Cepen Way (A350) to the south and southwest site boundary.

Access to the site is provided via a newly built development estate road to the east of the site. The estate road is connected to West Cepen Way at the south of the site and heads northeast connecting the wider Southpoint Business Park development.

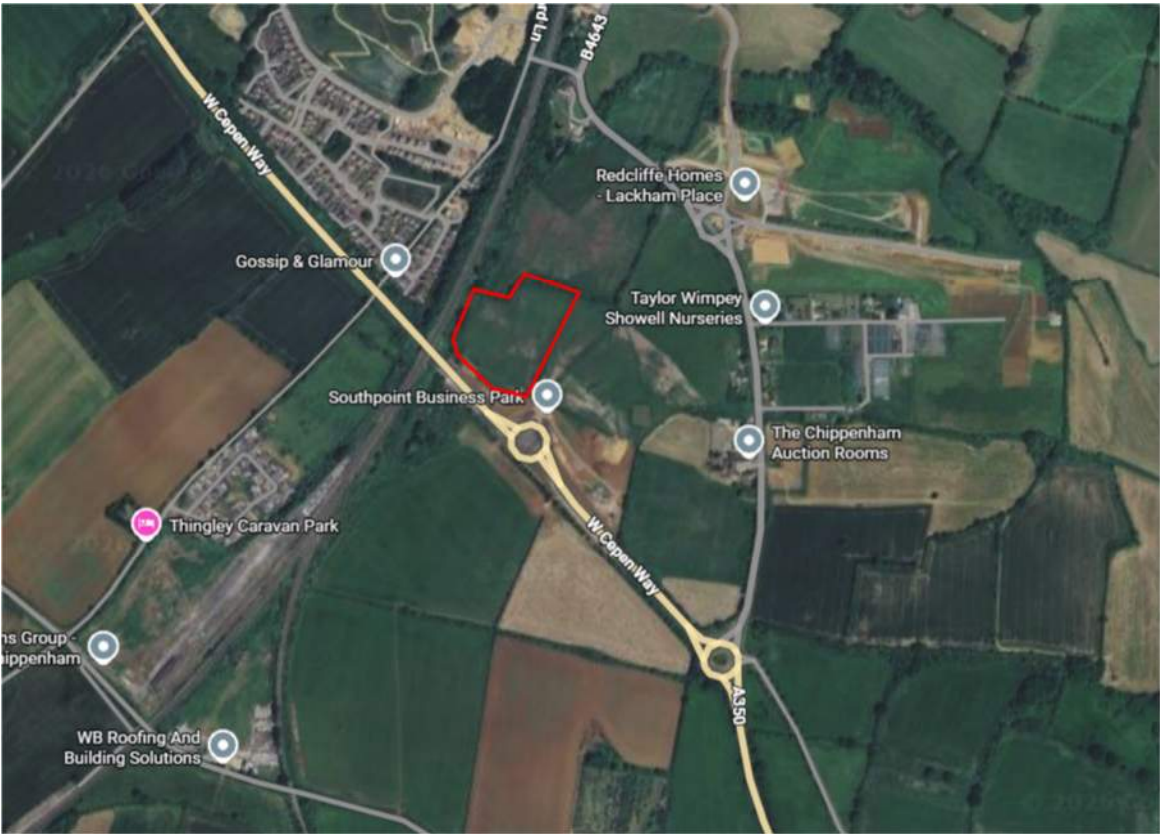


Figure 1: Aerial Photography indicating Site Location

2.2 EA Flood Zone

The site is shown by Environment Agency mapping to be almost entirely in fluvial Flood Zone 3, High probability, land assessed as having a 1% (1 in 100) or more chance of flooding from rivers and 0.5% (1 in 200) or more chance of flooding from the sea.

The flood risk to the site from various other potential sources of flooding is considered in more detail in **Section 5** of this report.

2.3 Site Topography

A topographical survey of the site was undertaken by Malcolm Hughes Land Surveyors in May 2025. The survey indicates that ground levels are highest in the southwest corner of the site, reaching approximately 56.90m AoD. The topography slopes predominantly towards the northeast with the levels reaching approximately 52.00m AoD.

The site extends approximately 270m from the southwest corner to the northeast corner, with a total level change of around 4.88m, this equates to an average gradient of approximately 1 in 44. The existing ground level at the proposed site access, which connects to the existing car park, is approximately 52.76m AOD.

The Topographical survey drawings can be found in **Appendix B**, and the existing site plan can be found in **Appendix C**.

2.4 Ground Conditions

The British Geological Society (BGS) online mapping indicates the site has no superficial or artificial deposits. The underlying bedrock geology is identified as Cornbrash Formation Limestone.

Geology	Description /strata	BGS General Description (Lexicon)
Artificial	None recorded	NA
Superficial	None recorded	NA
Bedrock	Cornbrash Formation	Limestone, medium- to fine-grained, predominantly bioclastic wackestone and packstone with sporadic peloids; generally and characteristically intensely bioturbated and consequently poorly bedded, although better bedded, commonly somewhat arenaceous units occur in places, particularly in the upper part. Generally bluish grey when fresh, but weathers to olive or yellowish brown. Thin argillaceous partings or interbeds of calcareous mudstone may occur

Figure 2: Published Geology from Phase II GI report

A Phase 2 ground investigation was completed by GeoCon Site Investigations Ltd in June 2025. A generalised summary of the ground conditions encountered, depths to base, and thickness recorded from the exploratory trial holes confirmed the BGS findings.

Geology / Strata	Depth (mbgl)				Thickness (m)		Locations encountered
	Top		Base		Min	Max	
	Min	Max	Min	Max			
Topsoil	0.00	-	0.10	0.30	0.10	0.30	TP01-TP10, DS06-DS09
Cornbrash Formation	0.10	0.30	0.50	3.50	0.50	3.20	All locations
Groundwater	Strike depth (mbgl)		Rest Level (mbgl)		Comments		Locations encountered
	Min	Max	Min	Max			
Strike	-	-	-	-	NA		Not encountered
SWL at end of borehole	-	-	-	-	NA		Not encountered
Seepage	1.80	2.30	1.80	2.30	NA		TP02, TP04, TP06, TP07, TP08, TP09, TP10.

Figure 3: Summary of Ground Conditions from Phase II GI report

The field work comprised of the following:

- Dynamic sampling at 8 to 10 locations to a nominal depth of up to 5.00 mbgl or refusal
- Trial pitting at 10 locations to a nominal depth of up to 3.50 mgbl or refusal
- 3 infiltration tests in accordance with BRE 365
- CBR tests at 20 to 25 locations using the TRL DCP method
- Plate loading tests at 6 locations
- Environmental and geotechnical sampling and insitu geotechnical testing
- 3 combined gas and groundwater monitoring standpipes to a nominal depth of 5.00 mbgl
- Subsequent gas and groundwater monitoring on 3 occasions over a 2-month period commencing approximately 1 week after the completion of the ground investigation

The bedrock aquifer designation for the site is Secondary A. There are 3 groundwater abstractions recorded within 1000m of the site all for spray irrigation.

Groundwater was encountered in 7 of the trial pits.

- TP02 seepage at 1.80mbgl
- TP04 seepage at 1.80mbgl
- TP06 seepage at 2.00mbgl
- TP07 seepage at 1.90mbgl
- TP08 seepage at 2.30mbgl
- TP09 seepage at 2.00mbgl
- TP10 seepage at 1.80mbgl

A groundwater monitoring regime was carried out on site to establish ground water levels. The results of the ground water monitoring are shown below.

Exploratory Hole ID	Base of Hole (mbgl)	Standing Water Level (SWL) (mbgl)		
		Date of Visit		
		11/07/2025	01/08/2025	18/08/2025
DS01	0.90	Dry	Dry	Dry
DS02	1.17	Dry	Dry	Dry
DS04	1.25	Dry	Dry	Dry
Old Location	11.60	1.55	1.50	1.60

Table 1: Summary of Water Strikes and Monitoring

During each visit the boreholes were dry. However, it should be recognised that that minor seepages were encountered within some of the trial pits suggesting that groundwater may be present in isolated perched pockets. Seasonal variation in groundwater levels is expected.

Soakaway tests were carried out in 3 locations (TP0/SA01, TP03/SA02, TP05/SA03) across the site to provide infiltration rates all conducted in each location in accordance with BRE 365.

Parameter	Soakaway Test Location TP01/SA01		
	Test 1	Test 2	Test 3
Assumed Drain Invert Level (mbgl)	1.02	-	-
Total Effective Storage Depth (m)	1.98	-	-
75% Effective Storage Depth (mbgl)	0.99	-	-
25% Effective Storage Depth (mbgl)	1.52	-	-
Volume (m3), $V_{p75} - V_{p25}$	0.89	-	-
Area (m2), a_{p50}	5.06	-	-
Time (minutes), t_{p75-25}	0.00	-	-
Soil infiltration rate (m/s), f	Unsuitable	-	-
Parameter	Soakaway Test Location TP03/SA02		
	Test 1	Test 2	Test 3
Assumed Drain Invert Level (mbgl)	1.80	-	-
Total Effective Storage Depth (m)	1.70	-	-
75% Effective Storage Depth (mbgl)	3.08	-	-
25% Effective Storage Depth (mbgl)	2.23	-	-
Volume (m3), $V_{p75} - V_{p25}$	0.77	-	-
Area (m2), a_{p50}	4.47	-	-
Time (minutes), t_{p75-25}	0.00	-	-
Soil infiltration rate (m/s), f	Unsuitable	-	-
Parameter	Soakaway Test Location TP05/SA03		
	Test 1	Test 2	Test 3
Assumed Drain Invert Level (mbgl)	1.13	-	-
Total Effective Storage Depth (m)	2.37	-	-
75% Effective Storage Depth (mbgl)	2.91	-	-
25% Effective Storage Depth (mbgl)	1.72	-	-
Volume (m3), $V_{p75} - V_{p25}$	1.07	-	-
Area (m2), a_{p50}	5.88	-	-
Time (minutes), t_{p75-25}	0.00	-	-
Soil infiltration rate (m/s), f	Unsuitable	-	-

Table 2: Summary of Infiltration Test Results

Tests were kept shallow to remain above the groundwater level. The derived infiltration rates are summarised above in **Table 2**.

Soakaway tests were allowed to drain over an eight-hour period and showed no signs of draining. All locations encountered natural groundwater at 3.00mbgl which then caused the water added for the test to rise. The 25% effective storage depth was not achieved during the 8hr working shift; therefore, it was not possible to calculate a soil infiltration rate.

Based on the results of the soakaway tests carried out, the site is therefore considered unsuitable for infiltration. A soil infiltration rate cannot be determined due to the soakaway test not draining to at least 75% effective storage depth within a 24hr period.

Relevant extracts of the Ground Investigation report can be found in **Appendix D**.

2.5 Existing Watercourses

The closest Main River to the site is the River Avon, located approximately 1.1km to the east. The River Avon is designated as a Main River by the Environment Agency and flows south through Wiltshire then west through Bath and Bristol before eventually into the Severn estuary.

Several branches of the River Avon meander through the surrounding agricultural and greenfield areas. The nearest of these is situated approximately 180m north of the site boundary shown to pass under the railway and head east past Hill top park from an area called Easton. Another branch is located 390m metres to the east, in an area called Lower Lodge.

2.6 Existing Site Drainage and Public Sewers

The site is currently laid to grass, with no evidence of a formal or positive drainage system within the boundary. Any greenfield runoff that does not infiltrate is therefore likely to flow naturally from the southwest corner toward the northeast corner of the site, continuing as overland flow onto the adjacent greenfield land.

There are no public sewers located within the site boundary. However, foul and surface water drainage networks are present within the development estate road to the east of the site. These networks serve the wider development and have been constructed to adoptable standards. Stantec, the engineer working on behalf of the developer of the wider 'Southpoint Business Park,' has provided a copy of the Section 104 as-built drainage drawings for the estate road infrastructure. These are included in **Appendix E**.

The as-built drawings indicate that the surface water network extends north along the estate road before turning east towards an attenuation basin. Discharge from the attenuation basin ultimately outfalls to an existing tributary of the River Avon, which flows eastward to the main river.

The foul drainage network is shown to discharge north along the development estate road to a new sewage pumping station. This infrastructure is understood to have been constructed to adoptable standards in consultation with Wessex Water.

2.7 Greenfield Runoff Rate

As mentioned above, the site is comprised of entirely greenfield space, thus runoff rates from the site can be calculated using standard greenfield runoff calculation tools. Based on using the FEH rural runoff calculation tool in HR Wallingford software, applied to the proposed catchment area of 3.33ha the QMED value has been calculated as 7.1 l/s, where QMED represents the median annual flood. A copy of the HR Wallingford calculations can be found in **Appendix F**.

QBAR, the average (mean) annual flood can be calculated by applying a factor to the QMED value, which the online Wallingford calculation tool shows to be 1.075, meaning that the QBAR value can be calculated as follows:

$$\text{QBAR} = 1.075 \times \text{QMED} = 1.075 \times 7.1 = 7.633 \text{ l/s.}$$

In addition, the online Wallingford calculation tool gives growth factors for various storm return periods, from which greenfield runoff rates for these return period storm events can be calculated, as summarised below in **Table 3**.

Table 3: Summary of Greenfield Runoff Rates

Return Period Event	Return Period Growth Factor	Greenfield Runoff Rate
QBAR	n/a	7.7 l/s
Q 1:1yr	0.78	6.0 l/s
Q 1:30yr	1.95	15.0 l/s
Q 1:100yr	2.43	18.6 l/s

3 PROPOSED DEVELOPMENT

The project proposes the construction of a new David Lloyd leisure club. The proposed development comprises of the main club building, including gym space and associated areas, indoor & outdoor swimming pools, tennis dome, external terrace, pool terrace, spa garden and relate space, children's play area, battle box, padel courts and additional padel social space, along with car parking & landscaping for the facility.

The two-way entrance to the proposed David Lloyd club site is to be formed joining a newly built development estate road on the eastern side of the site. From the new access, a 333-space car park is proposed, along the eastern and southern boundary of the site.

The main building, containing the gym and other areas is located directly west of the car park, with the tennis dome situated in the north-western corner of the site. The proposed outdoor swimming pool, external terraces, and other outdoor sports facilities extend along the southwestern part of the site.

A Proposed Site Plan is included in **Appendix G**.

4 PLANNING POLICY REQUIREMENTS

4.1 NPPF and Planning Practice Guidance

The principal aim of the National Planning Policy Framework (NPPF) is to achieve sustainable development, including ensuring that flood risk is taken into account at all stages of the planning process, avoiding inappropriate development in areas at risk of flooding, and directing development away from areas where risks are highest.

The document also states that surface water arising from a developed site should, as far as is practicable, be managed in a sustainable manner to mimic the surface water flows arising from the site prior to the proposed development. The document also states that opportunities to reduce the flood risk to the site itself and elsewhere, taking climate change into account, should be investigated.

The Flood Risk and Coastal Change category of the Planning Practice Guidance which accompanies the NPPF states that the aim should be to discharge surface runoff from a development as high up the following hierarchy of drainage options as reasonably practicable:

1. Into the ground (via infiltration)
2. To a surface water body
3. To a surface water sewer, highway drain, or another drainage system
4. To a combined sewer

4.2 Wiltshire Local Flood Risk Management Strategy (LFRMS)

Wiltshire Council is the Lead Local Flood Authority for the county and has prepared a Local Flood Risk Management Strategy (LFRMS) to describe its approach to flooding and how it will work with other organisations.

Flood risk within Wiltshire is often a complex interaction between river, surface, ground water and sewer flooding, and the local geology is a significant factor. Northern Wiltshire is predominantly underlain by clay, with flooding generally caused by river or excessive surface water runoff. The south is characterised by underlying chalk deposits, with potential risk of ground water flooding in many areas.

Wiltshire's Councils Local Flood Risk Strategy seeks to

- Improve knowledge regarding flood risk
- Improve protection from flooding
- Improve resilience to flooding
- Improve the environment
- Improve communications about flooding issues.

4.3 Wiltshire Council Strategic Flood Risk Assessment (SFRA)

Paragraph 160 of the NPPF states that '*strategic policies should be informed by a strategic flood risk assessment and should manage flood risk from all sources. They should consider cumulative impacts in, or affecting, local areas susceptible to flooding, and take account of advice from the Environment Agency (EA) and other relevant organisations, such as Lead Local Flood Authorities (LLFAs) and Internal Drainage Boards (IDBs).*'

Wiltshire Council Level 1 SFRA (2024) is used to inform decisions on the location of future development and preparation of land use planning policies for the long-term management of flood risk, reflecting the implications of the August 2022 changes to the Planning Practice Guidance (PPG).

After consultation with Wiltshire Council, eight sites were proposed to go forward for a Level 2 assessment as a result of the findings of the Level 1 site screening and councils sequential test and short-listing process. Land south of Chippenham and east of Showell Farm (PoI7) was selected.

The main objectives of the Level 2 SFRA (2025) are to:

1. Provide individual flood risk analysis for site options using the latest available flood risk data, thereby assisting the council in applying the Exception Test to their proposed site options in preparation of their local plan.
2. Using available data, provide information and comprehensive mapping presenting flood risk from all sources for the site.
3. Where the Exception Test is required, provide recommendations for making the site safe throughout its lifetime.
4. Take into account most recent policy and legislation in the NPPF, PPG and LLFA SuDS guidance.
5. Consider whether the catchments may be sensitive to new development in flood risk terms using the Level 1 SFRA Cumulative Impact Assessment (CIA).

4.4 Wiltshire Council Surface Water Management Plan (SWMP)

URS Scott Wilson produced a SWMP on behalf of Wiltshire Council in November 2011. The SWMP provides an understanding of the surface water flooding mechanism and provides an evidence base to inform the planning process.

There are no specific references to the site in the SWMP report.

5 FLOOD RISK ASSESSMENT

5.1 EA Fluvial Flood Zone and Sequential Test

The site is shown by the Environment Agency's online Flood Map for Planning (Rivers and Sea) to be located within Flood Zone 3, High probability, land assessed as having a 1% (1 in 100) or more chance of flooding from rivers and 0.5% (1 in 200) or more chance of flooding from the sea.

An extract of this mapping is shown below in **Figure 4**. **Figure 5** below shows the west side of the site to have a medium chance and the east side a high chance of flooding shown in blue and are associated with the associated brooks of the River Avon, with the closest approximately 390m to the east of the boundary of the site. The main body of the river Avon is approximately 1.1 km away.



Figure 4: Environmental Agency Flood Map for Planning



Figure 5: Yearly Chance of Flooding

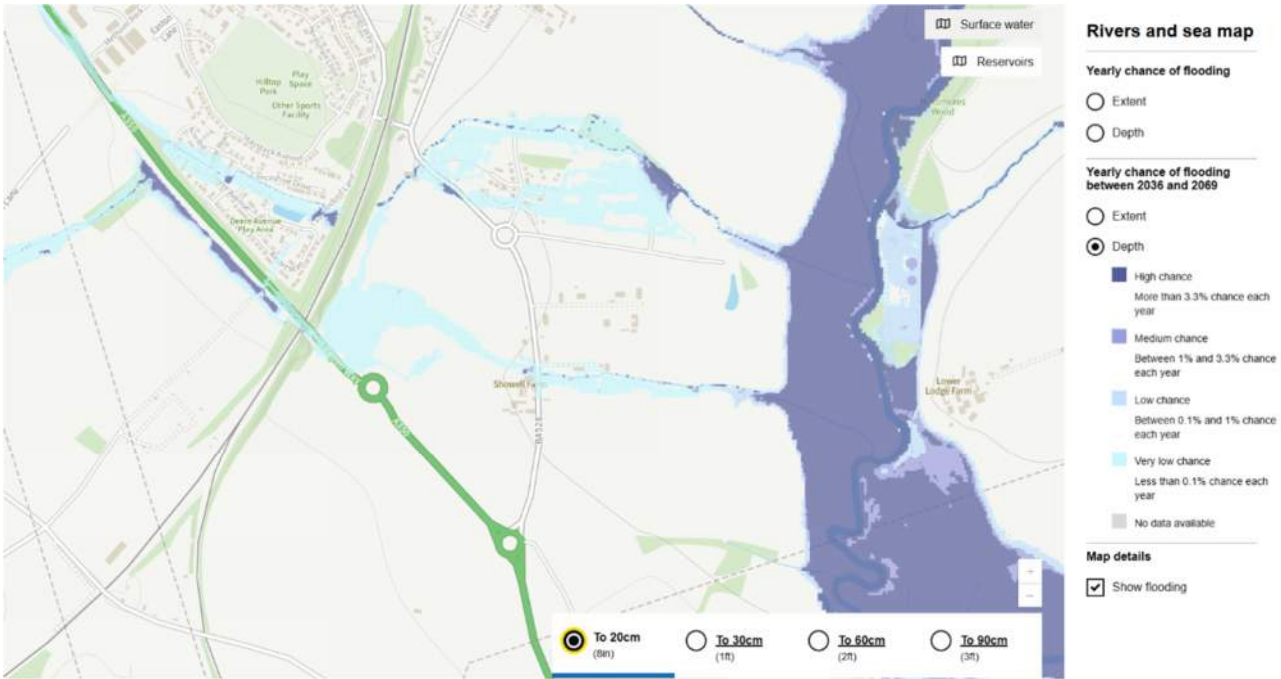


Figure 6: Yearly Chance of Flooding Depth

The above image shows the flood depth up to 20cm is very low (less than 0.1% chance each year) in a yearly chance of flooding between 2036 and 2069.

In summary there is a medium to high chance of flooding within the site boundary, but the chance of it reaching 20cm in depth is very low.

It is understood that Stantec have been working with the Environment Agency to better define the extent of flood risk to the site, and that the flooding indicated on the flood map for planning is understood to be resulting from overland flows coming on to the site from further west, along the A350. It is also understood that Stantec have agreed in principle a strategy of lowering site levels around the perimeter of the site so as to divert flooding around the edges of the site towards the watercourse further north, therefore reducing the flood risk to the development. Further details of these proposals will be provided in due course.

According to the National Planning Policy Framework (NPPF), the suitability of all new developments with respect to flood risk must be assessed through the application of the Sequential Test. This test is intended to demonstrate that there are no reasonably available sites in areas of lower flood risk that would be appropriate for the proposed type of development.

The application site lies almost entirely within Flood Zone 3, which is classified as having a high probability of flooding. On this basis the site will require a sequential test that will be done in due course. The access to the site is clear of the flood zone 2 & 3 extents. The estate access road is elevated above existing ground levels and access/ egress down to the A350 is outside the flood extents.

The flood map for planning and larger versions of the flooding images shown in Section 5 can be found in **Appendix H**.

The remainder of this section of the report provides an assessment of flood risk from other potential sources of flooding, in line with the requirements of the NPPF.

5.2 Surface Water Flood Risk

The Environment Agency's Flood Map for Planning (Surface Water) shows the level of flood risk from surface water runoff for the site and the surrounding area. Extracts of this mapping are shown below in **Figures 7 and 8**. The mapping shows most of the site is at Very Low Risk of surface water flooding (i.e. areas with no blue shading), defined as having a less than 1 in 1,000 annual probability of surface water flooding (<0.1%).

However, the mapping identifies a small, isolated area of elevated surface water flood risk in the north-eastern corner of the site. This location corresponds to a local low point where overland surface water is likely to accumulate. The topographical survey confirms that this area is at a lower level than the adjacent land on either side.

This area of higher surface water flood risk is located at lower elevations and a proposed attenuation basin, not an area for built development. As part of the development proposals, site levels will be re-graded to manage and direct surface water appropriately. Any residual surface water flooding that may occur along the eastern boundary of the site are expected to continue to run overland through the landscaped soft margins around the perimeter of the site, thereby not posing any flood risk to the proposed development.

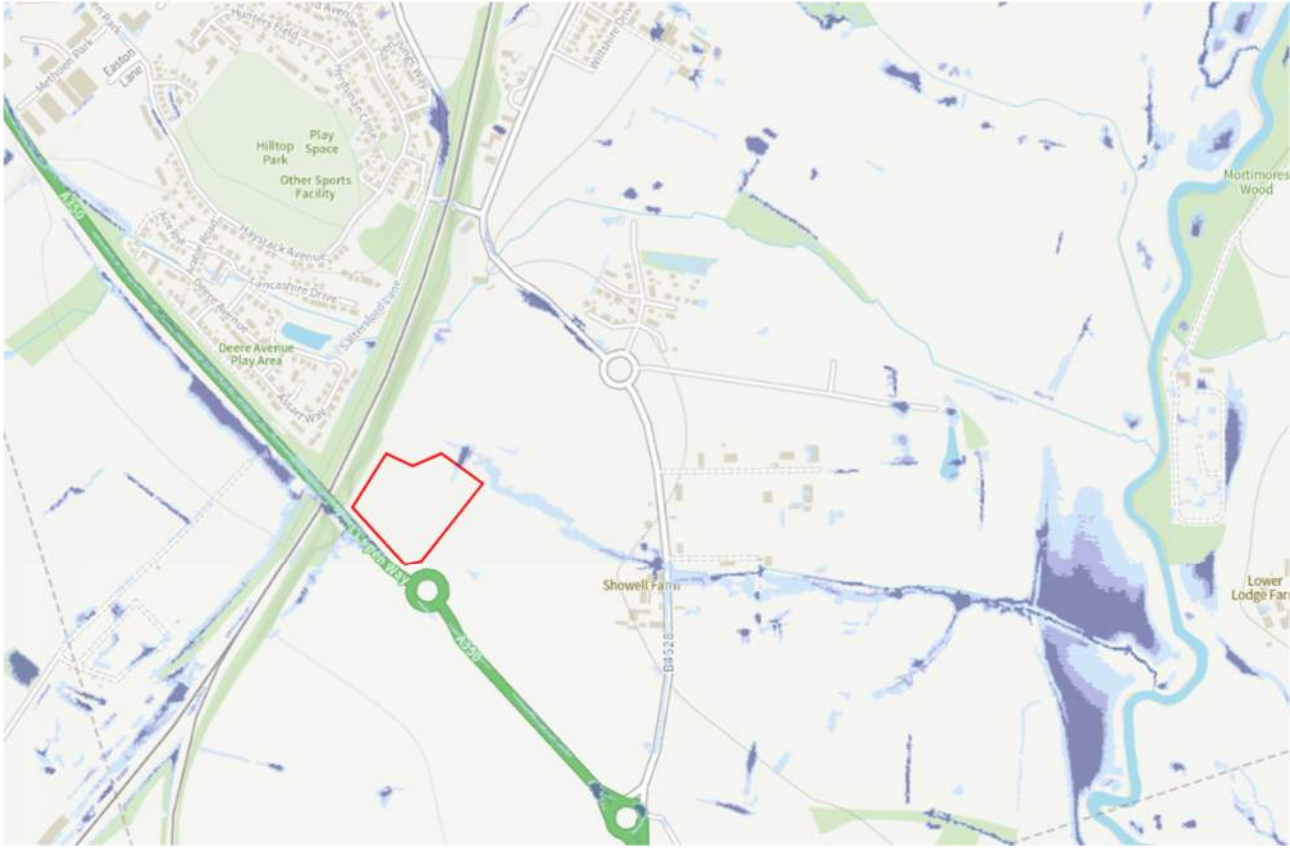


Figure 4: Environmental Agency Surface Water Flood Map yearly chance between 2040 and 2060

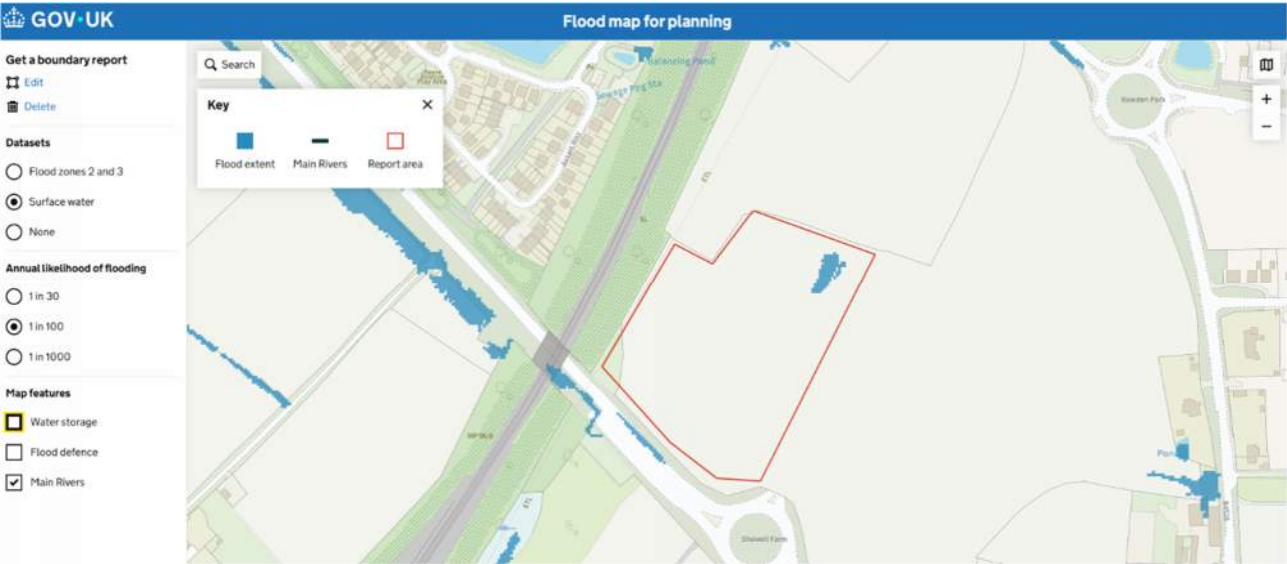


Figure 5: Environmental Agency Surface Water Flood Map for Planning (1 in 100 year)

5.3 Groundwater Flood Risk

According to section 5.4 of the SWMP level 1 and 2, there is potential for groundwater flooding in the Chippenham area. The SWMP uses the EA's 'Areas susceptible to groundwater flooding' dataset to determine groundwater flood risk across Wiltshire.

The site lies above a high vulnerability Secondary A Aquifer according to the EA's groundwater vulnerability maps and situated within the outer zone (zone 2) of Groundwater Source Protection Zone (SPZ).

As outlined in Section 2.4 of the report, minor groundwater seepages were encountered in several of the trial pits, indicating the potential presence of isolated perched groundwater pockets.

At all soakaway testing locations, groundwater was encountered at a depth of approximately 3.00 metres below ground level (mbgl). The presence of groundwater at this depth resulted in a rise in water levels during testing following the introduction of test water.

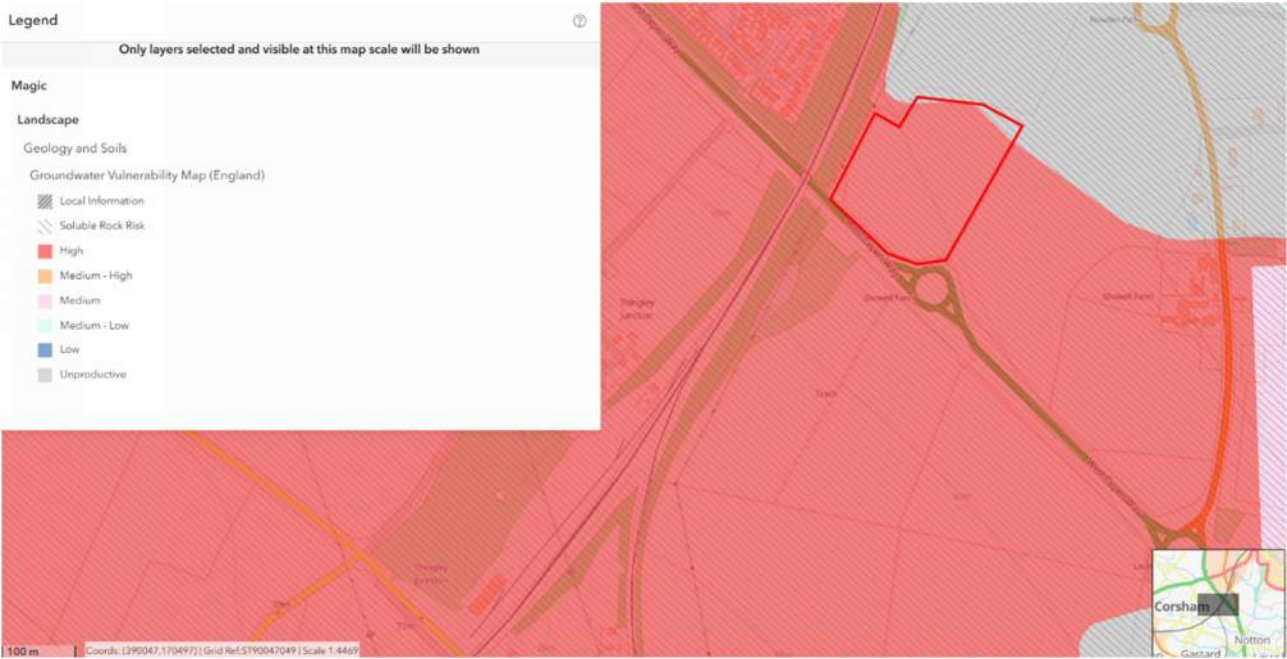


Figure 6: Aquifer Designation Map

In the event of any flooding from groundwater occurring, it would most likely occur within the lowest lying areas of the site towards the northeastern site boundary and would therefore have a direct overland flow away from the built development that is proposed, thereby not posing any flood risk to the proposed development.

5.4 Flood Risk from Sewers

As described in Section 2.6 of this report, there are no public foul or surface water sewers within the vicinity of the site. The upstream end cover level of the estate road drainage network serving the wider development is situated at least 1.3m below the ground levels in the northeastern corner of the David Lloyd site. Therefore, should any flooding occur because of a storm event causing the capacity of these estate road drainage networks to be exceeded, this would not pose a flood risk to the proposed development.

5.5 Flood Risk from Reservoirs, Canals and other Artificial Sources

A search on the Environment Agency website shows that flooding from reservoirs is possible in this area. Most of the site shows flooding when there is also flooding from rivers, with a small part of the eastern boundary showing flooding when river levels are normal. There are no canals or other artificial waterbodies in the immediate vicinity of the site.



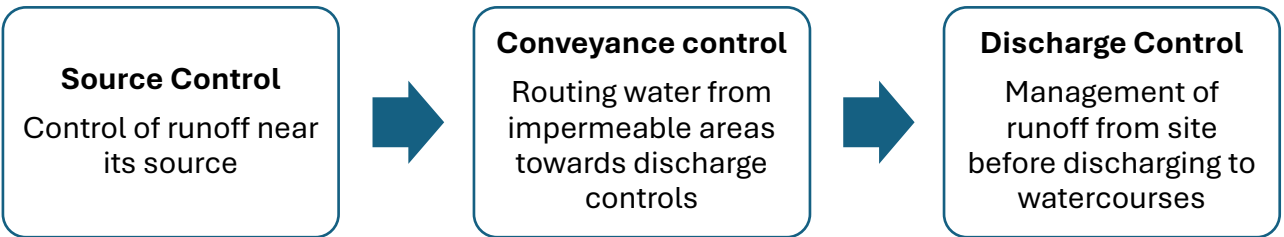
Figure 7: Reservoir Flooding Extent

6 SUSTAINABLE DRAINAGE STRATEGY

6.1 General Drainage Design Principles

In accordance with the NPPF and the Wiltshire Council Drainage and Betterment Strategy (June 2024), a SuDS approach has been used to develop a strategy for controlling the surface water runoff from the development. SuDS are water sensitive drainage systems which mimic natural catchment processes to manage runoff from developments. A "treatment train" approach, utilising various SuDS techniques, is fundamental to designing a successful SuDS strategy.

The "treatment train" philosophy uses drainage techniques to systematically control the three elements of runoff: pollution, flow rates, and volumes. This is typically achieved in three main steps: Source Control, Conveyance Control, and Discharge Control. Source Control techniques are preferred to those further down the treatment train, as they lead to the retention of pollutants and control of water before it enters the receiving drainage network(s) and/or watercourse(s).



6.2 Proposed Discharge Strategy

As discussed in Section 4.1 of this report, the aim of any sustainable drainage strategy should be to discharge surface runoff from a development as high up the following hierarchy of drainage options as reasonably practicable:

1. **Rainwater re-use** (rainwater harvesting / greywater recycling)
2. **Adequate soakaway** or other infiltration system
3. **Hybrid solution** of infiltration and discharging to a surface water body
4. To a **surface water body** (e.g. an ordinary watercourse)
5. To a **surface water sewer**, highway drain, or another drainage system
6. To a **combined sewer**

As described in Section 2.4 of this report, soakaway testing states that there is no infiltration option for the site due to the presence of a high groundwater table. The ground investigation (GI) report concluded that the widespread use of soakaways for drainage is not viable and recommended that an alternative drainage solution is sought. Given this, the use of infiltration as a means of disposal of surface water runoff from the development has been discounted.

Stantec have provided a surface water spur into the site and the onward drainage route to connect to the wider development drainage network. The as-built drawings indicate that the surface water network extends north along the estate road before turning east toward an attenuation basin. Discharge from the attenuation basin ultimately outfalls to an existing tributary of the River Avon, which flows eastward to the main river.

Table 4 below shows a list of potential SuDS features that have been considered for inclusion within the surface water drainage design for the development, along with brief commentary as to their suitability for this development.

Table 4: Summary of Potential SuDS Options

Potential SuDS Component	Description	Assessment of Suitability for inclusion in this development
Green / Blue Roofs	In green roofs, a planted soil layer is constructed on the roof of a building to create a living surface. Water is stored in the soil layer and absorbed by vegetation. Blue roofs provide attenuation typically constructed in thin geo-cellular "crates" at roof level.	Discounted due to structural implications and is unlikely to provide a sizeable improvement to discharge rates across the site.
Rainwater harvesting	Rainwater is collected from the roof of a building or from other paved surfaces and stored in an overground or underground tank for treatment and reuse. Water could be used for toilet flushing and/or irrigation.	May be feasible, subject to further consideration of potential for water re-use during detailed design stage. Water butts to be provided as a minimum for use on planting.
Permeable Paving	Paving which allows water to pass through it. Can be in the form of paving blocks with gaps between solid blocks or porous paving where water filters through the paving blocks themselves. Porous asphaltic materials are also available. Water can be stored in the sub-base beneath prior to discharge and/or be allowed to infiltrate into the ground below.	Could offer attenuation and treatment of runoff in the car park. Utilised in external sports facilities (tennis / padel courts & MUGA) to provide attenuation of runoff from these areas which will have minimal surface falls.
Filter Strips	Filter strips are grassed or planted areas that runoff is allowed to run across to promote infiltration and cleaning.	Discounted due to lack of space alongside main car parks / access roads.
Bioretention Areas / Raingardens / Trees	Vegetated areas with gravel & sand layers below designed to channel, filter, and cleanse water vertically. Water can infiltrate into the ground below or drain to a perforated pipe and be conveyed elsewhere.	Limited areas to provide such features. Due to site layout unlikely to provide reasonable storage for its incorporation.
Swales	Vegetated depressions designed to convey and filter water. These can be "wet" where water gathers above the surface or "dry" where water gathers in a gravel layer beneath. Can be lined or unlined to allow infiltration.	Limited potential due to spatial constraints within car park and proximity to site boundary. Whilst there is some potential for use as conveyance around the site perimeter, falls across the site would severely limit potential storage capacity.

Soakaways	A soakaway is designed to allow water to quickly soak into permeable layers of soil. Constructed like a dry well, an underground pit is dug and backfilled with gravel, where water is stored and allowed to seep into the ground. Alternatively larger soakaways are more commonly constructed using geo-cellular "crates".	Discounted due to generally poor infiltration rates and high groundwater levels reported by Ground Investigations.
Ponds / Basins	Ponds can be used to store and treat water. "Wet" ponds have a constant body of water and run-off is additional, where "dry" basins are empty during periods without rainfall. Ponds & basins can be designed to allow infiltration into the ground or to store water prior to discharge.	Attenuation basin has been incorporated into the site layout to facilitate attenuation and treatment train requirements.
Wetlands	Wetlands are shallow vegetated water bodies with a varying water level. Specially selected plant species are used to filter water. Water flows horizontally and is gradually treated before being discharged.	Discounted due to large spatial requirement and limited amount of soft landscaping.
Underground Storage	Water can be stored in tanks, gravel or plastic crates beneath the ground to provide attenuation.	Suitable for site as spatial layout limits use of surface features for attenuation, Not utilised on site as full attenuation can be provided in attenuation basin.

A surface water drainage strategy drawing can be found in **Appendix J**. This shows the general approach to surface water discharge across the site, which is summarised below:

- Surface water runoff from the development is to discharge via gravity into the as-built Chamber (reference A38) formed as part of the wider development at a peak run off rate of 4.99l/s (equivalent to 1.5l/s/ha).
- Runoff from the development is conveyed through the site via conventional gravity piped networks towards the point of discharge east of the site adjacent to the development estate road.
- Attenuation is predominantly provided by pervious paving and an attenuation basin.
- Surface water falling upon the external playing surfaces (MUGA, tennis & padel courts) is to convey through the permeable surfacing to these areas and into a 300mm deep porous sub-base beneath the MUGA and padel courts and the tennis courts, where it is to be attenuated prior to discharge to the main piped network.
- Surface water runoff falling upon the spa garden, outdoor pool terrace area, and the main building roof is conveyed and attenuated in the attenuation basin located in the northeastern region of the site, prior to discharge to the wider development main piped network.
- Surface water runoff from the car park is to discharge independently of other areas and is passed through a Class 1 bypass oil interceptor, before merging with runoff from other areas in the proposed attenuation basin.

- Prior to the proposed outfall to the existing surface water drainage system serving the wider development, all runoff from the site passes through an attenuation basin providing the final treatment requirements.

The volumes of attenuation provided across the proposed development are as follows:

- Permeable sub-bases to external areas = 292m³
- Attenuation basin = 1785m³
- TOTAL VOLUME = 2077 m³

The attenuation basin has been sized to accommodate the proposed future 'drive-through site', as indicated on the surface water drainage drawings. The site area is identified as 2,600 m², inclusive of a 10% allowance for soft landscaping.

A connection spur has not been shown on the drawings at this stage, as the preferred method of connecting the future drainage to the attenuation basin has not yet been confirmed.

6.3 Proposed Discharge Rates

Surface water from the site is proposed to discharge into the surface water drainage network within the development estate road. A discharge restriction of 1.5l/s/ha has been allocated to the site that is dictated by the wider development surface water drainage strategy.

The maximum flow rate of discharge from the site limited to 1.5 l/s/ha has a calculated runoff rate of 4.99l/s.

$$\text{Max flow} = 1.5 \times 3.33 = 4.995 \text{ l/s}$$

A betterment on greenfield run off rates is provided by limiting peak rates of discharge to a value less than Qbar rate for all storms up to and including the 1 in 100-year event plus an allowance for climate change.

6.4 Climate Change

NPPF guidance stipulates that to allow for the predicted impacts of climate change on surface water runoff, increase to peak rainfall intensity should be used.

The table below shows the climate change allowances for the ‘Avon Bristol and North Somerset streams management catchment peak rainfall allowances ‘

Table 5: Climate Change Allowances

Annual Exceedance Probability Event	Allowance	Total potential change anticipated for the ‘2050s’ Development lifetime up to 2060	Total potential change anticipated for the ‘2070s’ Development lifetime 2061 to 2125
3.3%	Upper end	35%	40%
3.3%	Central	20%	25%
1%	Upper end	40%	45%
1%	Central	25%	25%

A 25% climate change allowance will be applied to the 1 in 100-year storm event as for a development with a lifetime of between 2061 and 2100 the central allowance for the 2070s epoch (2061 to 2125) is used. This is applicable as the design life of the development is 50 years.

6.5 Summary of Hydraulic Calculations

Hydraulic calculations of the proposed surface water drainage systems have been carried out using InfoDrainage software and using FEH historical rainfall data. In accordance with the NPPF and Wiltshire Council Drainage and Betterment Strategy, an allowance for climate change has been added to the historical rainfall data used in the analysis. A 25% allowance for climate change has been allowed for in the calculations for the 1 in 30-year storm events, and a 25% allowance allowed for in the calculations for the 1 in 100-year storm events. This meets the guidance on the EA website, which shows that for the Avon Bristol and North Somerset catchment, 25% central allowances for 1 in 30-year and 1 in 100-year storm events respectively are appropriate for development with an expected lifespan beyond 2070.

The calculations have also included amendments to the default values in the software for both the summer and winter Volumetric Runoff Coefficient (CV). A CV value of 0.75 for summer and 0.84 for winter scenarios have been used.

Percentage impervious values have been set at 100% for most catchment areas draining to the surface water system, except for the PIMP values assigned to the following catchments:

- Outdoor Pool Area (50%) – approximately 50% of the area is occupied by the outdoor pool. Surface water falling directly into the pool will be filtered through the pool system and not contribute to runoff.
- Green spaces (30%)

A copy of the hydraulic modelling calculations can be found in **Appendix K**. These show that the proposed networks have been designed to fully retain surface water runoff on site from storm events up to and including the 1 in 100-year (1% AEP) storm events, including the 25% climate change allowance described above.

6.6 Water Quality

The CIRIA SuDS Manual (C753) recommends a simple index approach towards surface water treatment where there is a pollution hazard associated with the land use. The building roofs and non-vehicular external areas of the development would generate low pollution hazard level indices as outlined in Table 26.2 of Chapter 26 of the SuDS Manual, which is replicated in **Table 6** below. The car park and access road would be considered as having medium pollution hazard level indices, as they would have regular trafficking by visitor vehicles.

Table 6: Pollution Hazard Indices for Different Land Use Classifications

Land Use (<i>Applicable development land uses shaded yellow</i>)	Pollution Hazard Level	Total Suspended Solids (TSS)	Metals	Hydrocarbon
Residential Roofs	Very Low	0.2	0.2	0.05
Other Roofs (typically commercial / industrial roofs)	Low	0.3	0.2	0.05
Individual Property driveways, residential car park, low traffic zones (cul-de-sacs, home zones and general access roads) and non-residential car parks with infrequent change (e.g. schools, offices) i.e. < 300 traffic movements / day.	Low	0.5	0.4	0.4
Commercial yard and delivery areas, non-residential car parking with frequent change (e.g. hospitals, retail), all roads except low traffic roads and motorways	Medium	0.7	0.6	0.7
Sites with heavy pollution (e.g. haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels are to be delivered, handled, stored, used or manufactured, industrial sites, trunk roads and motorways.	High	0.8	0.8	0.9

Table 7: Indicative SuDS Component Mitigation Indices

Type of SuDS component	Mitigation Indices		
	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Filter Strip	0.4	0.4	0.5
Filter Drain	0.4	0.4	0.4
Swale	0.5	0.6	0.6
Bioretention system	0.8	0.8	0.8
Permeable Paving	0.7	0.6	0.7
Detention Basins	0.5	0.5	0.6
Ponds	0.7	0.7	0.5
Proprietary treatment systems	These must demonstrate that they can address each of the contaminant types to acceptable levels for frequent events up to approximately the 1 in 1-year return period event, for inflow concentrations relevant to the contributing drainage area.		

The table above indicates that the attenuation basin provides sufficient treatment of runoff from the low pollution hazard index areas (roofs and external non-vehicular paved areas), but that an additional treatment stage is required to provide sufficient treatment of runoff from the car park & access road as the attenuation basin alone does not provide sufficient treatment. A petrol interceptor is therefore included as an additional treatment method for runoff from the car park immediately upstream of the attenuation basin. The combined mitigation indices are sufficient to treat runoff from the car park and access road.

6.7 Overland Flows

Exceedance flows from storm events greater than the design storm event of 1 in 100-year (1% AEP) + 25% climate change will be conveyed away from the building in the eastern region of the site towards the northeast corner, and likely flow overland on to the adjacent estate road, matching the existing scenario. Levels across the estate road are such that exceedance flows would be routed north and east along the estate road and not directed towards existing buildings.

An Overland Flow Routes plan showing the exceedance flow routes described above is included in **Appendix L**.

7 MAINTENANCE AND MANAGEMENT PLAN

The drainage systems within the site, including the various SuDS systems incorporated into the surface water strategy described in Section 6 of this report, will remain private, and therefore the responsibility for their management and maintenance will ultimately sit with David Lloyd.

To ensure that the system is regularly maintained, it is anticipated that the drainage systems will be maintained by David Lloyd personnel and/or by suitable maintenance sub-contractors who will carry out periodic inspections and any necessary maintenance and/or remedial works.

The tables below present a suitable management and maintenance regime for the various sustainable drainage system components, in line with best practice guidance set out in relevant sections of CIRIA C753, The SuDS Manual.

Table 8: Maintenance Regime for General Drainage System Components (Pipes, Manholes & Inspection Chambers)

Maintenance Schedule	Required Action	Frequency
Inspection	CCTV Survey below ground pipe networks to inspect for any defects.	Every 5-10 years.
	Inspect all inlets, outlets and overflow pipes.	Annually, or as required.
Regular Maintenance	Jet clean below ground pipe networks to remove sediment build-up.	Every 5-10 years, or as required.
Remedial Works	Carry out remedial works to rectify any defects identified from CCTV Survey, or routine inspections of inlets, outlets and overflow pipes.	As required.

Table 9: Maintenance Regime for Permeable Paving

Maintenance Schedule	Required Action	Frequency
Regular Maintenance	Brushing and vacuuming (standard cosmetic sweep over whole surface)	Once a year, after autumn leaf fall, or reduced frequency as required, based on site-specific observations of clogging or manufacturer's recommendations – pay particular attention to areas where water runs onto pervious surface from adjacent impermeable areas as this area is most likely to collect the most sediment
Occasional maintenance	Stabilise and mow contributing and adjacent areas	As required.
	Removal of weeds or management using glyphosate applied directly into	As required – once per year on less frequently used pavements

	the weeds by an applicator rather than spraying	
Remedial Actions	Remediate and 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 rutting and cracked or broken blocks considered detrimental to the structural performance or a hazard to users, and replace lost jointing material	As required
	Rehabilitation of surface and upper substructure by remedial sweeping	Every 10 to 15 years or as required (if infiltration performance is reduced due to significant clogging)
Monitoring	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 h after large storms in first six months
	Inspect silt accumulation rates and establish appropriate brushing frequencies	Annually
	Monitor inspection chambers	Annually

Table 10: Maintenance Regime for Attenuation basins

Maintenance Schedule	Required Action	Frequency
Regular Maintenance	Debris and litter removal.	Monthly
	Grass cutting – for landscaped areas / maintenance routes	Monthly (during growing season) or as required
	Remove nuisance plants	As required
	Inspect inlets, outlets and overflows for blockages, and clear if required.	Monthly
	Inspect banksides, structures, pipework etc for evidence of physical damage	Monthly
	Inspect inlets and facility surface for silt accumulation. Establish appropriate silt removal frequencies.	Monthly
	Check flow control device	Annually
	Tidy all dead growth before start of growing season	Annually
	Remove sediment from inlets & outlet	Annually (or as required)
Occasional Maintenance	Re-seed areas that grow poorly	As required
	Prune and trim any trees and remove cuttings	Every 2 years, or as required
	Remove sediment from inlets, outlets and main basin when required	Every 5 years, or as required
Remedial actions	Repair erosion or other damage by reseedling or re-turfing	As required
	Repair/rehabilitation of inlets, outlets and overflows	As required
	Relevel uneven surfaces and reinstate design levels	As required

8 FOUL DRAINAGE STRATEGY

8.1 Existing Foul Drainage

As noted in Section 2.5 of this report, it is understood that there are no public sewers within the site boundary.

An as built of the newly built foul water network has been provided by Stantec and is included in **Appendix E**. Stantec have provided a foul spur into the proposed David Lloyd site along the access road that is shown to discharge north along the development estate road to a new sewage pumping station.

This infrastructure is understood to have been constructed to adoptable standards in consultation with Wessex Water.

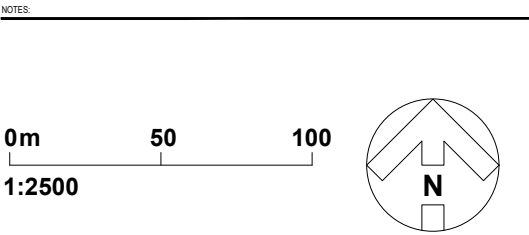
8.2 Proposed Foul Drainage Strategy

A foul water drainage strategy drawing can be found in **Appendix M**.

Peak foul drainage flows have been estimated based on records of water usage from several existing David Lloyd clubs of a comparable size to the club proposed here. This historical data has been provided in terms of water usage per 30-minute period over an entire calendar year. Based on this data, during peak periods, the flow rate equates to around 3.0l/s, and the mean flow rate during opening hours equates to around 1.8l/s. A summary of this historical flow data has been provided in **Appendix N**. As an additional Factor of Safety, an additional 25% has been added to the calculated 3.0l/s peak discharge rate, thereby giving a peak foul flow rate of 3.75l/s, which is what has been used in discussions with the vendor and Wessex water.

Stantec who is responsible for the wider development site has been liaising with Wessex Water regarding the proposed foul discharges and has confirmed that there is capacity in the receiving network.

APPENDIX A – SITE LOCATION PLAN



Application Boundary

REVISION: P01 BY: HP CHECKED: MPB DATE: 12/02/2026
Planning Issue.

STATUS

S4a | FOR PLANNING

CLIENT

David Lloyd
CLUBS

PROJECT

CHIPPENHAM
SOUTHPOINT BUSINESS PARK

TITLE

LOCATION PLAN

Hadfield Cawkwell Davidson

Broomgrove Lodge, 13 Broomgrove Rd, Sheffield, S10 2LZ T 0114 266 8181 www.hcd.co.uk

HCD PROJECT NO.	SCALE	REV.
24245	1:2500 @ A3	P01

PROJECT NO.	ORIGINATOR	VOLUME	LEVEL	TYPE	ROLE + NUMBER
24245	HCD	ZZ	00	DR	A-PL-001

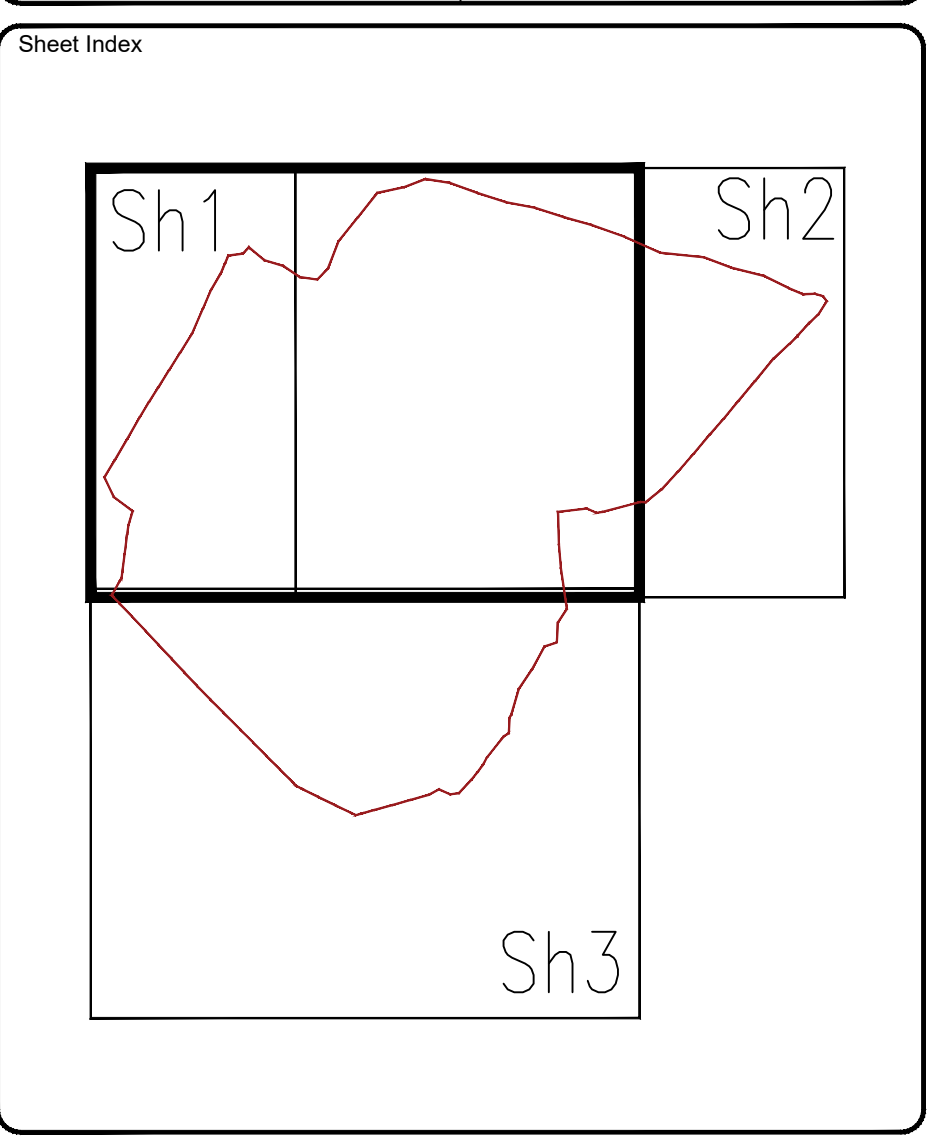
S:\Architecture\24245\Drawings\DR - Plotsheets\Planning\A-PL-001 - Location Plan.dwg | © HCD

APPENDIX B – TOPOGRAPHICAL SURVEY



Survey Notes	
Grid:	Local, plane metric connected to OS Grid
Levels:	OS Datum from GNSS positioning, connected using the National Geoid Model OSGM15.

Notes

[illegible]

Rev.	Date	Description
A	06/06/25	Original Issue



**MALCOLM
HUGHES**

**CHARTERED
LAND SURVEYORS**

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Accreditations








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Surrey, KT21 2BT

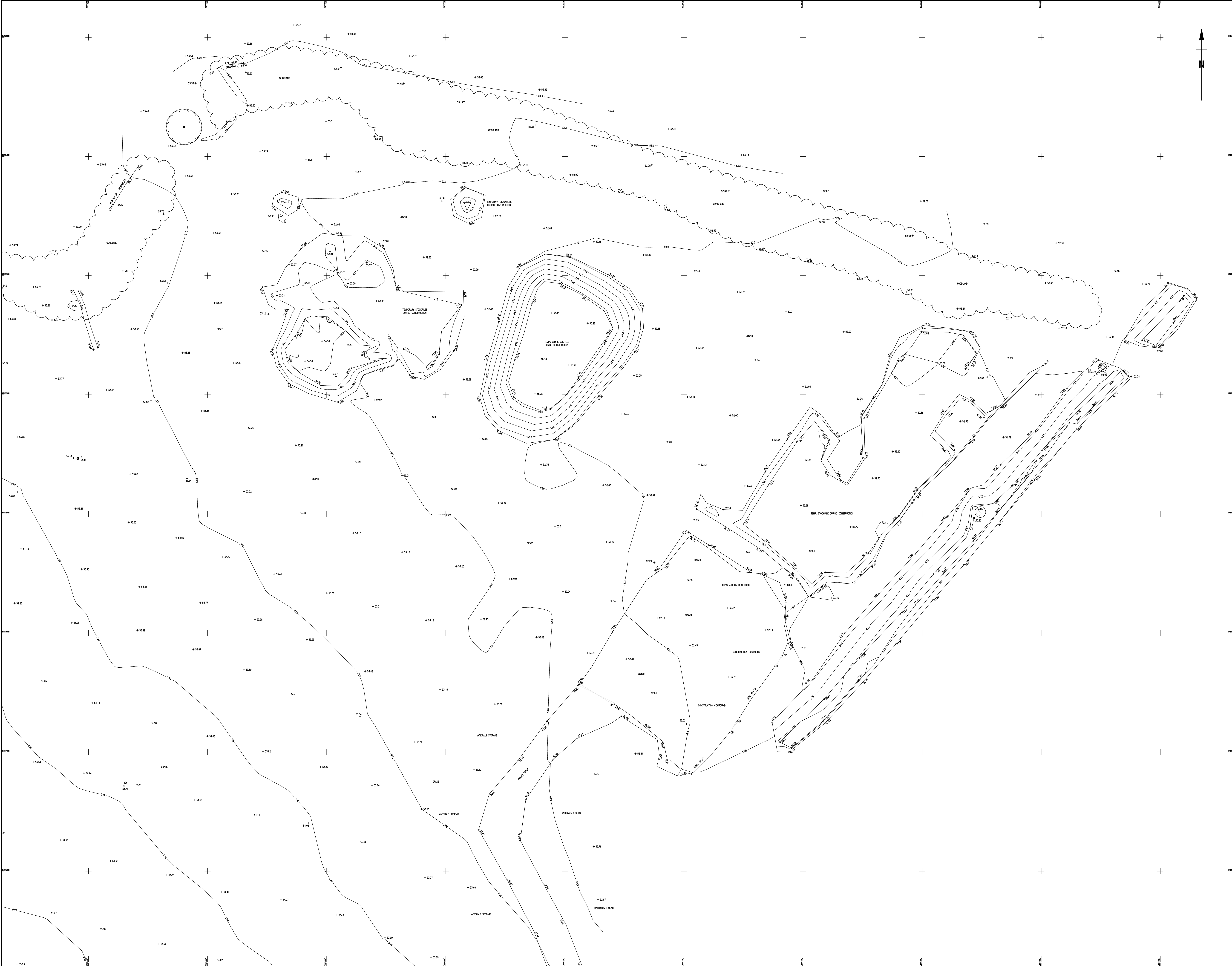
Project

David Lloyd

Chippenham South Point

Drawing Title	Topographical Survey
	Sheet 1 of 3

Drawn By	AR	06/06/25	Survey Date	May 2025
Checked by	EAD	06/06/25	Scale	1:200@A0
Drawing No 63009-MH-VTO-Chippenham_South-M2-G-0001				



Survey Notes
Grid: Local plane metric connected to OS Grid
Levels: OS Datum from OSGB3 positioning, connected using the National Grid Model OSGM15.

Notes

Topographical Survey Legend

BUILDINGS AND WALLS

Building

Building Base

Run

Passage

Wall with Height

Retaining Wall

1/2" = 1' 0"

1/4" = 1' 0"

1/8" = 1' 0"

1/16" = 1' 0"

1/32" = 1' 0"

1/64" = 1' 0"

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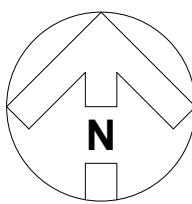
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APPENDIX C – EXISTING SITE PLAN



NOTES

0m 10 20
1:500



Application Boundary
3.33 hectares / 8.23 acres

REVISION: P01 BY: HP CHECKED: MPB DATE: 12/02/2026
Planning Issue.

STATUS
S4a | FOR PLANNING

CLIENT
David Lloyd
CLUBS

PROJECT
CHIPPENHAM
SOUTHPOINT BUSINESS PARK

TITLE
EXISTING
SITE PLAN

Hadfield Cawkwell Davidson

Broomgrove Lodge, 13 Broomgrove Rd, Sheffield, S10 2LZ T 0114 266 8181 www.hcd.co.uk

HCD PROJECT NO. 24245	SCALE 1:500 @ A1	REV P01
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PROJECT NO. 24245	ORIGINATOR HCD	VOLUME ZZ	LEVEL 00	TYPE DR	ROLE - NUMBER A-PL-002
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S:\Architecture\24245\Drawings\DR - Plotsheets\Planning\A-PL-002 - Existing Site Plan.dwg | © HCD

APPENDIX D – PHASE II GI REPORT EXTRACTS

4.0 GROUND CONDITIONS

4.1 Published Geology

- 4.1.1 The British Geological Survey shows the site to be underlain by the geological succession outlined below Table 4.1: Published Geology.

Table 4.1: Published Geology

Geology	Description /strata	BGS General Description (Lexicon)
Artificial	None recorded	NA
Superficial	None recorded	NA
Bedrock	Cornbrash Formation	Limestone, medium- to fine-grained, predominantly bioclastic wackestone and packstone with sporadic peloids; generally and characteristically intensely bioturbated and consequently poorly bedded, although better bedded, commonly somewhat arenaceous units occur in places, particularly in the upper part. Generally bluish grey when fresh, but weathers to olive or yellowish brown. Thin argillaceous partings or interbeds of calcareous mudstone may occur

4.2 General

- 4.2.1 A generalised summary of the ground conditions encountered, depths to base, and thickness recorded from the exploratory hole records is presented below in Table 4.2: Summary of Ground Conditions.

Table 4.2: Summary of Ground Conditions

Geology / Strata	Depth (mbgl)				Thickness (m)		Locations encountered
	Top		Base		Min	Max	
	Min	Max	Min	Max			
Topsoil	0.00	-	0.10	0.30	0.10	0.30	TP01-TP10, DS06-DS09
Cornbrash Formation	0.10	0.30	0.50	3.50	0.50	3.20	All locations
Groundwater	Strike depth (mbgl)		Rest Level (mbgl)		Comments		Locations encountered
	Min	Max	Min	Max			
Strike	-	-	-	-	NA		Not encountered
SWL at end of borehole	-	-	-	-	NA		Not encountered
Seepage	1.80	2.30	1.80	2.30	NA		TP02, TP04, TP06, TP07, TP08, TP09, TP10.

- 4.2.2 The general ground conditions encountered have been summarised below.
- 4.2.3 Detailed strata descriptions are presented on the dynamic sample borehole logs, and trial pit logs presented in Appendix B and C respectively.
- 4.2.4 The actual ground conditions encountered across the site were generally uniform and comprised topsoil and the Cornbrash Formation.
- 4.2.5 No Made Ground or artificial deposits were encountered or observed during this ground investigation.

4.3 Topsoil

- 4.3.1 Topsoil was encountered in TP01-TP10, and DS06-DS09 0.from ground level to depths of between 0.10 and 0.30mbgl and generally comprised slightly clayey gravelly sand with frequent rootlets.

4.4 Superficial Deposits

- 4.4.1 No superficial deposits were encountered during this ground investigation.

4.5 Bedrock

- 4.5.1 Soils considered to represent the Cornbrash Formation Limestone were encountered in all exploratory hole locations directly beneath the Topsoil with the exception of DS01-DS05 where the topsoil had been scrapped back prior to advancing the boreholes.
- 4.5.2 The Cornbrash Formation was proven to depths of between 0.50 and 3.50mbgl.
- 4.5.3 The Cornbrash Formation was initially recovered in a highly weathered state and generally comprised very weak, limestone recovered as gravelly sand to depths of around 0.69 and 2.10mbgl; beyond this depth the Cornbrash Formation was recorded as a very stiff blueish grey clay.

4.6 Groundwater

- 4.6.1 Groundwater was encountered in TP02, TP04, TP06, TP07, TP08, TP09 and TP10. Further details are as follows:-

- TP02 seepage at 1.80mbgl.
- TP04 seepage at 1.80mbgl.
- TP06 seepage at 2.00mbgl.
- TP07 seepage at 1.90mbgl.
- TP08 seepage at 2.30mbgl.
- TP09 seepage at 2.00mbgl.
- TP10 seepage at 1.80mbgl.

4.7 Groundwater Monitoring

- 4.7.1 A groundwater monitoring regime was carried out at the site to establish the ground water levels beneath the site, which included monitoring on three occasions. Table 4.3: Groundwater Monitoring Results, below presents the details of the groundwater monitoring regime. All boreholes were purged and developed accordingly to prior to the groundwater monitoring.

Table 4.3: Groundwater Monitoring Results

Exploratory Hole ID	Base of Hole (mbgl)	Standing Water Level (SWL) (mbgl)		
		Date of Visit		
		11/07/2025	01/08/2025	18/08/2025
DS01	0.90	Dry	Dry	Dry
DS02	1.17	Dry	Dry	Dry
DS04	1.25	Dry	Dry	Dry
Old Location	11.60	1.55	1.50	1.60

N/R = Not recorded

9.0 SOAKAWAY TESTING AND RESULTS

9.1 General

9.1.1 Soakaway testing was carried out at three locations (TP01/SA01, TP03/SA02, TP05/SA03) across the site to provide infiltration rates where possible and to determine whether a sustainable drainage system is suitable for use at this site to deal with surface water runoff.

9.1.2 The soakaway investigation has been carried out in accordance with BRE 365.

9.1.3 In accordance with BRE Digest 365 entitled 'Soakaway design', soakaway test pits should be constructed so that they represent a section of the proposed soakaway.

9.1.4 As specified within BRE Digest 365, three soakaway tests were undertaken within the test pit.

9.1.5 No natural groundwater was encountered in the soakaway test pits prior to carrying out the test.

9.2 Soakaway Test Pits

9.2.1 The soakaway pit dimensions are presented below in Table 9.1: Soakaway Pit Dimensions:

Table 9.1: Soakaway Pit Dimensions

TP / SA ID	Width (m)	Length (m)	Depth (m)
TP01/SA01	0.60	1.50	2.10
TP03/SA02	0.60	1.50	1.80
TP05/SA03	0.60	1.50	1.80

9.2.2 The trial pit / soakaway logs are presented in Appendix C.

9.2.3 The soakaway trial pits remained stables once excavated, therefore they were left open following filling and to allow the free drainage of water from the pits once filled.

9.3 Filling

9.3.1 Each trial pit was excavated using a JCB 3CX and was filled via a water tanker; water was pumped rapidly into the pit in one continuous motion. The water level monitoring commenced immediately after the pit had been filled.

9.3.2 A slotted pipe was placed in the centre of the pit to allow water level measurements to be taken at regular timed intervals.

9.3.3 Standard practice as recommended in BRE 365 is to fill each soakaway pit and allow to drain completely three times, however based on the underlying soils beneath the site, the soakaway pits showed no signs of draining after eight hours, therefore the tests were terminated.

9.4 Soakaway Testing

9.4.1 It should be noted that the water level recorded at the start of the tests has been taken to be the proposed drain invert level.

9.4.2 The soakaway test data obtained (tabulated and graphed) is presented in Appendix E.

9.5 Backfill

9.5.1 On completion, the soakaway pits were backfilled with airings to a similar standard as that prior to the works and made safe and level at the surface.

9.6 Soakaway Test Results

- 9.6.1 The results of the soakaway testing have been used to calculate the soakaway parameters, and are presented below in accordance with the methodology set out within BRE Digest 365, values of a_{p50} and V_{p75-25} were calculated, and values of t_{p75-25} were estimated from the graphs in Appendix C. Soil infiltration rates were then determined for each test. It should be noted that the water level recorded at the start of each test has been taken to be the proposed drain invert level.
- 9.6.2 The results of the soakaway testing have been used to calculate the soakaway parameters, and are presented below in Table 9.2: Soakaway Parameters.
- 9.6.3 Table 9.2: Soakaway Parameters In accordance with the methodology set out within BRE Digest 365, values of a_{p50} and V_{p75-25} were calculated, and values of t_{p75-25} were estimated from the graphs in Appendix C. Soil infiltration rates were then determined for each test. It should be noted that the water level recorded at the start of each test has been taken to be the proposed drain invert level.
- 9.6.4 The results of the soakaway testing have been used to calculate the soakaway parameters, and are presented below in Table 9.2: Soakaway Parameters.

Table 9.2: Soakaway Parameters

Parameter	Soakaway Test Location TP01/SA01		
	Test 1	Test 2	Test 3
Assumed Drain Invert Level (mbgl)	1.02	-	-
Total Effective Storage Depth (m)	1.98	-	-
75% Effective Storage Depth (mbgl)	0.99	-	-
25% Effective Storage Depth (mbgl)	1.52	-	-
Volume (m3), $V_{p75} - V_{p25}$	0.89	-	-
Area (m2), a_{p50}	5.06	-	-
Time (minutes), t_{p75-25}	0.00	-	-
Soil infiltration rate (m/s), f	Unsuitable	-	-
Parameter	Soakaway Test Location TP03/SA02		
	Test 1	Test 2	Test 3
Assumed Drain Invert Level (mbgl)	1.80	-	-
Total Effective Storage Depth (m)	1.70	-	-
75% Effective Storage Depth (mbgl)	3.08	-	-
25% Effective Storage Depth (mbgl)	2.23	-	-
Volume (m3), $V_{p75} - V_{p25}$	0.77	-	-
Area (m2), a_{p50}	4.47	-	-
Time (minutes), t_{p75-25}	0.00	-	-
Soil infiltration rate (m/s), f	Unsuitable	-	-
Parameter	Soakaway Test Location TP05/SA03		
	Test 1	Test 2	Test 3
Assumed Drain Invert Level (mbgl)	1.13	-	-
Total Effective Storage Depth (m)	2.37	-	-
75% Effective Storage Depth (mbgl)	2.91	-	-
25% Effective Storage Depth (mbgl)	1.72	-	-
Volume (m3), $V_{p75} - V_{p25}$	1.07	-	-
Area (m2), a_{p50}	5.88	-	-

Time (minutes), t_{p75-25}	0.00	-	-
Soil infiltration rate (m/s), f	Unsuitable	-	-

9.7 Soakaway Conclusions and Recommendations

General

- 9.7.1 Soakaway test pits should be constructed so that they represent a section of the proposed soakaway. BRE Digest 365 states that natural groundwater should not rise to the level of the base of the soakaway during annual variations in the water table. Local building control and / or planning authorities should advise where fluctuations in groundwater level may cause a problem in the long term for any proposed depth of excavation.

Soakaway Results

- 9.7.2 The soakaway test data obtained (tabulated and graphed) is presented in Appendix E.
- 9.7.3 Soakaway tests were allowed to drain over an eight-hour period and showed no signs of draining. All locations encountered natural groundwater at around 3.00mbgl which then caused the water added for the test to rise, which is demonstrated on the graphs. The 25% effective storage depth was therefore not achieved during the 8 hour working shift, and therefore it is not possible to calculate a soil infiltration rate.

Conclusions

- 9.7.4 Based on the results of the soakaway test carried out at the site, the site is therefore considered unsuitable for a Sustainable Drainage System.
- 9.7.5 A soil infiltration rate cannot be determined due to the soakaway test not draining to at least 75% effective storage depth within a 24 hour period.

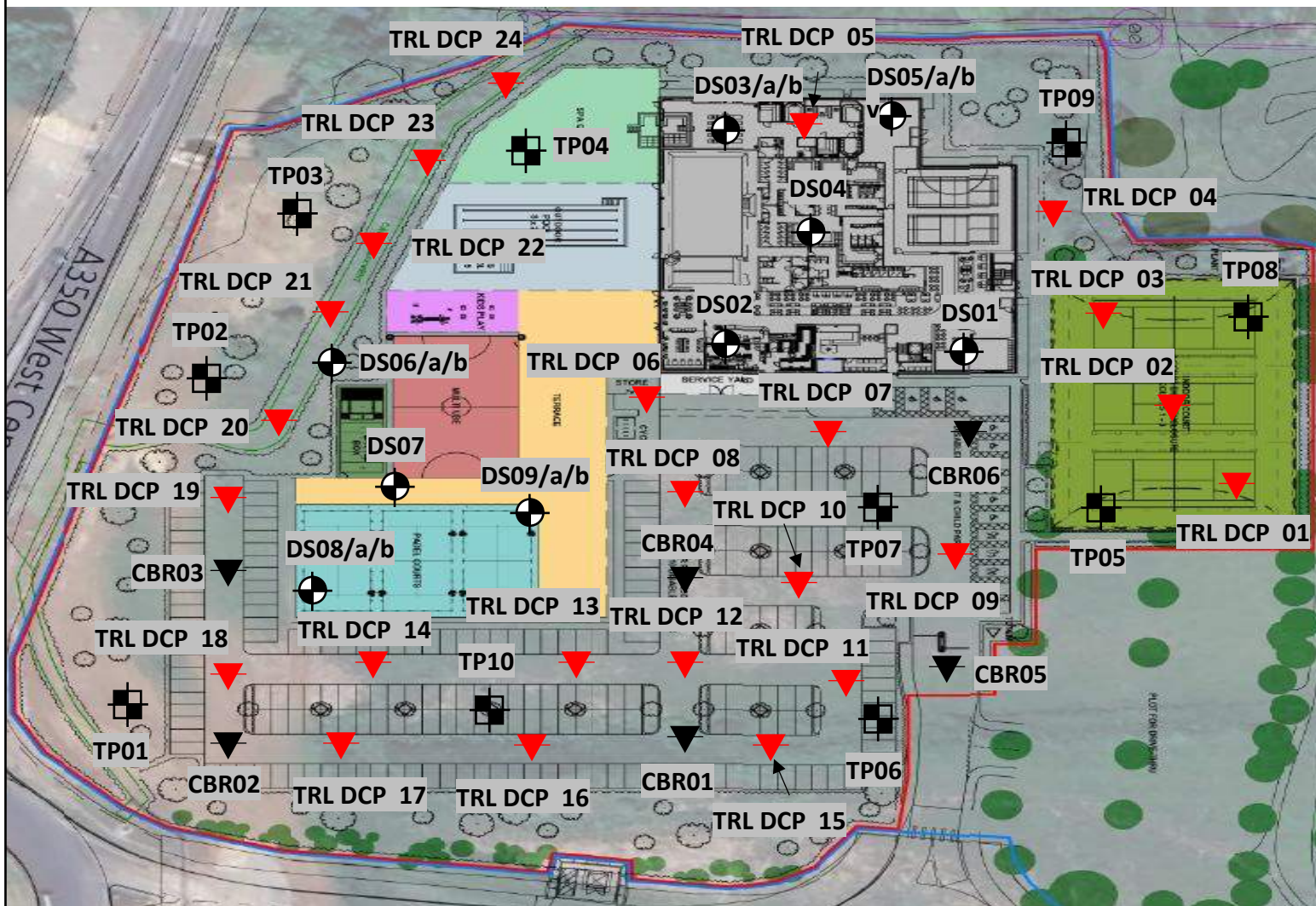
Recommendations

- 9.7.6 It is therefore recommended that the following options would be available for the site:
- The ground is engineered to accommodate a below ground water storage area. This would need to be designed to accommodate sufficient volumes and account for critical storm duration.
 - Contact the local water provider to discuss connecting to their infrastructure.
 - Consider alternative options.



Legend:

-  Dynamic Sample Borehole Location
-  Trial Pit Location
-  TRL DCP Location
-  PLT Location



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PROJECT ID

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TITLE

Exploratory Hole Location Plan

DRAWING NUMBER

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SCALE

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DATE

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