Tidcombe Hall, Tiverton

Tidcombe Holdings LVA LLP

Flood Risk Assessment

Tidcombe Hall, Tiverton

Flood Risk Assessment

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Contents

Appendices

1 Introduction

1.1 Awcock Ward Partnership (AWP) have been commissioned by Tidcombe Holdings LVA LLP to prepare a Flood Risk Assessment (FRA) in support of new development at Tidcombe Hall, Tiverton.

1.2 The proposed development comprises up to 100 residential dwellings including: the conversion of Tidcombe Hall and its outbuildings into up to 17 dwellings, community allotments, community orchard, associated access, garaging, parking, landscaping, drainage and engineering works.

1.3 The location of the proposed development is shown on Figure 1.1.

Figure 1.1 - Site Location – Wide Area

1.4 The proposed development site is, in part, allocated as a contingency site within the adopted Mid Devon Local Plan as AL/TIV/21. Within the local plan review the site has again been allocated as a contingency site (TIV/13).

National Planning Policy Framework

1.5 The National Planning Policy Framework (NPPF) and the accompanying Flood Risk and Coastal Change section of the Planning Practice Guidance (PPG) was updated most recently published by the Department for Communities and Local Government in September 2023 and June 2021 respectively.

1.6 The NPPF states that "*A site-specific flood risk assessment should be provided for all development in Flood Zones 2 and 3. In Flood Zone 1, an assessment should accompany all proposals involving: sites of 1 hectare or more; land which has been identified by the Environment Agency as having critical drainage problems; land identified in a strategic flood risk assessment as being at increased flood risk in future; or land that may be subject to other sources of flooding, where its development would introduce a more vulnerable use*".

1.7 The aim of a site-specific flood risk assessment is to demonstrate that "*the development should be made safe for its lifetime without increasing flood risk elsewhere*".

Consultation

1.8 To scope out any site specific or catchment specific flood risk or drainage requirements, we have engaged with various parties.

1.9 We have liaised with Steve Densham of Mid Devon District Council (MDDC), Devon County Council (DCC)'s Flood Risk and Drainage Team, as Lead Local Flood Authority (LLFA), and with South West Water, as the appropriate Water Company for this catchment.

1.10 We have also liaised with Mark Baker, Grand Western Canal manager, to evaluate any constraints or opportunities arising from the downstream culvert which passes beneath the Grand Western Canal.

1.11 The output of the above consultation process has helped to inform this FRA and the inherent SWMP.

Reference

1.12 This FRA has been prepared by reference to the following documents:

- National Planning Policy Framework (September 2023);
- Planning Practice Guidance – Flood Risk and Coastal Change (August 2022)
- Environment Agency (EA) Flood Warning Information Service 'Flood Risk from Rivers or the Sea' and 'Flood Risk from Surface Water' (online);
- Ruddlesden Geotechnical Ltd Ground Investigation & Contamination Report (October 2018)
- CIRIA Guide 753 – The SuDS Manual (November 2015); and,
- South West Water's (SWW) Internet Mapping (online).

2 Existing Conditions

Context

2.1 The proposed site is located on the eastern edge of Tiverton, south of the Grand Western Canal and adjacent existing arable land. The location of the proposed site in relation to its surroundings is shown on Figure 2.1:

Figure 2.1 - Site Location – Local Area

Existing land uses

2.2 The proposed development site comprises the existing Tidcombe Hall (residential use) at its western extent, with undeveloped greenfield land elsewhere.

2.3 The eastern extents of the site bound ‘Little Tidcombe’ (residential use), with rights of access retained through the application site.

Surrounding land use

2.4 The site bounds ‘Little Tidcombe’ and is bordered to the south by existing cottages at Warnicombe Lane. The northern edge of the site lies adjacent the offside buffer for the Grand Western Canal, whilst the eastern and western edges of site are bound by greenfield land and Tidcombe Lane respectively.

Topographic survey

2.5 A topographic survey has been undertaken and indicates that the site falls in a northerly direction, towards the Grand Western Canal, from a high point of 116.65mAOD, to a low point of 92.69mAOD.

2.6 An 'Existing Site Plan' has been prepared to set the context of the pre-development site and can be found as drawing 0759-XS-101, within Appendix A of this report.

Existing Flood Risk

2.7 The EA's 'Flood Warning Information Service' provides flood risk information and mapping throughout England.

2.8 An extract of the 'Flood Risk from Rivers or the Sea' mapping has been reproduced as Figure 2.2 and shows the site to be within 'Flood Zone 1', as land assessed as having a less than 1 in 1,000 annual probability of river or sea flooding (<0.1%).

Figure 2.2 – Flood Risk from Rivers or the Sea

2.9 An extract of the 'Flood Risk from Surface Water' mapping has been reproduced as Figure 2.3. This mapping is based on LIDAR data and indicates the typical conveyance routes of surface water runoff.

Figure 2.3 – Flood Risk from Surface Water

2.10 The 'flooding from surface water' map suggests the only concentrations of surface water passing through the application site are deemed 'low risk', these having between 0.1% and 1% annual probability.

2.11 The only area at medium or high risk of flooding from surface water is a localised area adjacent the northern boundary. This is where rural runoff will pond on-site, alongside the offside buffer strip for the Grand Western Canal.

2.12 It is considered that the development of this site will provide a managed surface water regime, with a significant reduction in rural overland flow, and will therefore mitigate this risk.

Existing Drainage Infrastructure

2.13 SWW's asset records have been transposed onto the Existing Utilities drawing (ref. 0759-UD-101) within Appendix B of this report.

2.14 The records confirm there are no public sewer networks within the site. The nearest public sewers serve existing residential developments to the west of Tidcombe Lane.

2.15 An existing water distribution main is located within Tidcombe Lane to the west and Warnicombe Lane to the south.

Existing Site Drainage

2.16 Following an existing site walk-over we can confirm that runoff generated by Tidcombe Hall is conveyed through a series of private drainage networks, with two drainage outfalls beyond the boundary wall of the property. Two separate ditches convey flows from the outfalls towards an existing culvert which backdrops beneath the Grand Western Canal (DCC ref. GWC Culvert 3).

2.17 The existing drainage regime for the remainder of the site represents that of a typical greenfield site, with some surface water runoff soaking into the underlying strata and the remainder following the natural topography of the site, towards the northern boundary.

2.18 An existing ditch lies within the offside buffer of the Grand Western Canal and naturally intercepts runoff from the greenfield site. The ditch backdrops beneath the Grand Western Canal (DCC ref. GWC Culvert 4). The culvert emerges as an ordinary watercourse to the east of Rippon Close and Westcott Road.

2.19 The ordinary watercourse continues north before turning west adjacent the Alsa Brook and flows through the Tidcombe Lane Fen Site of Special Scientific Interest (SSSI). The drainage route from site to the SSSI can be seen identified on Figure 2.4 and measures approximately 700m.

Figure 2.4 – Off-site Drainage Route

2.20 The proposed location of the development lies within the Impact Risk Zone/Catchment Risk Zone for the SSSI and therefore any development at this site must provide suitable mitigation to avoid any adverse impacts within the designated site.

Ground Conditions

2.21 A 'Ground Investigation & Contamination Assessment Report' (GICAR) was completed by Ruddlesden Geotechnical Ltd in October 2018 and included soakaway testing in accordance with BRE Digest 365.

2.22 The output of the testing confirms that infiltration does not present a viable method of surface water disposal for this site. Instead, an attenuated discharge should be considered.

2.23 The investigation identified groundwater levels across. These levels must be considered within any future drainage design to ensure that groundwater ingress does not impact any open SuDS features.

2.24 Relevant extracts from the GICAR are included within Appendix C.

3 Development Proposals

Introduction

3.1 The proposed development comprises of upto 100 residential dwellings including: the conversion of Tidcombe Hall and its outbuildings into up to 17 dwellings, community allotments, community orchard, associated access, garaging, parking, landscaping, drainage and engineering works.

3.2 A copy of the illustrative masterplan for the scheme can be found within Appendix D of this report.

Vulnerability

3.3 In accordance with the Planning Practice Guidance, residential dwellings are considered to be "More Vulnerable". However, given the entire site is located within 'Flood Zone 1', Table 3 of the Planning Practice Guidance confirms this as being an appropriate form of development at this site.

Sequential Test

3.4 The site is located within 'Flood Zone 1' and therefore passes the Sequential Test, as there are no competing sites with a lower flood risk classification.

Cross sections and finished levels

3.5 It is anticipated that the existing ground profile will be modified locally to reflect the requirements of the new development.

3.6 Any future level design should aim to minimise the extent of any re-profiling works and should look to retain existing catchment areas wherever possible.

Safe access and egress

3.7 The full extents of the site and all roads surrounding the site are within 'Flood Zone 1' and hence access and egress for motorised and non-motorised vehicles will not be affected during flood events.

Drainage strategy requirements

3.8 'CIRIA C753 – The SuDS Manual' advises that surface water disposal should be prioritised in the following order:

1. Infiltration
2. Discharge to surface waters
3. Discharge to a surface water drainage system
4. Discharge to a combined sewer

3.9 Site-specific soakaway testing has been undertaken in accordance with BRE Digest 365 and confirmed that infiltration is not a viable method of surface water disposal. Instead the surface water management plan should seek to utilise an attenuated discharge to surface water.

3.10 As required by the NPPF, the drainage strategy must demonstrate that the development will be safe throughout its lifetime, without increasing flood risk elsewhere, whilst also taking account of the impacts of climate change.

Climate change impacts

3.11 The NPPF requires that the impact of climate change be considered to minimise vulnerability and provide resilience. The NPPF and Planning Practice Guidance explain that an FRA should demonstrate how flood risk will be managed across the development's lifetime, taking climate change into account.

3.12 Climate change allowances for peak rainfall in England is published online by the Department for Environment, Flood and Rural Affairs. The 'East Devon Management Catchment peak rainfall allowances are summarised in Table 3.1. The climate change recommendations provide for developments with a lifetime up to 2125 (epoch 2070s).

Table 3.1 – East Devon Management Catchment peak rainfall intensity allowances

Table 3.1 – East Devon Management Catchment peak rainfall intensity allowances

Allowance category	2050s epoch (lifetime up to 2060)	2070s epoch (lifetime 2060 to 2125)
Upper end (90th Percentile)	30%	45%
Central (50th Percentile)	25%	40%

3.13 The NPPF guidance states for peak rainfall intensity, Flood Risk Assessments should "assess both the central and upper end allowances to understand the range of impact".

3.14 The on-site attenuation for this proposed development has been sized to offer flood protection for the development and its downstream catchment throughout its lifetime, with the upper end allowance of 45% being utilised to present a worst-case scenario.

4 Surface Water Management Plan

Existing surface water runoff

4.1 The existing site is composed of two distinct catchments.

- Catchment A – located in the north-west of the site, is partially brownfield in character comprised of Tidcombe Hall and its associated access, outbuildings and immediately adjacent green space.
- Catchment B – Comprising the southern extents of development, predominantly greenfield in nature. This catchment also includes the area to be developed as plots 14-17.

4.2 Existing run-off from Catchment A is collected by existing drainage systems and discharged to the north via either the existing pond or watercourse, to an existing culvert (Culvert 3) that discharges north under the Great Western Canal. Beyond the capacity of the existing drainage systems stormwater would flow north above ground along existing overland flood flow routes before entering the same pond or watercourse to be discharged to the same receiving watercourse via Culvert 3.

4.3 Existing run-off from Catchment B is that of a typical greenfield site, with rainfall infiltrate according to permeability of the soils and sub-soils And the residual continuing overland as run-off. Runoff will flow according to the site topography, heading north towards the cut-off ditches along the southern bank of the Great Western Canal. The cut-off ditches drain via Culvert 4, which routes beneath the Canal to the receiving watercourse to the north.

4.4 Causeway's FLOW has been used to assess the greenfield runoff rates for both catchments of the existing site using the FEH statistical methodology, which is based on the proposed impermeable area only as required by DCC. This method complies with the interim guidance resulting from the Environment Agency "Estimating flood peaks and hydrographs for small catchments: Phase 1 Project SC090031". This study reviewed current methods for rainfall-runoff estimation in small catchments and concludes; "*Flood estimates on small catchments should be derived from FEH or the Revitalised Flood Hydrograph (ReFH) rainfall-runoff model, except on: highly permeable catchments (BFIHOST>0.65), where ReFH should be* *avoided. On urban catchments (URBEXT2000>0.15), where the results of the ReFH model can be less reliable."*

4.5 A copy of the greenfield runoff assessment has been included within Appendix E of this report, with the results summarised in Table 4.1 below:

Table 4.1 – Equivalent Greenfield Runoff Rates

Return Period	Greenfield runoff rate 1.685ha (l/s)
2 years	4.8
30 years	10.6
100 years	13.5

4.6 To ensure the development will be safe throughout its lifetime and that it does not increase flood risk elsewhere, the drainage strategy will include appropriate mitigation measures, so that the equivalent greenfield runoff rates are not increased. This will offer significant betterment compared to existing developed areas of the site which do not utilise an attenuated discharge.

Proposed Surface Water Strategy

4.7 The SWMP has been developed in accordance with DCC's 'SuDS Guidance for Devon' to assess how surface water runoff can be safely managed.

4.8 To ensure the development is safe throughout its lifetime, the surface water strategy accounts for runoff in up to the 100 year return period.

4.9 The strategy safeguards against the upper end allowances for climate change (45%), which provides betterment over the existing undeveloped site where greenfield runoff would continue to increase as climate change occurs.

4.10 The strategy also provides an additional 10% allowance for urban creep applied to domestic poperties. The allowance for urban creep provides for future growth in impermeable area that homeowners make under any permitted development rights. The inclusion of urban creep provides a reduction in flood risk until the 10% allowance is realised.

4.11 Site specific soakaway testing has been undertaken in accordance with BRE Digest 365 and confirmed that infiltration is not a viable method of surface water disposal. Instead the surface water management plan should seek to utilise an attenuated discharge to surface water.

4.12 Hydraulic controls will be utilised to restrict the peak rates of discharge to greenfield rates and at the detailed design stage will seek to provide a further 10% reduction to generate additional capacity within the downstream culverts.

4.13 The hydraulic controls will also ensure that Long-Term Storage is being mobilised and discharged at no greater than 2 l/s/ha to mitigate the impacts of any increased volume of runoff.

4.14 Runoff generated by the re-development of Tidcombe Hall and associated outbuildings (Catchment A) will be conveyed to a new detention basin (Basin 1). Connectivity varies for different parts of Catchment A, but is achieved through a mix of pipes, swales and a new raingarden.

4.15 Runoff generated by the remainder of the development will drain through adopted sewers to a conveyance swale situated along the eastern site boundary. The swale will use online check-dams and a raingarden for the treatment and mobilisation of surface runoff and to promote sedimentation. From the raingarden, in common with catchment A, surface water from Catchment B will discharge to Basin 1.

4.16 The outflow from Basin 1 is directed towards a new attenuation pond (Pond 1) which provides additional attenuation storage, whilst also improviding water quality amenity and biodiversity.

4.17 Basin 1 will include a sedimentary forebay and both Basin 1 and Pond 1 will be sized to accommodate runoff in up to the 100 year +45%CC storm with 300mm freeboard.

4.18 The depth of all new SuDS should be set above the level of groundwater encountered during the site investigation or otherwise must include measures to prevent groundwater ingress.

4.19 The pond will discharge to the existing ditch located within the offside buffer of the Grand Western Canal.

4.20 The receiving ditch drains to GWC Culvert 4. The proposed flow restrictions and surface water treatment (various stages of silt control) were agreed through consultation with Mark Baker, Canal Manager, to actively reduce the adverse impacts of silt accumulation within the existing culverts.

4.21 To facilitate future maintenance of GWC Culvert 4, it is agreeable that the existing access gates to the offside buffer strip will remain accessible and that an informal area can be made available to enable temporary bunding for culvert de-silting purposes. The temporary bund could form a permanent landscaped feature to minimise (infrequent) disruption of the public open space. Photo 4.1 identifies previous bunding carried out on site.

Photo 4.1 – Maintenance of the culvert beneath the canal

4.22 The proposed SWMP not only promotes best-practice SuDS, providing multiple stages of treatment (3 of 4 SuDS features), but also mitigates any increased rate or volume of runoff and protects against the impacts of climate change, therefore providing a beneficial impact to downstream receptors, including the Tidcombe Fenn SSSI.

4.23 The drawing included within Appendix F (reference 0759-PDL-101) shows a preliminary drainage layout for the site.

Water Quality Management

4.24 Runoff generated by the development will pass through a best practice SuDS train, which includes swales, a raingarden, a detention basin with sediment forebay and a new attenuation pond.

4.25 Consideration should be given at detailed design stage to the inclusion of further SuDS features including under-drained permeable pavements, tree-pits and raingardens. The various stages of treatment will offer filtration of runoff and sedimentation of any suspended solids.

4.26 In line with the pollution indices set out in CIRIA SuDS Manual (C753) the development is required to provide 'SuDS component features' which have a 'total SuDS mitigation index' greater than or equal to the 'pollution hazard index'.

4.27 Table 4.2 outlines the pollution hazard indices required for residential developments, taken from Table 26.2 of C753.

Table 4.2 – Pollution Hazard Indices (Ref. C753 Table 26.2)

Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Low	**0.5**	**0.4**	**0.4**

4.28 The SuDS components proposed for the site and their relevant 'mitigation indices' are identified within Tables 4.3 and 4.4, with a factor of 0.5 to account for reduced performance associated with any secondary SuDS components.

Table 4.3 – SuDS Mitigation Indices Catchment A

SuDS Component	Total Suspended Solids	Metals	Hydrocarbons
Swale	0.5	0.6	0.6
Detention Basin (x0.5)	0.25	0.25	0.3
Pond (x0.5)	0.35	0.35	0.25
Total	**1.1**	**1.2**	**1.15**

Table 4.4 – SuDS Mitigation Indices Catchment B

SuDS Component	Total Suspended Solids	Metals	Hydrocarbons
Swale	0.5	0.6	0.6
Raingarden (x0.5)	0.4	0.4	0.4
Detention Basin (x0.5)	0.25	0.25	0.3
Pond (x0.5)	0.35	0.35	0.25
Total	**1.5**	**1.6**	**1.55**

4.29 Tables 4.3 and 4.4 demonstrate that the total 'SuDS mitigation index' for both catchments is more than double the 'pollution hazard index' required, and therefore offers a robust level of water quality management, capable of mitigating the sites pollution hazard levels.

Long-term storage volume

4.30 The required long-term storage (LTS) volume has been calculated utilising Equation 24.10 within CIRIA C753 'The SuDS Manual'.

4.31 A copy of the calculation sheet has been included in Appendix G of this report, with the result summarised by Table 4.5 below;

Table 4.5 – Long-term Storage Volume (3.626ha)

Long-term Storage Volume (m^3)	229.32
Long-term Storage Discharge (l/s)	7.25

Attenuation storage volumes

4.32 Causeway FLOW has been used to determine the attenuation requirements for the development. This includes attenuation features, sized to accommodate runoff in up to the 100 year return period, with allowances for climate change.

4.33 Drawing 0759-PDL-101 identifies the SWMP for the proposed development, with a copy included in Appendix F of this report.

4.34 The output of the models can be seen within Appendix H of this report, with the results summarised in Table 4.6 below.

Table 4.6 – SuDS Storage Requirements

Feature	Imp. Catchment (ha)	100yr + 45% Vol. (m^3)
Detention Basin 1	1.766	730.7
Attenuation Pond 1	*Inflow from Detention Basin 1*	578.7
Total	**1.766**	**1,309.4**

Exceedance events

4.35 During exceedance events, beyond the 100 year critical storm, surface water runoff will overflow from the aforementioned systems.

4.36 Overland flows will follow the topography of the site and where possible will be stored aboveground, within parking courts, areas of public open space or within freeboard allowances of any open SuDs features. Beyond these, flows will be intercepted by the ordinary watercourse at the site's northern boundary and will continue off-site as per the pre-development scenario.

Proposed foul water strategy

4.37 A point of connection enquiry was submitted to SWW, with their response confirming the proposed point of connection for foul flows.

4.38 The nearest available foul network is located within Limetree Mead to the west of the site but requires access via a narrow public footpath. If this is infeasible (due to existing service locations) then an alternative location has been identified at the junction of Canal Hill and Cudmore Park.

4.39 Foul flows generated by the development will route through a new adoptable foul water network towards an adoptable pumping station located within the north eastern extents of the site. Flows will be pumped off-site to one of the available points of connection.

4.40 We have shown an indicative alignment for the new adoptable foul sewerage system on the preliminary drainage layout included within Appendix F of this report.

4.41 Copies of our correspondence with SWW are included within Appendix I of this report.

Maintenance

4.42 Any adoptable sewerage networks will be designed in accordance with the Design and Construction Guidance (DCG) as part of the Sewer Sector Guidance (SSG) and will be offered to SWW for adoption..

4.43 Any storm drainage which solely serves the adopted highway will be offered to Devon County Council for adoption.

4.44 Any private drainage will be designed in accordance with Building Regulations Part H and will become the responsibility of the respective homeowner, or where otherwise an appointed management company.

4.45 The operation and maintenance of all SuDS features will be undertaken by an appointed management company, with all works to be carried out strictly in accordance with 'CIRIA C753 – The SUDS Manual, Chapter 32 – Operation and Maintenance', securing long-term function, operation and performance.

5 Miscellaneous Issues

Construction issues

5.1 It is good practice to offer a Construction Environmental Management Plan (CEMP) to allow the construction and phasing of drainage works to be closely monitored. Prior to the commencement of construction, it is recommended the contractor produce a CEMP and agree it with the LLFA.

5.2 Any facilities for the storage of oils, fuels or chemicals need to be situated in suitable bunded bases that will be equivalent to at least the volume of the tank plus 10%.

Residual flood risks

5.3 The proposed development site is located wholly within 'Flood Zone 1' and is not significantly impacted by flooding from surface water. There are therefore no significant residual flood risks with regards to high risk flood zones or surface runoff.

5.4 The residual risk of blockage or failure of any key component within the proposed drainage strategy will be reduced through appropriate operation and maintenance procedures.

5.5 At the detailed design stage, the residual risks from exceedance storms will be reduced through appropriate design of the external works and highway alignments. The design will aim to steer exceedance flows towards convenient holding points such as areas of public open space and the proposed attenuation features.

5.6 Safe access and egress has been identified.

6 Mitigation, Conclusions and Recommendations

Mitigation

6.1 The proposed development has been assessed in line with the NPPF, to allow the planning application to be progressed and to show that the development can be undertaken in an acceptable manner from a flood risk perspective.

6.2 This proposed development site is located within 'Flood Zone 1', which means it is not at risk of flooding from fluvial sources in up to the 1 in 1000 year return period flood.

6.3 The surface water strategy for this site has been developed to respect the masterplan, accounting for runoff up to and including the 100 year critical storm event.

6.4 The strategy safeguards against the upper end allowances for climate change (45%), and makes allowance for urban creep. Both these measures will provide further betterment compared to the undeveloped site, where runoff would continue to increase as climate change occurs.

6.5 Site specific soakaway testing has been undertaken in accordance with BRE Digest 365 and confirmed that infiltration is not a viable method of surface water disposal. Instead the surface water management plan utilises an attenuated discharge to surface water.

6.6 The proposed drainage strategy utilises a combination of private and adoptable networks, swales with check-dams, raingardens, a new detention basin with forebay, a new attenuation pond and hydraulic controls.

6.7 The peak rates of discharge will be limited to the site's greenfield runoff rates and will include Long-Term Storage to mitigate the impact of any increased volume of runoff. The final designs will provide a further 10% reduction to generate additional capacity within the downstream culverts.

6.8 The use of SuDS will promote sedimentation of fines, reducing siltation of the downstream culverts which pass beneath the Grand Western Canal.

6.9 Exceedance flows will route towards convenient holding points, away from dwellings and primary access routes.

6.10 Foul flows generated by the development will be pumped off-site to available points of connection, as agreed with South West Water.

6.11 The proposed development not only promotes best-practice SuDS, providing multiple stages of treatment, but also mitigates any increased rate or volume of runoff and protects against the impacts of climate change, therefore providing a beneficial impact to downstream receptors, including the Tidcombe Fenn SSSI.

Conclusions

This Flood Risk Assessment has been assessed in line with the NPPF. It is concluded that the development can be undertaken in a sustainable manner, whilst also reducing the flood risk to existing properties in the downstream catchment.

The FRA does not attempt to present a final design of the surface water system. Detailed design of the surface water network and inherent features will commence upon approval of the outline strategy and will include assessments due to further site investigations, health and safety, CDM

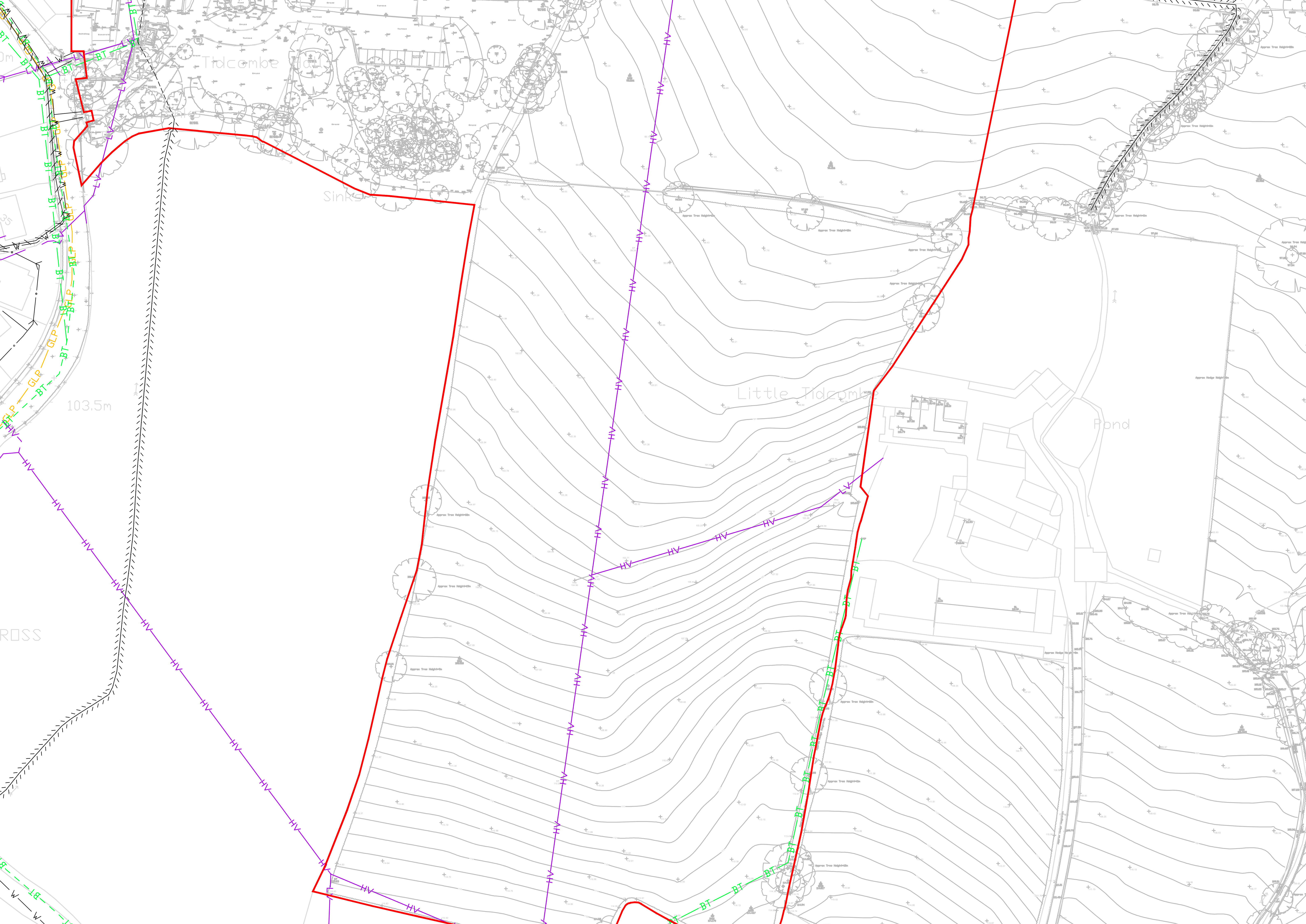
Recommendations

6.12 As the development will be safe from flooding for its design life and will actively reduce flood risk to properties in the downstream catchment, it is recommended that the Lead Local Flood Authority advise the local planning authority that they have no objections to the proposed development.

Appendix A Existing Site Plan

Approx Tree Height=11m
Approx Tree Height=10m
Approx Tree Height=15m
Approx Tree Height=12m
Approx Tree Height=19m
Approx Tree Height=9m
Approx Hedge Height=3m
Gate
Stay
EP
TP

Appendix B Combined Utilities Plan



Tidcombe Hall
Sinks
103.5m
Little Tidcombe
Pond
ROSS
HV
LV
BT
GLP

Appendix C Ruddlesden Ground Investigation Extract

remain dry. Therefore, some de-watering of temporary excavations is likely to be required across the majority of the site.

It is noted that groundwater levels fluctuate according to the season and from year to year. In the weeks prior to the investigation the weather had been average for the time of year. Therefore, higher groundwater levels may be encountered during periods of wetter weather. Likewise though, lower groundwater levels may be encountered during the drier summer months.

Some collapse of trial pit sides was recorded during the investigation, particularly within the Colluvium deposits. Therefore, some shoring of temporary excavations is likely to be required.

No problems with excavatability are foreseen across the majority of the site. However, one of the boreholes (WS04) refused (SPT >50) at a depth of 1.45m. Therefore, it is possible that some heavy plant and/ or mechanical breaking may be required locally, if depths greater than this are required.

7.5 Roads

In-situ CBR testing (TRL DCP method) produced estimated CBR values ranging from 3% to 75%.

The TRL DCP can sometimes produce artificially high CBR values. The laboratory testing results showed the near surface clays to be of low to intermediate plasticity. With reference to the Table of Sub-Grade CBR Estimation within Highways Agency Interim Advice Note IAN 73/06, the laboratory tests results and anticipated long-term groundwater levels, it is recommended that a CBR value of 2.5% be used for road pavement design at this site for the natural soils.

Based on the laboratory testing results, it is considered that the soils are frost-susceptible.

If highways are to be adopted, additional in-situ CBR testing may need to be undertaken by the adopting authority along the line of the highway at and below road formation level to confirm the CBR value.

7.6 Soakaways

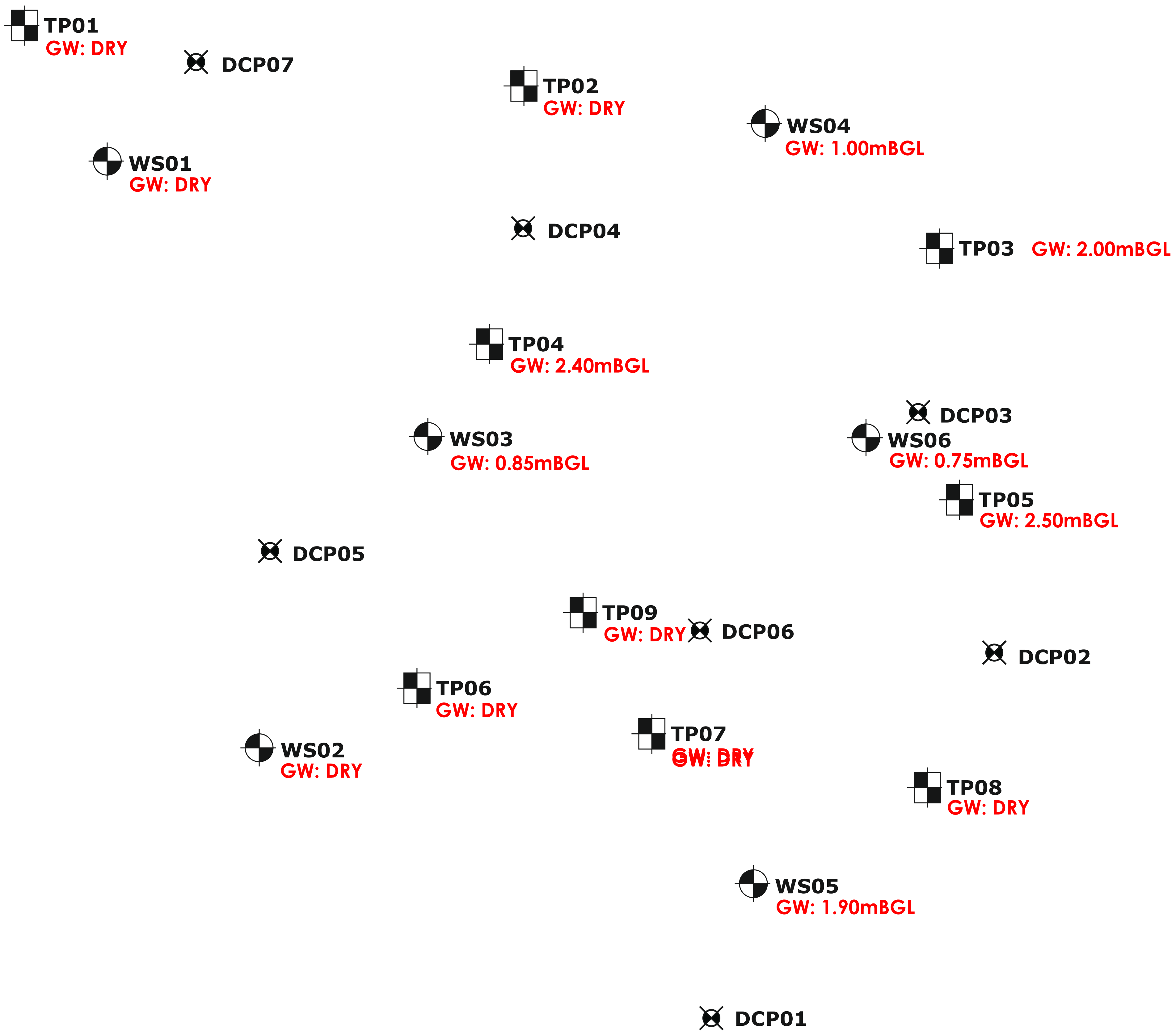
In-situ soakaway testing was undertaken at eight locations in general accordance with BRE DG 365: Soakaway Design.

Water level falls of between 0.00m and 0.77m were recorded over the course of the testing, although the water level within TP05 was observed to rise by 0.10m, due to groundwater ingress. All of the tests failed to reach 75% of the effective depth.

These results indicate that the ground has a very low to low permeability and is not likely to be suitable for the use of soakaway drainage, as any soakaways would necessarily be quite large and would probably not be able to fulfil the criteria to half-empty in a 24-hour period. In addition, the presence of a relatively high groundwater table at the site is likely make soakaway drainage unfeasible.

On-site attenuation combined with off-site discharge is considered likely to be the most suitable drainage solution at this site.

The preferable drainage solution at this site would appear to be to discharge into the sewer or suitable outfall. If necessary, underground attenuation tanks with a throttled outflow valve may be able to be installed to allow water to be discharged at an agreed rate so that during storm periods discharge is not increased from the present situation.



LEGEND:

- Borehole
- Trial Pit

NOTES:

Drawn on plan supplied by client

Job Title:	LAND AT TIDCOMBE HALL TIVERTON DEVON
Drawing Title:	EXPLORATORY HOLE LOCATION PLAN
Client:	LAND VALUE ALLIANCES LLP

The Stables
65 Langaton Lane
Pinhoe Exeter
EX1 3SP
www.ruddlesden.co.uk

Dwg. No:	18439/02
Date:	OCT-18
Scale:	NTS

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP01

Trial Pit Dimensions

Length (m):	2.00
Width (m):	0.75
Depth (m):	2.10
Start Water Level (m):	1.00
Total Depth of Test	1.10

Field Results

Time (minutes)	Water Level (mBGL)
0	1.00
43	1.04
115	1.05
175	1.07
235	1.08

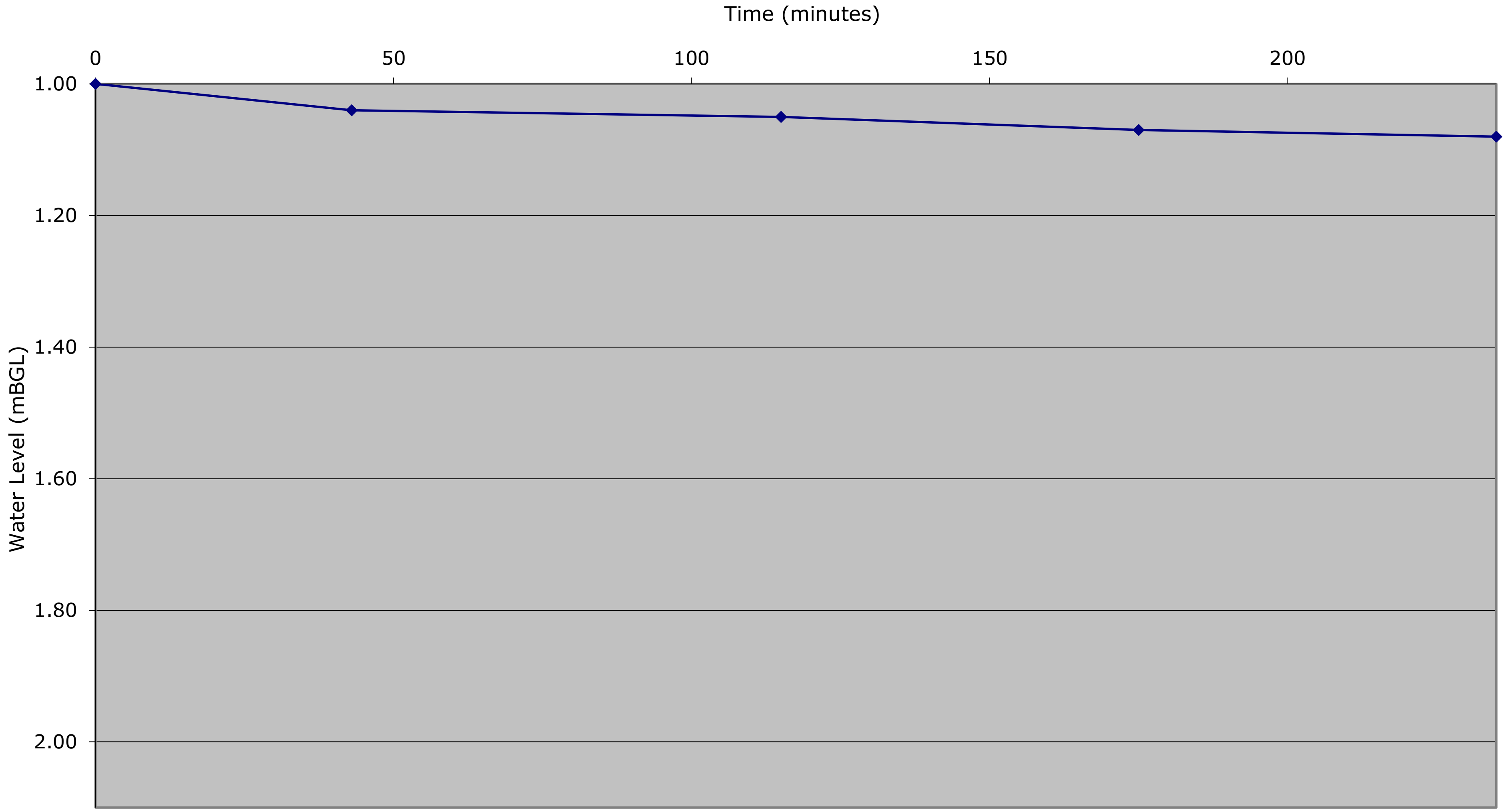
Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
Where V_{p75-25}	=	effective storage volume of water in the trial pit between 75% and 25% effective depth
	=	2.00 x 0.75 x 0.55
	=	0.825 m^3
a_{p50}	=	internal surface area of the trial pit up to 50% effective depth and including the base area
	=	0.83 + 2.20 + 1.50
	=	4.525 m^2
t_{p75-25}	=	time for the water level to fall from 75% to 25% effective depth 25% effective depth = 1.275 75% effective depth = 1.825
	=	- mins
	=	0 mins
	=	0 secs
Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
	=	0.825 / 4.525 x 0
	=	#DIV/0! m/s

OTHER NOTES:

Soakaway Test Results - TP01



Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP02

Trial Pit Dimensions

Length (m):	1.90
Width (m):	0.70
Depth (m):	2.40
Start Water Level (m):	1.20
Total Depth of Test	1.20

Field Results

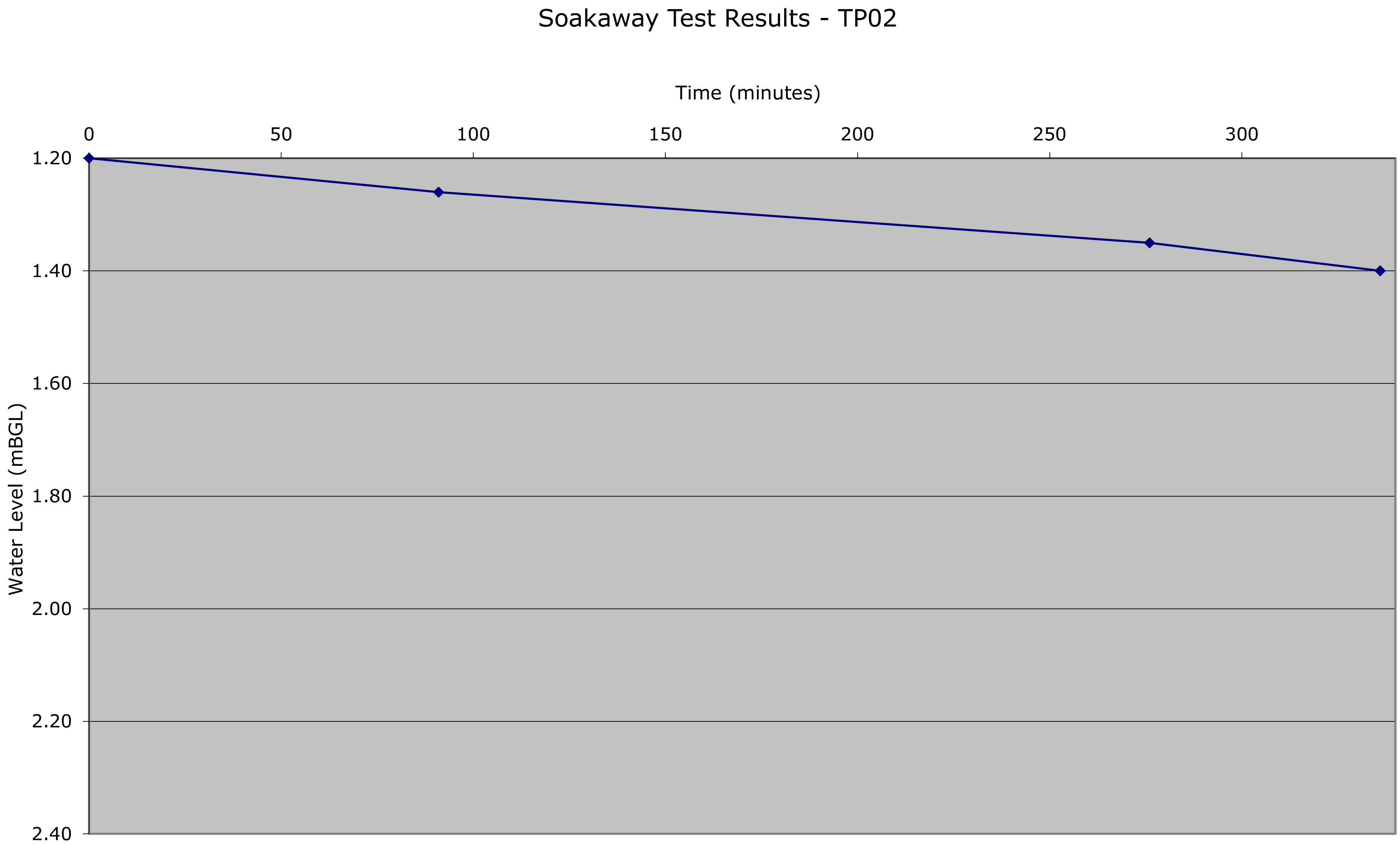
Time (minutes)	Water Level (mBGL)
0	1.20
91	1.26
276	1.35
336	1.40

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
Where		
V_{p75-25}	=	effective storage volume of water in the trial pit between 75% and 25% effective depth
	=	1.90 x 0.70 x 0.60
	=	0.798 m^3
a_{p50}	=	internal surface area of the trial pit up to 50% effective depth and including the base area
	=	0.84 + 2.28 + 1.33
	=	4.45 m^2
t_{p75-25}	=	time for the water level to fall from 75% to 25% effective depth 25% effective depth = 1.5 75% effective depth = 2.1
	=	- mins
	=	0 mins
	=	0 secs
Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
	=	0.798 / 4.45 x 0
	=	#DIV/0! m/s

OTHER NOTES:


Soakaway Test Results - TP02
Time (minutes)
0
50
100
150
200
250
300
1.20
1.40
1.60
1.80
2.00
2.20
2.40
Water Level (mBGL)

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP03

Trial Pit Dimensions

Length (m):	2.20
Width (m):	0.80
Depth (m):	2.50
Start Water Level (m):	1.15
Total Depth of Test	1.35

Field Results

Time (minutes)	Water Level (mBGL)
0	1.15
45	1.15
110	1.15
170	1.15
230	1.15

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f) = $(V_{p75-25}) / (a_{p50} \times t_{p75-25})$

Where

V_{p75-25} = effective storage volume of water in the trial pit between 75% and 25% effective depth

= 2.20 x 0.80 x 0.68

= 1.188 m^3

a_{p50} = internal surface area of the trial pit up to 50% effective depth and including the base area

= 1.08 + 2.97 + 1.76

= 5.81 m^2

t_{p75-25} = time for the water level to fall from 75% to 25% effective depth

25% effective depth = 1.4875

75% effective depth = 2.1625

= - mins

= 0 mins

= 0 secs

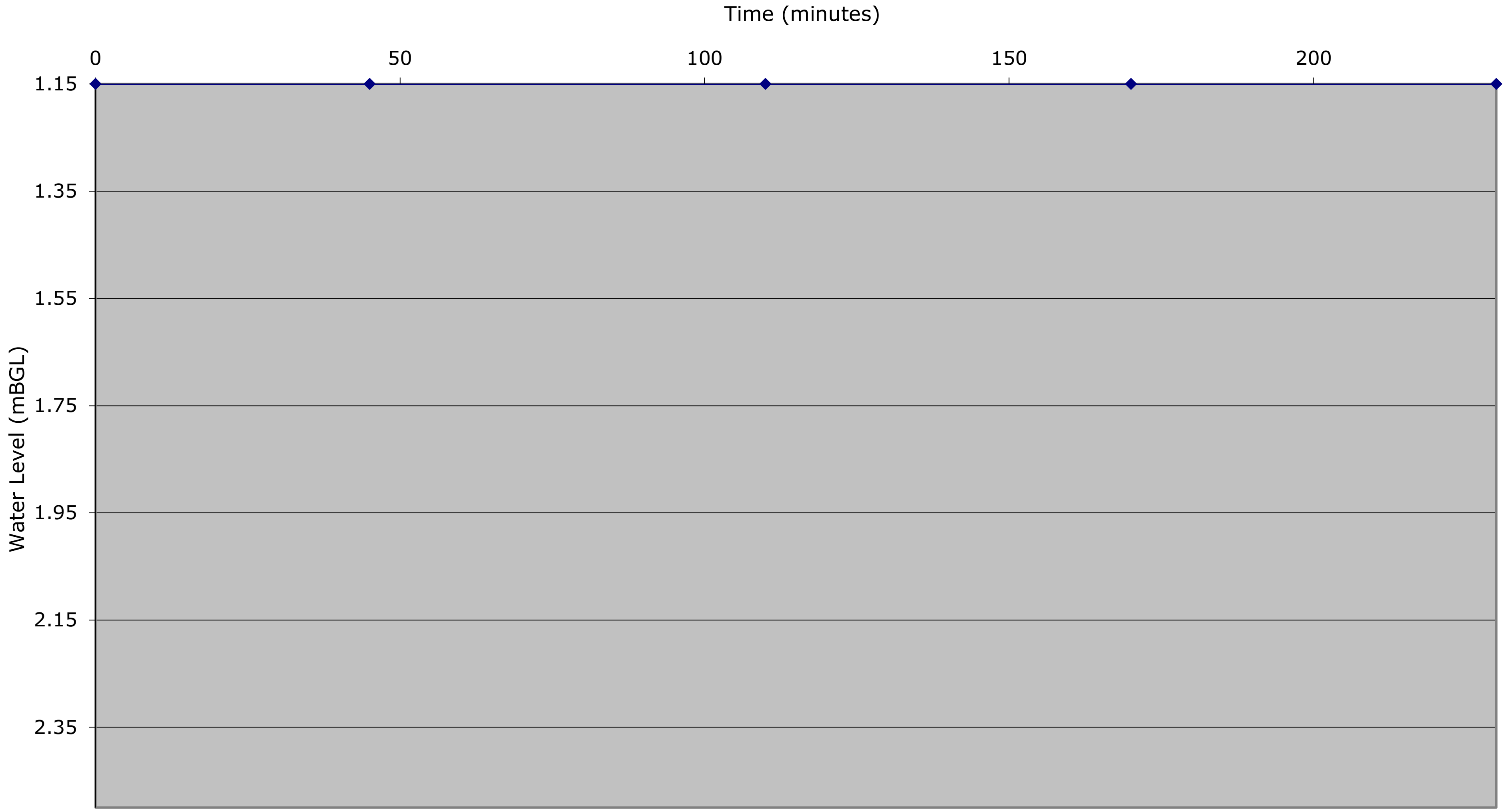
Soil Infiltration Rate (f) = $(V_{p75-25}) / (a_{p50} \times t_{p75-25})$

= 1.188 / 5.81 x 0

= #DIV/0! m/s

OTHER NOTES:

Soakaway Test Results - TP03



Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP04

Trial Pit Dimensions

Length (m):	2.20
Width (m):	0.70
Depth (m):	2.40
Start Water Level (m):	1.17
Total Depth of Test	1.23

Field Results

Time (minutes)	Water Level (mBGL)
0	1.17
122	1.17
317	1.19
377	1.19

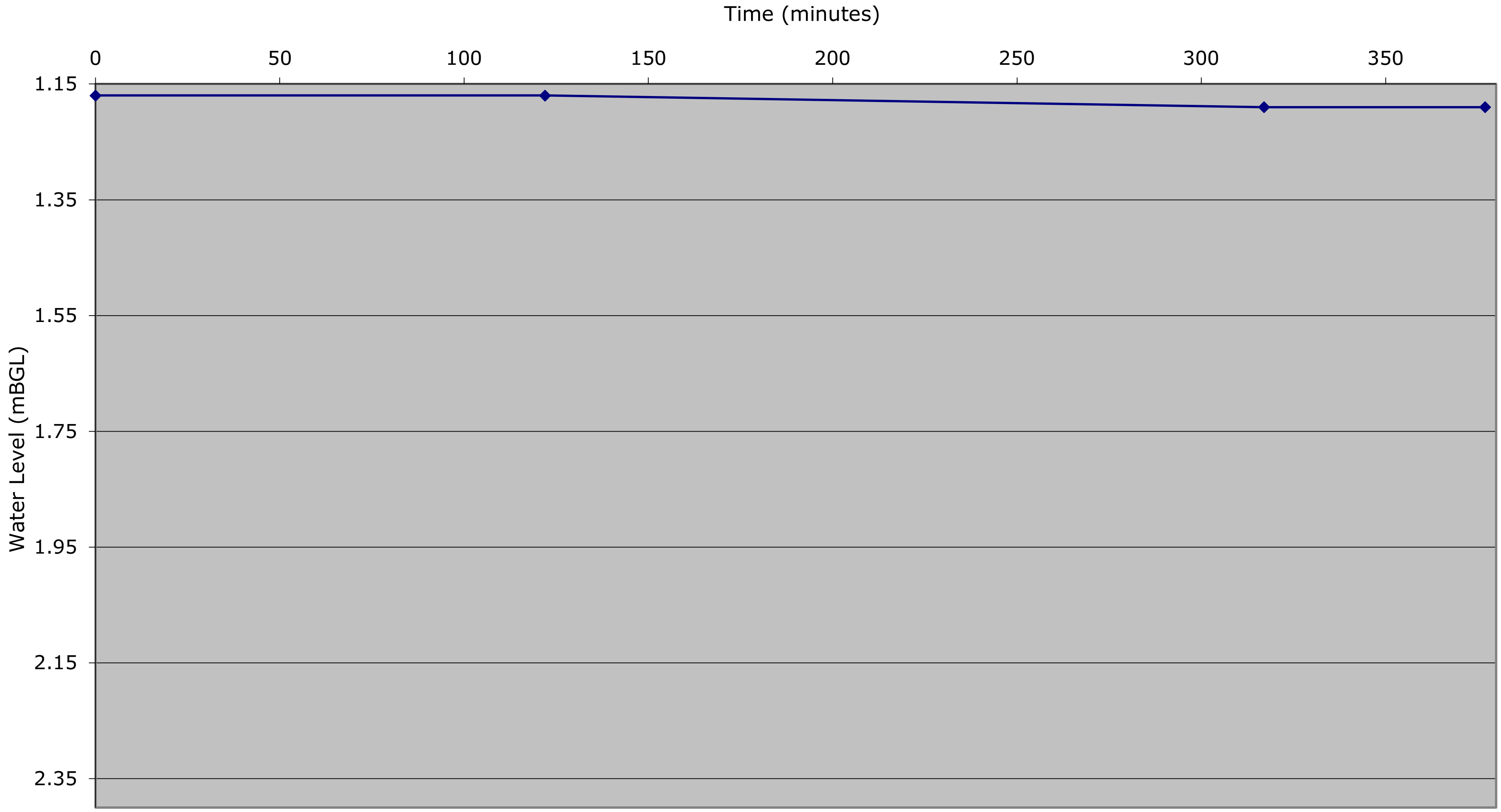
Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
Where		
V_{p75-25}	=	effective storage volume of water in the trial pit between 75% and 25% effective depth
	=	2.20 x 0.70 x 0.62
	=	0.9471 m^3
a_{p50}	=	internal surface area of the trial pit up to 50% effective depth and including the base area
	=	0.86 + 2.71 + 1.54
	=	5.107 m^2
t_{p75-25}	=	time for the water level to fall from 75% to 25% effective depth 25% effective depth = 1.4775 75% effective depth = 2.0925
	=	- mins
	=	0 mins
	=	0 secs
Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
	=	0.9471 / 5.107 x 0
	=	#DIV/0! m/s

OTHER NOTES:

Soakaway Test Results - TP04



Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP05

Trial Pit Dimensions

Length (m):	2.10
Width (m):	0.90
Depth (m):	2.50
Start Water Level (m):	1.00
Total Depth of Test	1.50

Field Results

Time (minutes)	Water Level (mBGL)
0	1.50
75	1.40
150	1.40
210	1.40
270	1.40

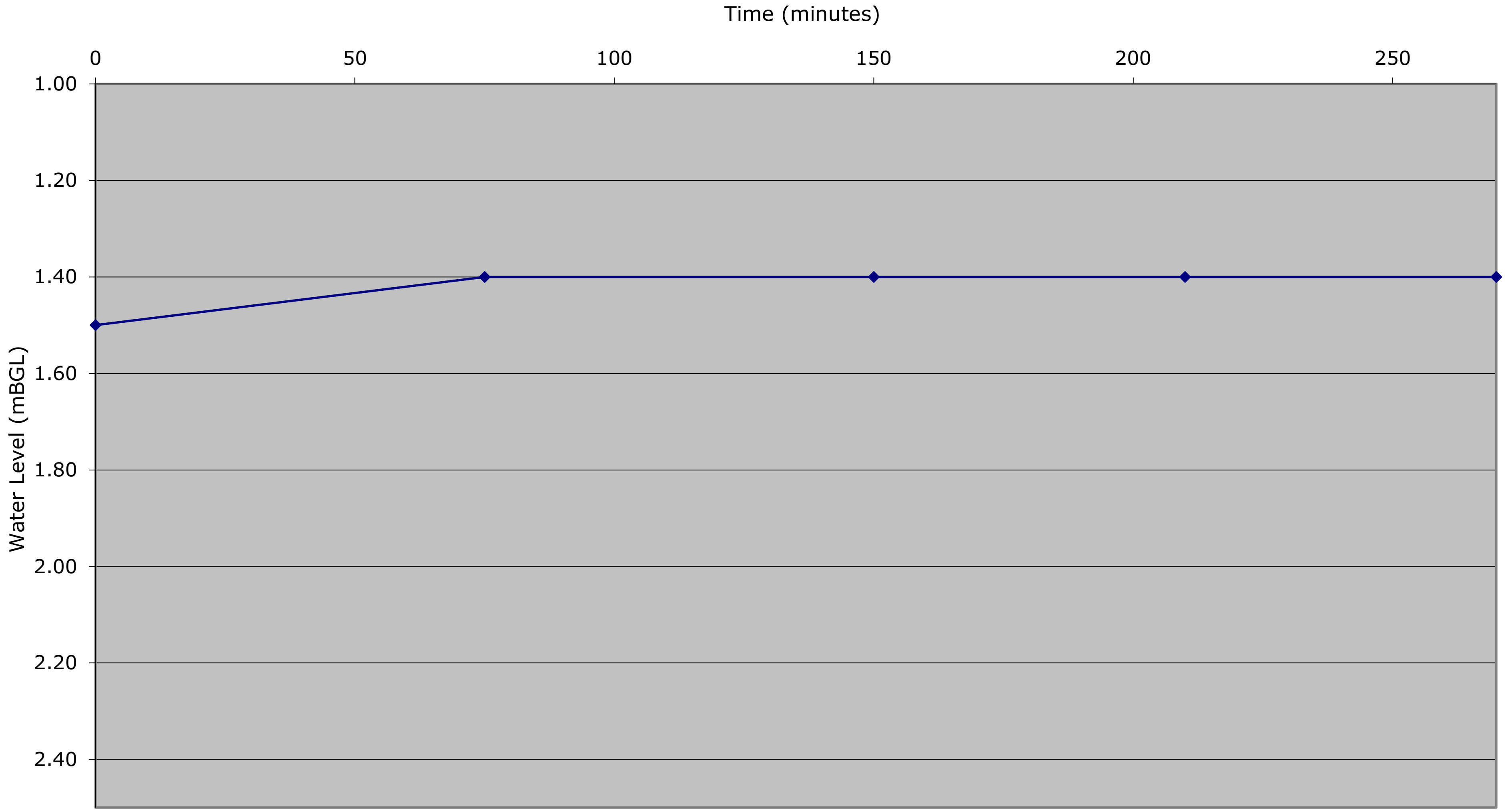
Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
Where V_{p75-25}	=	effective storage volume of water in the trial pit between 75% and 25% effective depth
	=	2.10 x 0.90 x 0.75
	=	1.4175 m^3
a_{p50}	=	internal surface area of the trial pit up to 50% effective depth and including the base area
	=	1.35 + 3.15 + 1.89
	=	6.39 m^2
t_{p75-25}	=	time for the water level to fall from 75% to 25% effective depth 25% effective depth = 1.375 75% effective depth = 2.125
	=	- mins
	=	0 mins
	=	0 secs
Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
	=	1.4175 / 6.39 x 0
	=	#DIV/0! m/s

OTHER NOTES:

Soakaway Test Results - TP05



Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP06

Trial Pit Dimensions

Length (m):	2.10
Width (m):	0.90
Depth (m):	2.50
Start Water Level (m):	1.00
Total Depth of Test	1.50

Field Results

Time (minutes)	Water Level (mBGL)
0	1.00
145	1.16
280	1.30
340	1.39

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f) = $(V_{p75-25}) / (a_{p50} \times t_{p75-25})$

Where

V_{p75-25} = effective storage volume of water in the trial pit between 75% and 25% effective depth

= 2.10 x 0.90 x 0.75

= 1.4175 m^3

a_{p50} = internal surface area of the trial pit up to 50% effective depth and including the base area

= 1.35 + 3.15 + 1.89

= 6.39 m^2

t_{p75-25} = time for the water level to fall from 75% to 25% effective depth

25% effective depth = 1.375

75% effective depth = 2.125

= - mins

= 0 mins

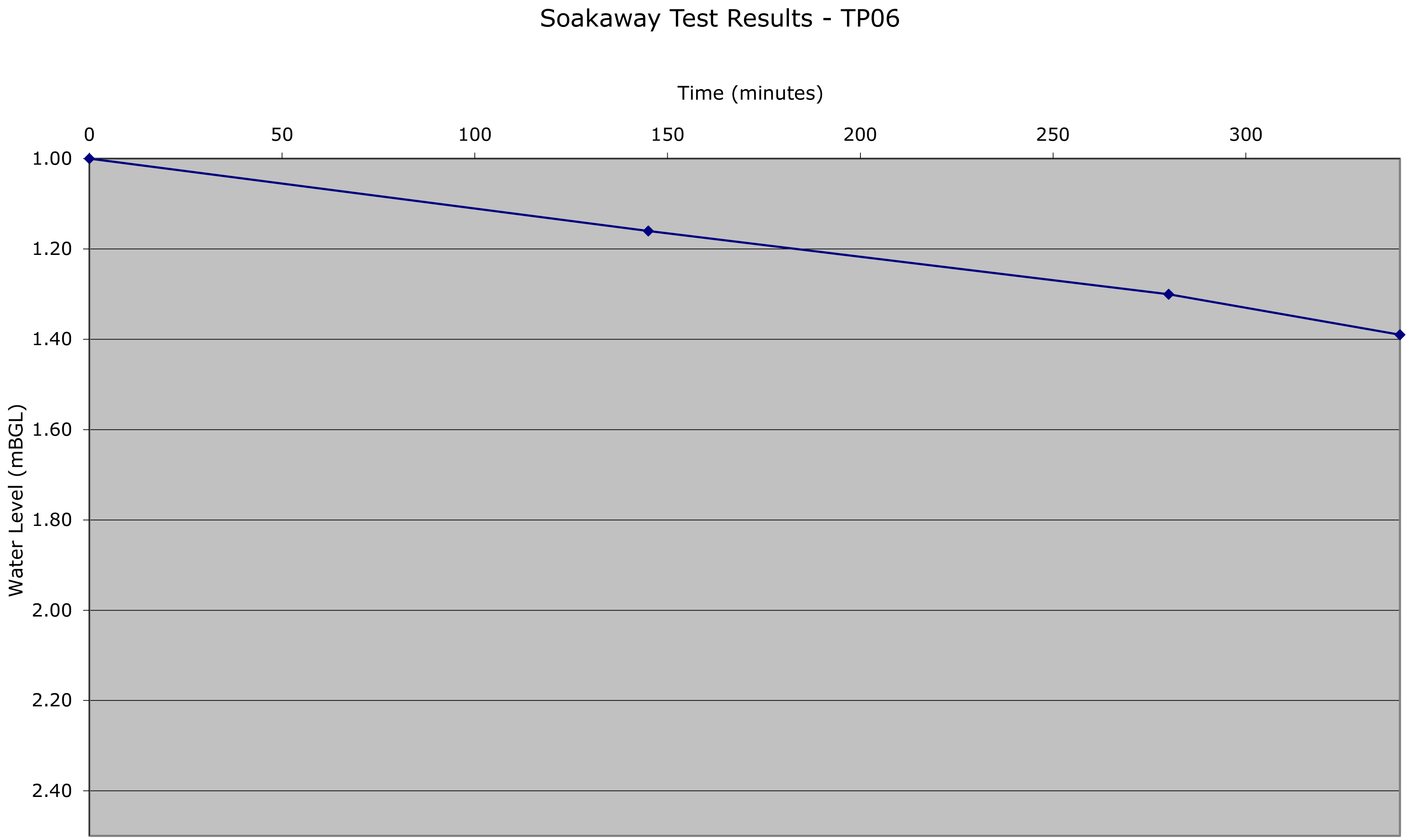
= 0 secs

Soil Infiltration Rate (f) = $(V_{p75-25}) / (a_{p50} \times t_{p75-25})$

= 1.4175 / 6.39 x 0

= #DIV/0! m/s

OTHER NOTES:


Soakaway Test Results - TP06
Time (minutes)
0
50
100
150
200
250
300
Water Level (mBGL)
1.00
1.20
1.40
1.60
1.80
2.00
2.20
2.40

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP07

Trial Pit Dimensions

Length (m):	2.00
Width (m):	0.70
Depth (m):	2.40
Start Water Level (m):	1.00
Total Depth of Test	1.40

Field Results

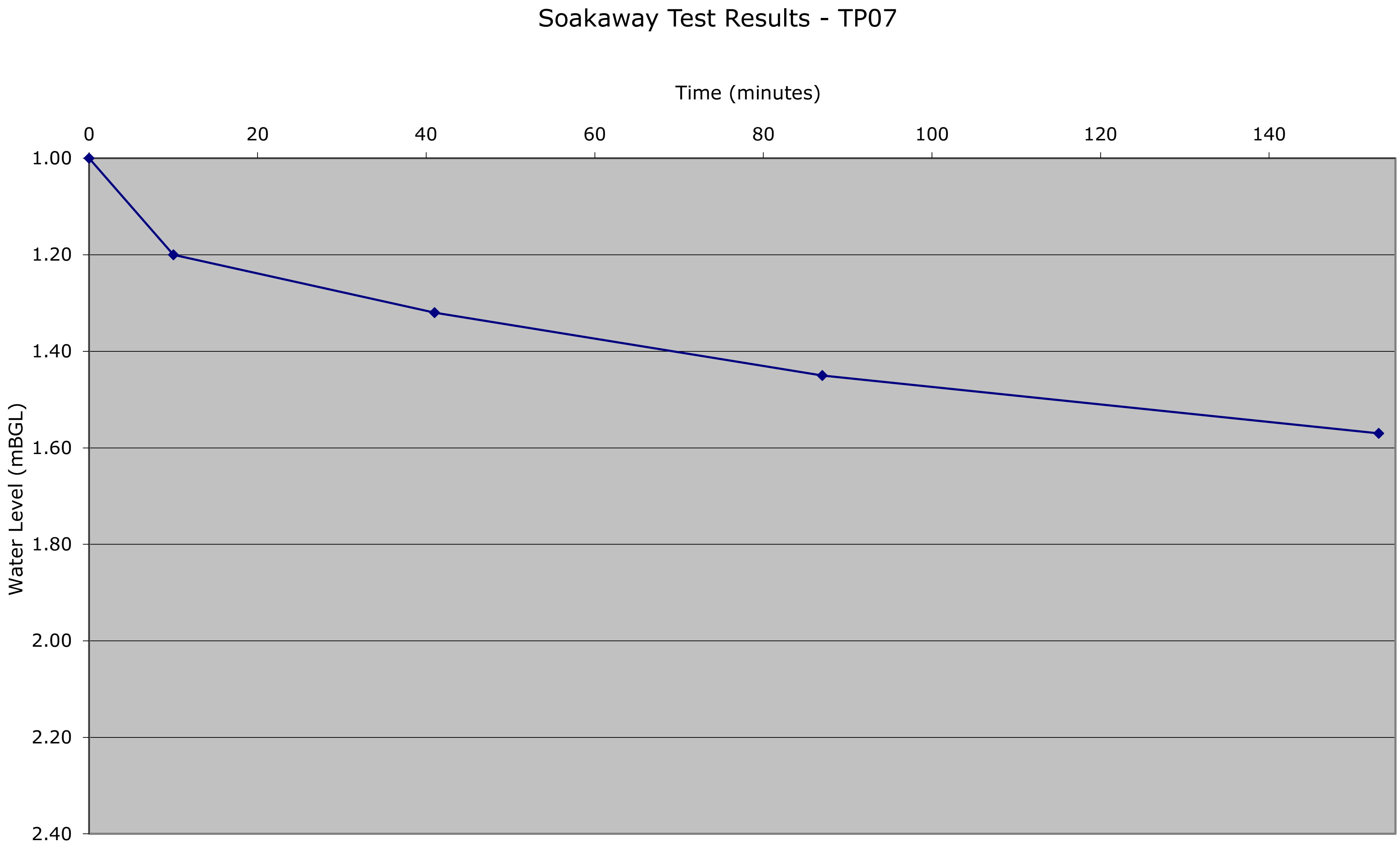
Time (minutes)	Water Level (mBGL)
0	1.00
10	1.20
41	1.32
87	1.45
153	1.57

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
Where		
V_{p75-25}	=	effective storage volume of water in the trial pit between 75% and 25% effective depth
	=	2.00 x 0.70 x 0.70
	=	0.98 m^3
a_{p50}	=	internal surface area of the trial pit up to 50% effective depth and including the base area
	=	0.98 + 2.80 + 1.40
	=	5.18 m^2
t_{p75-25}	=	time for the water level to fall from 75% to 25% effective depth 25% effective depth = 1.35 75% effective depth = 2.05
	=	- mins
	=	0 mins
	=	0 secs
Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
	=	0.98 / 5.18 x 0
	=	#DIV/0! m/s

OTHER NOTES:


Soakaway Test Results - TP07
Time (minutes)
0
20
40
60
80
100
120
140
1.00
1.20
1.40
1.60
1.80
2.00
2.20
2.40
Water Level (mBGL)

Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Job Title:	Land at Tidcombe Hall, Tiverton, Devon
Job No.:	18439
Client:	Land Value Alliances LLP
Date:	Oct-18

Test No. TP08

Trial Pit Dimensions

Length (m):	1.70
Width (m):	0.70
Depth (m):	2.50
Start Water Level (m):	1.30
Total Depth of Test	1.20

Field Results

Time (minutes)	Water Level (mBGL)
0	1.30
2	1.35
16	1.55
62	1.80
81	1.85
197	2.07

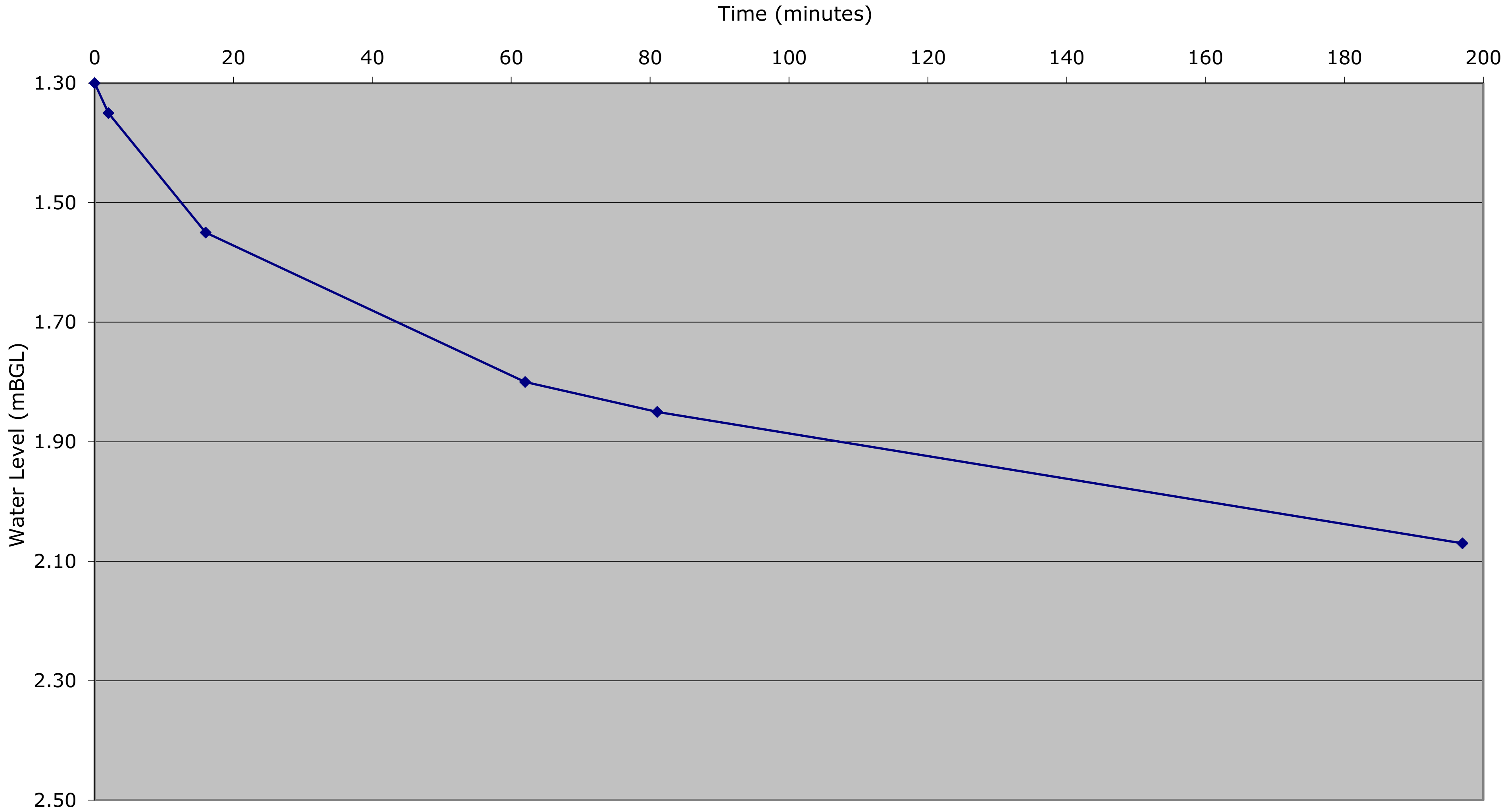
Soakaway Test Results
In Accordance with BRE 365 "Soakaway Design"

Calculations

Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
Where		
V_{p75-25}	=	effective storage volume of water in the trial pit between 75% and 25% effective depth
	=	1.70 x 0.70 x 0.60
	=	0.714 m^3
a_{p50}	=	internal surface area of the trial pit up to 50% effective depth and including the base area
	=	0.84 + 2.04 + 1.19
	=	4.07 m^2
t_{p75-25}	=	time for the water level to fall from 75% to 25% effective depth 25% effective depth = 1.6 75% effective depth = 2.2
	=	- mins
	=	0 mins
	=	0 secs
Soil Infiltration Rate (f)	=	$(V_{p75-25}) / (a_{p50} \times t_{p75-25})$
	=	0.714 / 4.07 x 0
	=	#DIV/0! m/s

OTHER NOTES:

Soakaway Test Results - TP08



Appendix D Illustrative Masterplan

Tidcombe Hall, Tiverton

Illustrative layout

To Blundells Road

To Tiverton Town Centre

To Cullompton

- Application boundary
- 1 Vehicular access from Tidcombe Lane
- 2 Restored Tidcombe Hall and entrance space
- 3 High quality courtyard development - inc sympathetic conversion of existing outbuildings
- 4 Existing driveway - cycle/ pedestrian access
- 5 Community growing areas
- 6 Growing area and parking (including EV charging)
- 7 Existing landscape entrance retained and enhanced
- 8 Primary access route
- 9 Courtyard housing
- 10 Existing trees and hedgerows retained and enhanced
- 11 Public open space - Parkland landscape and enhanced Grand Western Canal corridor (including areas for SUDs)
- 12 Structured residential development - enabling high quality living environments and public realm
- 13 Low density courtyard style development - transitional development edge
- 14 Landscape buffer planting enhancing wildlife corridors
- 15 Opportunities for orchard tree planting
- 16 Proposed bat roost building
- 17 10m wide dark crossing point over access road to allow for bat movement

N

CliftonEmerydesign

Ref: 230301 L 02 02 F

Appendix E Greenfield Runoff FEH Assessment

Simulation Settings

Rainfall Methodology	FEH-13	Skip Steady State	x	2 year (l/s)	4.8
Summer CV	0.750	Drain Down Time (mins)	240	30 year (l/s)	10.6
Winter CV	0.840	Additional Storage (m³/ha)	20.0	100 year (l/s)	13.5
Analysis Speed	Normal	Check Discharge Rate(s)	✓	Check Discharge Volume	x

Storm Durations

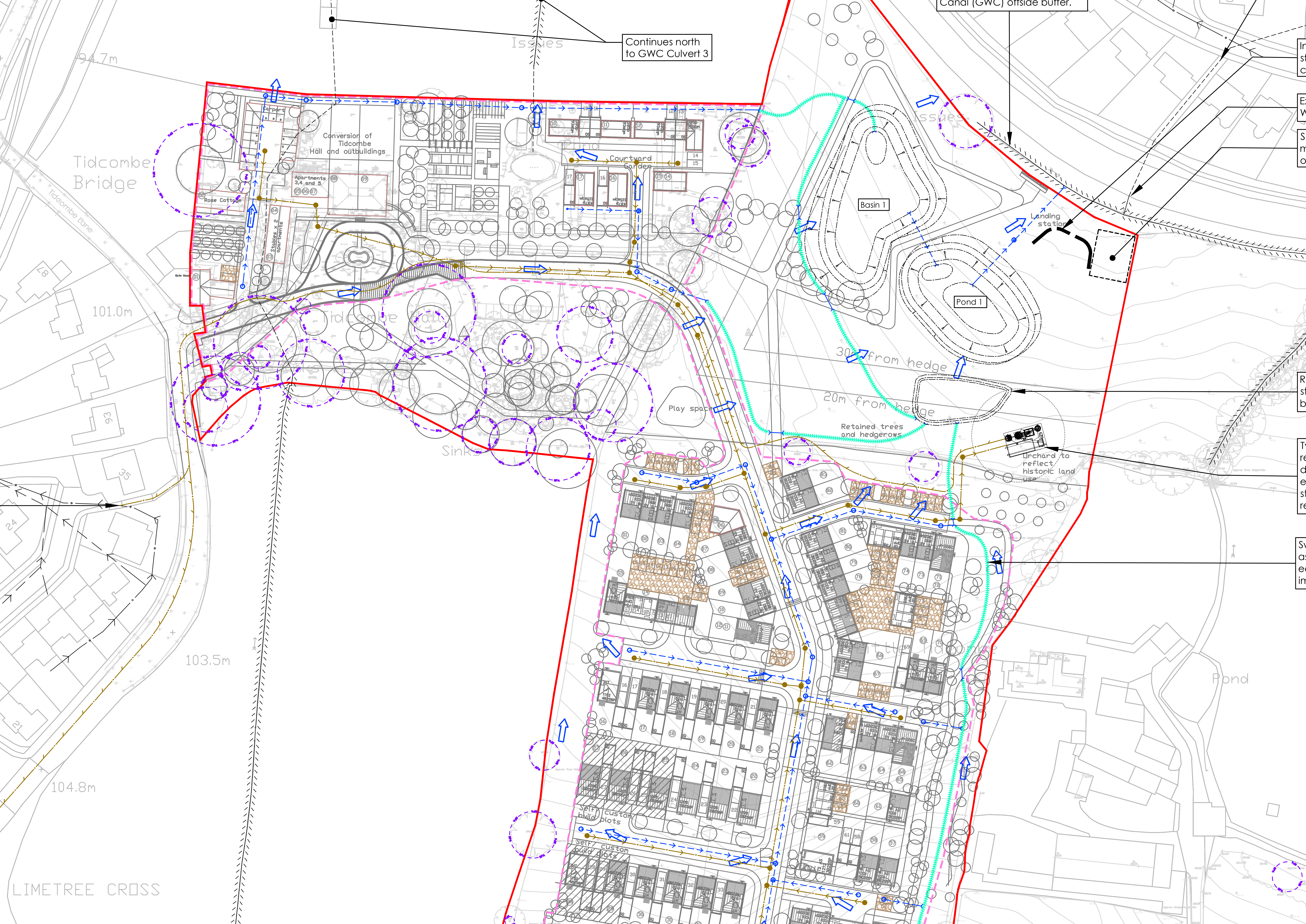
15	60	180	360	600	960	2160	4320	7200	10080
30	120	240	480	720	1440	2880	5760	8640	

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
2	40	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)	1.685	Betterment (%)	0
SAAR (mm)	967	QMed	5.1
Host	1	QBar	5.5
BFIHost	0.694	Q 2 year (l/s)	4.8
Region	8	Q 30 year (l/s)	10.6
QBar/QMed conversion factor	1.075	Q 100 year (l/s)	13.5
Growth Factor 2 year	0.88		

Appendix F Preliminary Drainage Layout



Canal (GWC) offside buffer.
Continues north
to GWC Culvert 3
Issues
94.7m
Tidcombe
Bridge
Tidcombe Lane
Conversion of
Tidcombe
Hall and outbuildings
Apartments
3,4 and 5
Rose Cottage
Stables x 2
apartments
Carport
Courtyard
Garden
Basin 1
Pond 1
Landing
station
30m from hedge
20m from hedge
Retained trees
and hedgerows
Play space
Orchard to
reflect
historic land
use
101.0m
103.5m
104.8m
Sinks
Self custom
build plots
Self custom
build plots
Pond
LIMETREE CROSS

Appendix G Long Term Storage Calculations

Long Term Storage (LTS) Volume Calculation

Project No.	*0759*
Project Title	*Tidcombe Hall, Tiverton*
Client	*Tidcombe Holdings LVA LLP*
Sheet Ref	*LTS Volume Calculation*

Calcs by	*VS*
Reviewed by	*JB*
Date	*08/09/2023*
Revision	*B*

LTS calculation method based on equation 24.10 from CIRIA C753 - The SuDS Manual (2015);

$$Vol_{xs} = RD \; x \; A \; x \; 10 \; [PIMP/100 \; x \; (\alpha \; x \; Cv) + (1\text{-}PIMP/100) \; x \; (\beta \; x \; SPR) - SPR]$$

Where; Vol_{xs} Extra runoff volume from a dev. site compared to the greenfield equivalent during the 100 yr 6 hr storm

RD	Rainfall Depth	70	mm	*(for 100 year 6 hour storm)*
A	*Site Area*	3.626	ha	*(Exc. large undeveloped areas)*
	Impermeable Catchment	1.685	ha	
PIMP	*Percentage Impermeable*	46.5	%	
α	Proportion Impermeable to Network	1.0		
Cv	Impermeable Runoff Coefficient	0.84		*(0.84 Modified Rational Method)*
	Permeable Catchment	0.00	ha	
	Permeable Catchment to Network	0.00	ha	
β	Proportion Perm. to Network	0.00		
SPR	Soil Proportion Runoff	0.30		*(Ref. to WRAP map)*

$$Vol_{xs} = \underset{RD}{70} \times \underset{A}{3.63} \times 10 \times ((\underset{PIMP}{46} / 100) \times (\underset{\alpha}{1.00} \times \underset{Cv}{0.84}) + (1 - \underset{PIMP}{46} / 100) \times (\underset{\beta}{0.00} \times \underset{SPR}{0.30}) - \underset{SPR}{0.30})$$

$Volume_{xs}$	**229.32**	
LTS Discharge Rate	**7.25**	*(2 l/s/ha)*

As above, assuming all permeable surfaces do not enter the drainage system

Vol_{xs} = 229.32

As above, assuming all permeable surfaces enter the drainage system

Vol_{xs} = 636.93

Appendix H Causeway FLOW Calculations

Design Settings

Rainfall Methodology	FEH-13	Minimum Velocity (m/s)	1.00
Return Period (years)	100	Connection Type	Level Soffits
Additional Flow (%)	45	Minimum Backdrop Height (m)	0.200
CV	0.750	Preferred Cover Depth (m)	1.200
Time of Entry (mins)	4.00	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)	Easting (m)	Northing (m)	Depth (m)
1	1.766	4.00	95.000	1200	297659.530	112176.098	1.100
2	0.000	4.00	95.000	1200	297631.508	112204.553	1.100
OUT	0.000	4.00	92.700	1050	297669.567	112213.731	1.000

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
1	1	2	5.000	0.600	93.900	93.900	0.000	0.0	300	4.08	50.0
2	2	OUT	5.000	0.600	93.900	91.700	2.200	2.3	300	4.09	50.0

Name	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
1	1.000	70.7	347.0	0.800	0.800	1.766	0.0	0	∞
2	10.500	742.2	347.0	0.800	0.700	1.766	0.0	144	10.339

Simulation Settings

Rainfall Methodology	FEH-13	Skip Steady State	x	2 year (l/s)	4.8
Summer CV	0.750	Drain Down Time (mins)	240	30 year (l/s)	10.6
Winter CV	0.840	Additional Storage (m³/ha)	20.0	100 year (l/s)	13.5
Analysis Speed	Normal	Check Discharge Rate(s)	✓	Check Discharge Volume	x

Storm Durations

15	60	180	360	600	960	2160	4320	7200	10080
30	120	240	480	720	1440	2880	5760	8640	

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

Pre-development Discharge Rate

Site Makeup	Greenfield	Region	8
Greenfield Method	FEH	QBar/QMed conversion factor	1.075
Positively Drained Area (ha)	1.685	Growth Factor 2 year	0.88
SAAR (mm)	967	Growth Factor 30 year	1.95
Host	1	Growth Factor 100 year	2.48
BFIHost	0.694	Betterment (%)	0

Pre-development Discharge Rate

QMed	5.1	Q 30 year (l/s)	10.6
QBar	5.5	Q 100 year (l/s)	13.5
Q 2 year (l/s)	4.8		

Node 2 Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	x	Sump Available	✓
Invert Level (m)	93.900	Product Number	CTL-SHE-0110-4800-0500-4800
Design Depth (m)	0.500	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	4.8	Min Node Diameter (mm)	1200

Node 2 Online Orifice Control

Flap Valve	x	Design Depth (m)	0.200	Discharge Coefficient	0.600
Replaces Downstream Link	x	Design Flow (l/s)	8.7		
Invert Level (m)	94.500	Diameter (m)	0.098		

Node 1 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	93.900
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	755.0	0.0	1.100	1110.6	0.0

Node 2 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	93.900
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	610.0	0.0	1.100	933.1	0.0

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
2 year +40% CC 15 minute summer	153.681	43.486
2 year +40% CC 15 minute winter	107.846	43.486
2 year +40% CC 30 minute summer	101.315	28.669
2 year +40% CC 30 minute winter	71.099	28.669
2 year +40% CC 60 minute summer	68.869	18.200
2 year +40% CC 60 minute winter	45.755	18.200
2 year +40% CC 120 minute summer	45.425	12.004
2 year +40% CC 120 minute winter	30.179	12.004
2 year +40% CC 180 minute summer	35.912	9.241
2 year +40% CC 180 minute winter	23.344	9.241
2 year +40% CC 240 minute summer	28.891	7.635
2 year +40% CC 240 minute winter	19.195	7.635
2 year +40% CC 360 minute summer	22.560	5.805
2 year +40% CC 360 minute winter	14.664	5.805

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
2 year +40% CC 480 minute summer	18.043	4.768
2 year +40% CC 480 minute winter	11.987	4.768
2 year +40% CC 600 minute summer	14.956	4.091
2 year +40% CC 600 minute winter	10.219	4.091
2 year +40% CC 720 minute summer	13.466	3.609
2 year +40% CC 720 minute winter	9.050	3.609
2 year +40% CC 960 minute summer	11.251	2.963
2 year +40% CC 960 minute winter	7.453	2.963
2 year +40% CC 1440 minute summer	8.418	2.256
2 year +40% CC 1440 minute winter	5.657	2.256
2 year +40% CC 2160 minute summer	6.275	1.734
2 year +40% CC 2160 minute winter	4.324	1.734
2 year +40% CC 2880 minute summer	5.408	1.449
2 year +40% CC 2880 minute winter	3.635	1.449
2 year +40% CC 4320 minute summer	4.366	1.142
2 year +40% CC 4320 minute winter	2.875	1.142
2 year +40% CC 5760 minute summer	3.806	0.974
2 year +40% CC 5760 minute winter	2.463	0.974
2 year +40% CC 7200 minute summer	3.403	0.868
2 year +40% CC 7200 minute winter	2.196	0.868
2 year +40% CC 8640 minute summer	3.113	0.794
2 year +40% CC 8640 minute winter	2.009	0.794
2 year +40% CC 10080 minute summer	2.901	0.740
2 year +40% CC 10080 minute winter	1.872	0.740
30 year +40% CC 15 minute summer	368.762	104.347
30 year +40% CC 15 minute winter	258.781	104.347
30 year +40% CC 30 minute summer	246.457	69.739
30 year +40% CC 30 minute winter	172.952	69.739
30 year +40% CC 60 minute summer	169.387	44.764
30 year +40% CC 60 minute winter	112.537	44.764
30 year +40% CC 120 minute summer	98.999	26.163
30 year +40% CC 120 minute winter	65.773	26.163
30 year +40% CC 180 minute summer	74.202	19.095
30 year +40% CC 180 minute winter	48.233	19.095
30 year +40% CC 240 minute summer	57.795	15.273
30 year +40% CC 240 minute winter	38.398	15.273
30 year +40% CC 360 minute summer	43.364	11.159
30 year +40% CC 360 minute winter	28.188	11.159
30 year +40% CC 480 minute summer	33.866	8.950
30 year +40% CC 480 minute winter	22.500	8.950
30 year +40% CC 600 minute summer	27.612	7.553
30 year +40% CC 600 minute winter	18.866	7.553
30 year +40% CC 720 minute summer	24.556	6.581
30 year +40% CC 720 minute winter	16.503	6.581
30 year +40% CC 960 minute summer	20.156	5.308
30 year +40% CC 960 minute winter	13.352	5.308
30 year +40% CC 1440 minute summer	14.711	3.943
30 year +40% CC 1440 minute winter	9.887	3.943
30 year +40% CC 2160 minute summer	10.658	2.946
30 year +40% CC 2160 minute winter	7.344	2.946
30 year +40% CC 2880 minute summer	8.986	2.408
30 year +40% CC 2880 minute winter	6.039	2.408

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
30 year +40% CC 4320 minute summer	7.011	1.833
30 year +40% CC 4320 minute winter	4.617	1.833
30 year +40% CC 5760 minute summer	5.968	1.528
30 year +40% CC 5760 minute winter	3.862	1.528
30 year +40% CC 7200 minute summer	5.255	1.341
30 year +40% CC 7200 minute winter	3.392	1.341
30 year +40% CC 8640 minute summer	4.757	1.214
30 year +40% CC 8640 minute winter	3.070	1.214
30 year +40% CC 10080 minute summer	4.398	1.122
30 year +40% CC 10080 minute winter	2.838	1.122
100 year +45% CC 15 minute summer	481.390	136.217
100 year +45% CC 15 minute winter	337.817	136.217
100 year +45% CC 30 minute summer	325.547	92.119
100 year +45% CC 30 minute winter	228.454	92.119
100 year +45% CC 60 minute summer	225.292	59.538
100 year +45% CC 60 minute winter	149.679	59.538
100 year +45% CC 120 minute summer	129.929	34.337
100 year +45% CC 120 minute winter	86.322	34.337
100 year +45% CC 180 minute summer	97.037	24.971
100 year +45% CC 180 minute winter	63.077	24.971
100 year +45% CC 240 minute summer	75.541	19.963
100 year +45% CC 240 minute winter	50.188	19.963
100 year +45% CC 360 minute summer	56.816	14.621
100 year +45% CC 360 minute winter	36.932	14.621
100 year +45% CC 480 minute summer	44.469	11.752
100 year +45% CC 480 minute winter	29.544	11.752
100 year +45% CC 600 minute summer	36.313	9.932
100 year +45% CC 600 minute winter	24.811	9.932
100 year +45% CC 720 minute summer	32.325	8.664
100 year +45% CC 720 minute winter	21.725	8.664
100 year +45% CC 960 minute summer	26.548	6.991
100 year +45% CC 960 minute winter	17.586	6.991
100 year +45% CC 1440 minute summer	19.298	5.172
100 year +45% CC 1440 minute winter	12.970	5.172
100 year +45% CC 2160 minute summer	13.815	3.818
100 year +45% CC 2160 minute winter	9.519	3.818
100 year +45% CC 2880 minute summer	11.501	3.082
100 year +45% CC 2880 minute winter	7.729	3.082
100 year +45% CC 4320 minute summer	8.746	2.287
100 year +45% CC 4320 minute winter	5.760	2.287
100 year +45% CC 5760 minute summer	7.295	1.867
100 year +45% CC 5760 minute winter	4.721	1.867
100 year +45% CC 7200 minute summer	6.343	1.618
100 year +45% CC 7200 minute winter	4.094	1.618
100 year +45% CC 8640 minute summer	5.690	1.452
100 year +45% CC 8640 minute winter	3.672	1.452
100 year +45% CC 10080 minute summer	5.226	1.333
100 year +45% CC 10080 minute winter	3.373	1.333

Results for 2 year +40% CC Critical Storm Duration. Lowest mass balance: 99.98%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
2880 minute winter	1	2160	94.273	0.373	15.0	316.6934	0.0000	SURCHARGED
2880 minute winter	2	2160	94.273	0.373	9.3	248.4195	0.0000	SURCHARGED
120 minute summer	OUT	176	91.717	0.017	4.8	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
2880 minute winter	1	1	2	9.3	0.618	0.131	0.3521	
2880 minute winter	2	2	OUT	4.8	2.943	0.006	0.0081	612.1

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 99.98%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
2880 minute winter	1	2100	94.561	0.661	24.9	591.4716	0.0000	SURCHARGED
2880 minute winter	2	2100	94.560	0.660	13.5	467.5378	0.0000	SURCHARGED
2880 minute winter	OUT	2100	91.721	0.021	7.5	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
2880 minute winter	1	1	2	13.5	0.738	0.191	0.3521	
2880 minute winter	2	2	OUT	7.5	3.358	0.010	0.0112	815.9

Results for 100 year +45% CC Critical Storm Duration. Lowest mass balance: 99.98%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
1440 minute winter	1	1080	94.697	0.797	53.4	730.7480	0.0000	SURCHARGED
1440 minute winter	2	1110	94.695	0.795	26.4	578.7179	0.0000	SURCHARGED
1440 minute winter	OUT	1110	91.727	0.027	13.5	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
1440 minute winter	1	1	2	26.4	1.011	0.373	0.3521	
1440 minute winter	2	2	OUT	13.5	3.977	0.018	0.0170	759.4

Volume vs discharge Pond 1 & Basin 1

Appendix I SWW Correspondence

Peninsula House, Rydon Lane, Exeter, EX2 7HR
www.southwestwater.co.uk

Mr Tom Richards
Awcock Ward Partnership
Ada House
Pynes Hill
Exeter
Devon
EX2 5TU

Direct line: (01392) 443661
Planning Team: (01392) 442836
Our ref: WR 3810789/AB
Email: developerservicesplanning@southwestwater.co.uk

19th September 2023

Dear Mr Richards

Pre Planning: Point of connection enquiry – Provision of new public sewers

Proposal: Residential development of 100no new dwellings
Location: Tidcombe Hall, Tiverton, EX16 4EJ

Further to my letter dated 5th September 2023 regarding the Pre Planning Point of Connection Enquiry for the above proposal, I am now able to provide the following response.

The following has been based upon the information in your completed application form and accompanying correspondence. Therefore, should any of the information now be different, please ensure that you inform South West Water of any amendments to ensure the response is accurate.

Please note: The following information is a desk-top budget estimate to provide an approximation of the costs for the above proposed development. If you would like South West Water to provide a formal offer for any of the activities detailed in this letter, please forward the relevant application to Developer Services.

To download these applications and view associated timescales for these activities, please visit our website: www.southwestwater.co.uk/developers

The estimates provided are based on the New Connection and Developer Services – Charging Arrangements 2023-24 and is valid until 31 March 2024. For further information, please refer to the company's Charging Arrangements 2023-24 document. This can be located on our website: www.southwestwater.co.uk/developerservices.

The estimate has been split into sections for ease of use:

Section 1: Site specific charges waste water sewer requisitions

Section 2: Sewer connections

Section 3: Adoption of public sewers

Section 4: Infrastructure charges

Section 5: Income Offsets

Section 6: Environmental Incentives

Section 7: Surface Water Run-off Destination Hierarchy

Application forms and timescale for delivery of these processes can be found on our website at www.southwestwater.co.uk/developerservices.

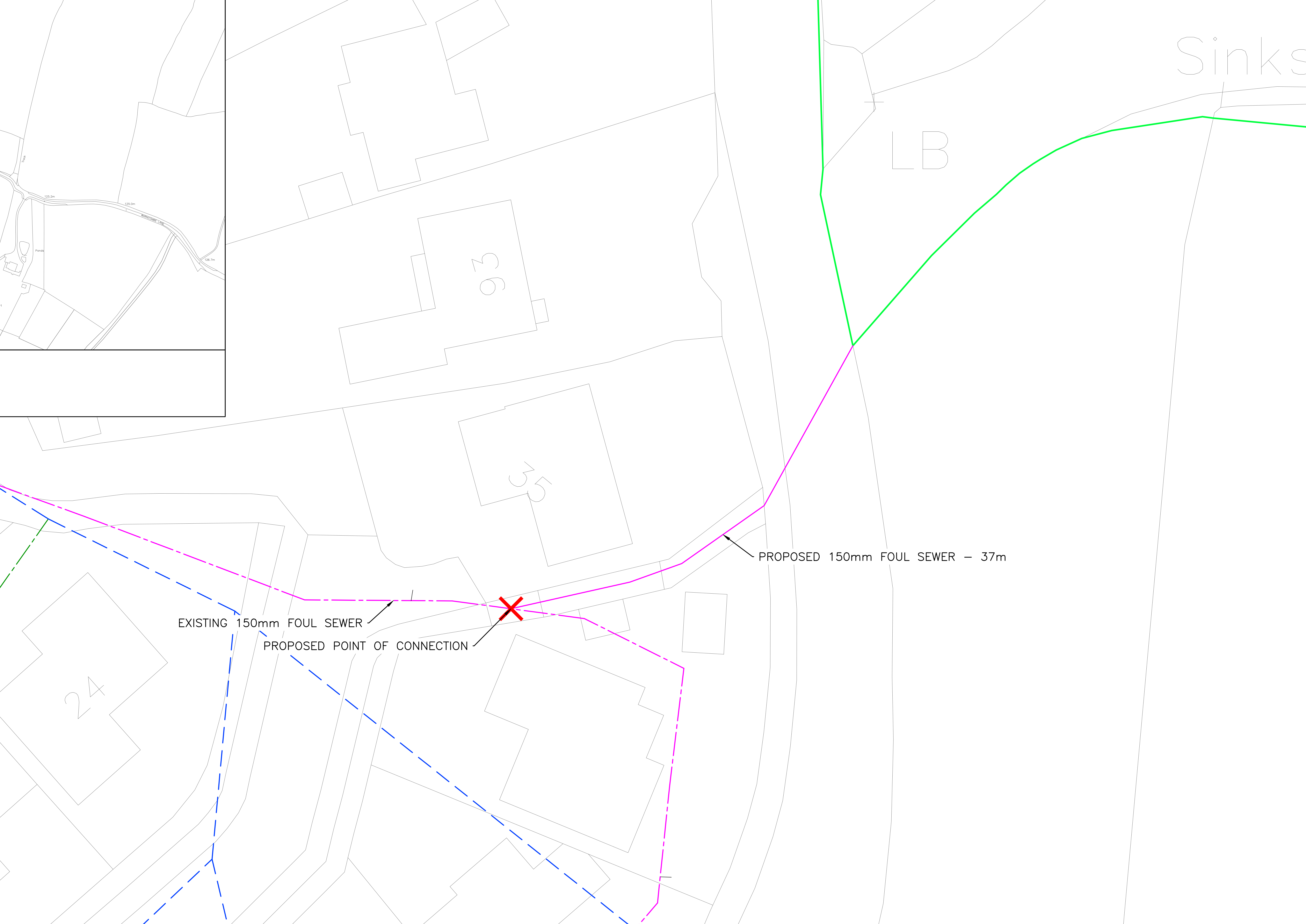
You can if you wish, use an alternative provider, i.e., another Undertaker to supply your site. This is known as New Appointees and Variations (NAV's). Further details of what a NAV is can be found at https://www.ofwat.gov.uk/regulated-companies/markets/nav-market
Details of how South West Water interact with a NAV can be found at - https://www.southwestwater.co.uk/developer-services/water-services-and-connections/using-a-nav/

I trust this provides the information required for the proposed development. However, if you have any questions or queries, please contact me on direct line: 01392 443661.

Alternatively, you can contact the Pre Development Team on 01392 442836 or via email: DeveloperServicesPlanning@southwestwater.co.uk.

Yours sincerely

Angie Brown
Pre Development Coordinator


Sinks
LB
36
35
24
EXISTING 150mm FOUL SEWER
PROPOSED POINT OF CONNECTION
PROPOSED 150mm FOUL SEWER – 37m