Preliminary Drainage Strategy

Project:

Proposed Residential Development, The Drove, Osbournby, Sleaford, Lincolnshire

Client:

Y6 Architectural

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Contents

Introduction

Woodside consulting engineers have been appointed Y6 Architectural, to undertake a drainage strategy, forming part of the supporting evidence of the planning submission for the proposed residential development, The Drove, Osbournby, Sleaford, Lincolnshire. The following document is to assist the planners and Lead Local Flood Authority (LLFA) in demonstrating that the development is acceptable in terms of sustainable drainage strategy and that it does not cause offsite flood risk as a result of the development.

Existing Site

The site lies to the south of The Drove, Osbournby, SLeaford, approximate postcode is NG34 0DH. An aerial location plan is shown in figure 1, please note that the red line is indicative and does not indicate the planning boundary.

Figure 1 - Site Location

Site Topography

A site survey has been included with the proposed site plan provide by Y6 Architectural, found on drawing: 019-005-02. The approximate total site area is 0.68 hectares.

The current site is greenfield. There is an existing access to the field off of The Drove, the is located to the north west corner, and also appears to act as substation access.

The general topography of the site shows a gradual fall from the north west to the south east corner, with an approximate gradient of 1 in 50. The level range is approximately between 20.20 to 18.17m AOD.

There is a substantial ditch to the south and lower eastern boundary. These ditches are typically over 1m deep and flow towards to south east corner. The west boundary backs on to residential property, with the boundary being defined with close boarded fencing. There is a small ditch to the northern boundary. This ditch is only around 300mm deep and appears to flow to the east, naturally with he road. It is assumed that the north ditch connects into the east, where it flows to the south east corner.

The existing road, The Drove, at the frontage of the site falls from west to east falls at an approximate gradient of 1 in 88. The road is cambered and edged with HB2 kerbs. The road appears to be drained via. offlet kerbs, which presumably discharge into the northern ditch. This area during the surveys and historic photographic evidence has always been overgrown, but the levels would suggest that these are connected, although possibly connected.

Ground Conditions

A phase 1 or 2 ground investigation report is not available for this site currently, and the findings of this section are based on publicly available information. A full ground investigation will be undertaken prior to detailed design and assessed by the appointed engineer.

The bedrock geology of the site is described as 'Kellaways Sand Member – Sandstone and Siltstone.' There are no records for superficial deposits. A previous drainage strategy was undertaken for this site approximately 6 years ago, it is understood that during that design, infiltration testing was undertaken and this proved to not support infiltration.

There is limited borehole information in close proximity of the site. Any ground water levels will need to be assessed at detailed design stage.

Both British Geological Survey definitions for the bedrock and superficial deposits can be found respectively in figures 2 and 3 below.

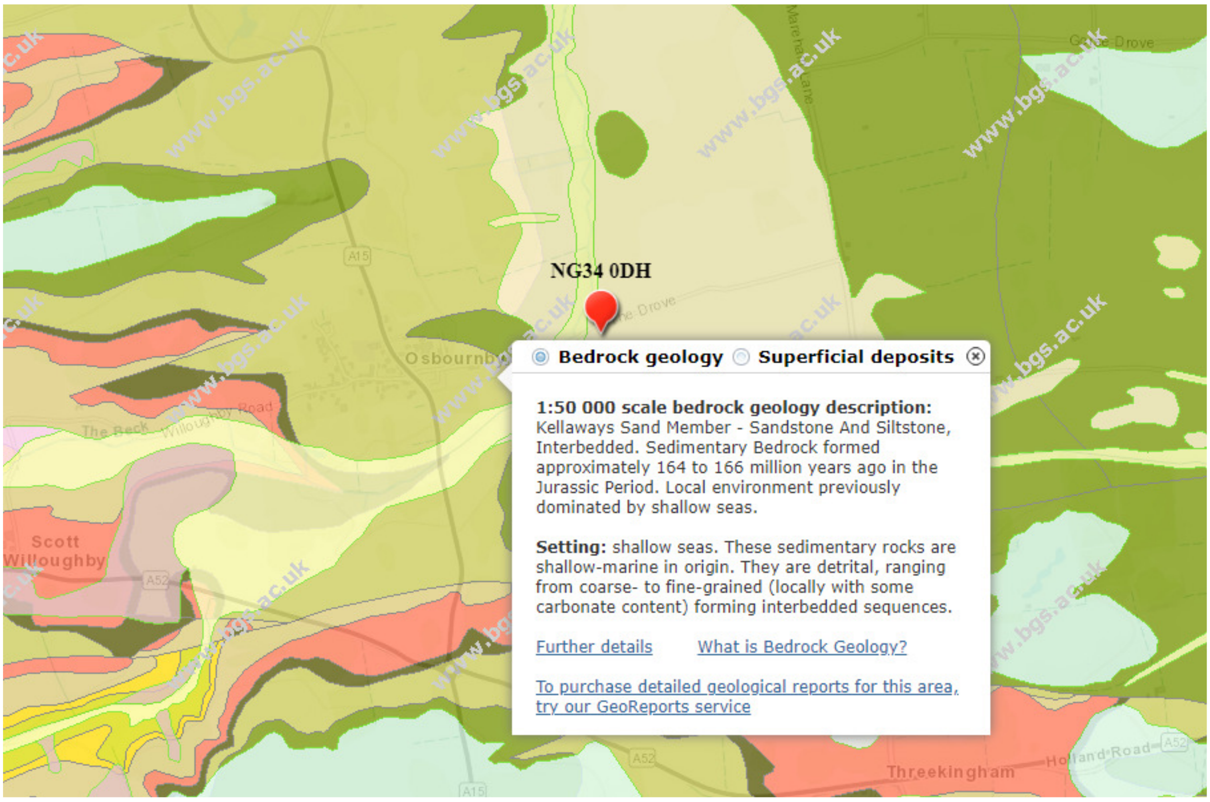


Figure 2 - BGS Bedrock Geology

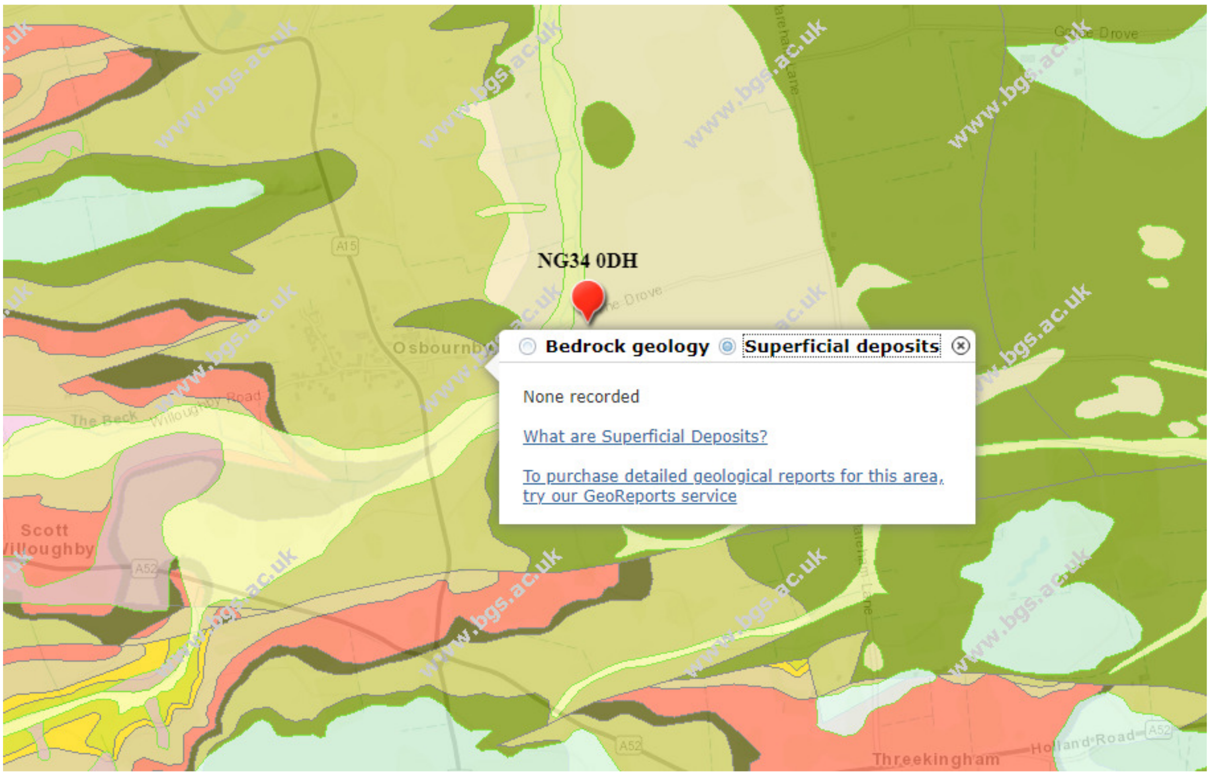
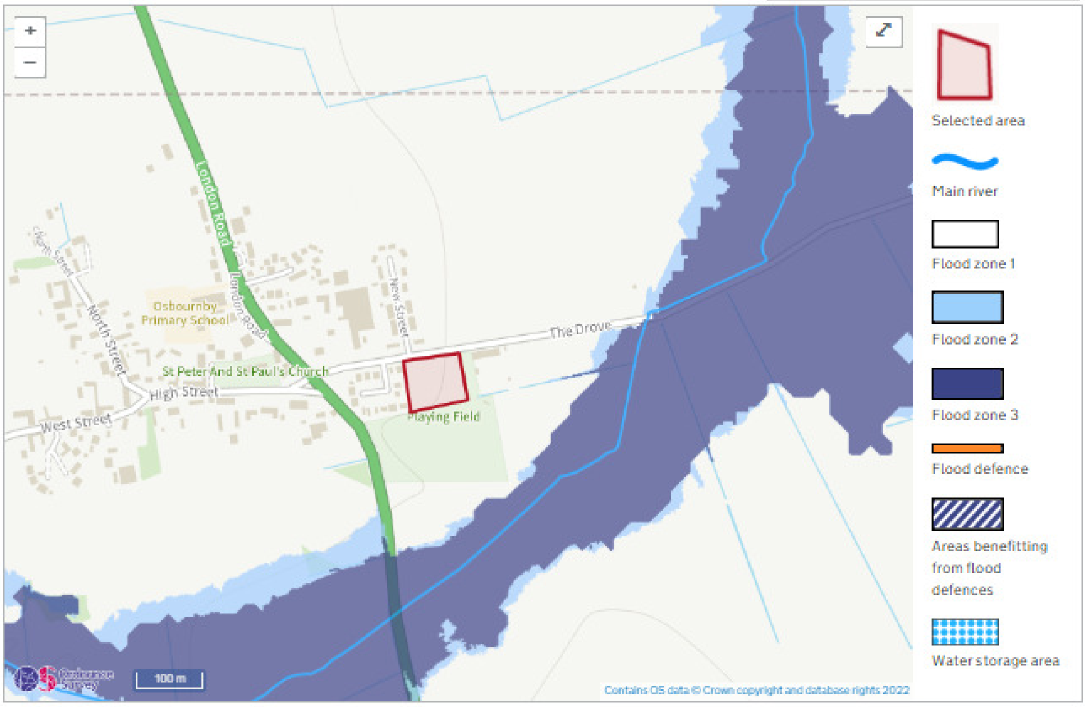


Figure 3 - BGS Superficial Deposits

Flood Risk

A basic check on the flood risk has been carried out, utilising the governments 'flood risk for planning' service, shown in the figure below. The criteria within the figure shows that the site is located in flood zone 1, the site is less than a hectare, and we are unaware of any critical drainage issues. Furthermore, the surface water flooding has been checked, and the site is a low risk of surface flooding – which would be deemed acceptable for this site. The EA and LLFA will notify of any specific issues within the site as part of the planning process. The full summary is found in appendix A.



You will need to do a flood risk assessment if your development is **any of the following:**

- bigger than 1 hectare (ha)
- in an area with critical drainage problems as notified by the Environment Agency
- identified as being at increased flood risk in future by the local authority's strategic flood risk assessment
- at risk from other sources of flooding (such as surface water or reservoirs) and its development would increase the vulnerability of its use (such as constructing an office on an undeveloped site or converting a shop to a dwelling)

If your development **does not** fall into any of these categories, you only need to download the flood map for planning on this page showing your flood zone to include in your planning application.

Figure 4 - Flood Map for Planning

Development Proposals

The development consists of the erection of 20 new residential dwellings and associated infrastructure. The dwellings are typically detached, although a semi-detached and terraced row are present to the north east of the site. Some of the dwellings have garages, but mostly just have drives. The indicative site plan is shown in the figure below.

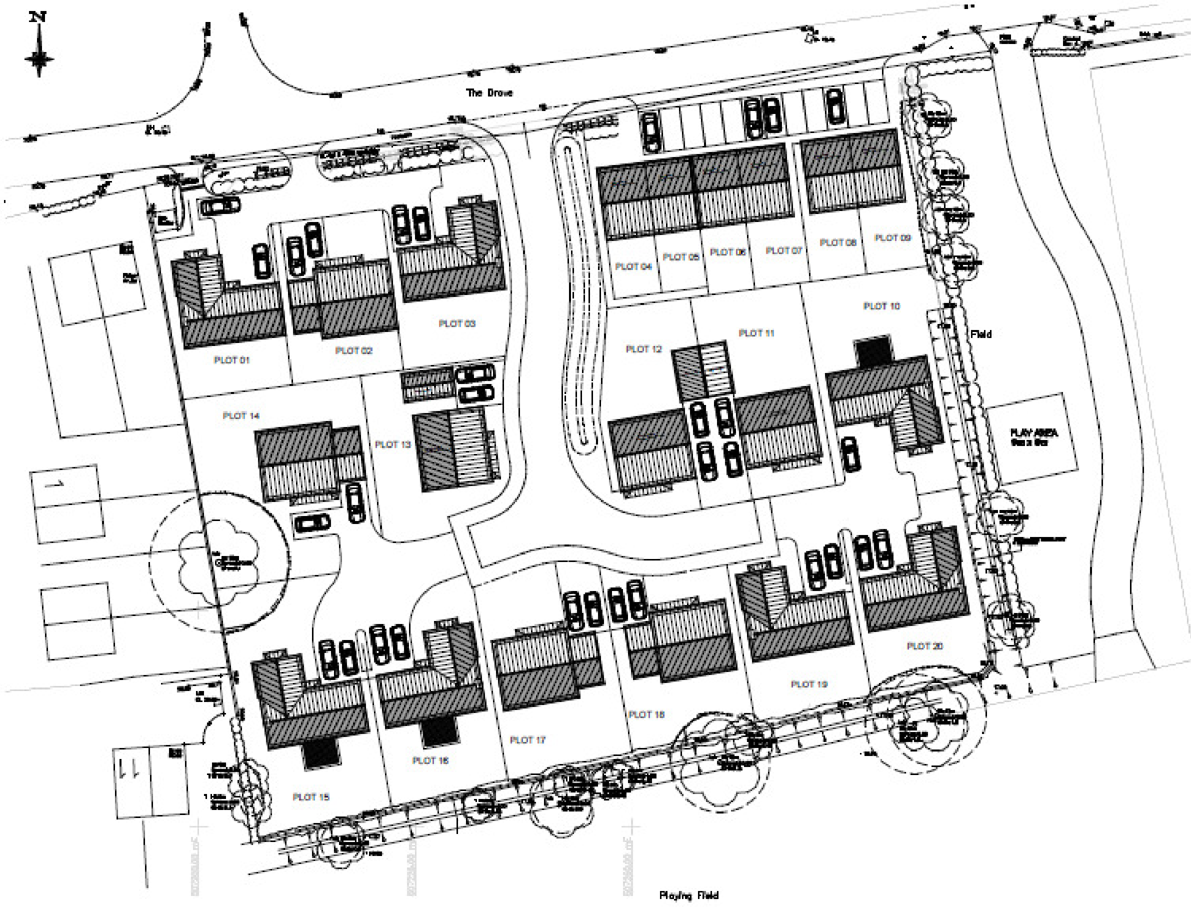


Figure 5 - Indicative Site Plan

Drainage Strategy

The following section provides narrative on the principles behind the drainage strategy and has been carried out in general accordance with Lincolnshire County Council's "Sustainable Drainage Design and Evaluation Guide" (SDDEG), and "CIRIA's C753 – the SUDS Manual", where appropriate.

Surface Water Drainage

For new developments there is a requirement to apply sustainable drainage principles (SuDS) to the disposal of surface water from the site where practicable. As required by Building Regulations and Defra's "Non-statutory Technical Standards for Sustainable Drainage Systems" (NTS), surface water must discharge to the following, listed in priority.

1. To ground – in an adequate soakaway or some other adequate infiltration system.
2. To a watercourse.
3. To a surface water sewer, highway drain or other drainage system.
4. To a combined sewer.

Infiltration

At present infiltration testing has not been undertaken, but it is understood that historic testing was undertaken at this site – this proved that infiltration was not a viable solution. Furthermore, the BGS map descriptions support this, and at this stage it is assumed that infiltration is not viable.

At detailed design, when a phase 2 ground investigation report is undertaken, the use of infiltration will be rechecked, and if it is deemed viable then the strategy will be reviewed.

Watercourse

As previously discussed, the current site has a drainage ditch to the southern and eastern boundary, and given the natural falls of the site, this would receive the entire greenfield run off from the current field. There is a small watercourse to the south of the adopted road, which appears to cater for the highway.

The overall area is part of the Black Sluice IDB's extended district, and it is expected that they will require a contribution and byelaw consent, as the water will ultimately reach their system. Although the site is in the maps extended district, it is close to their full district, as shown in the figure below.

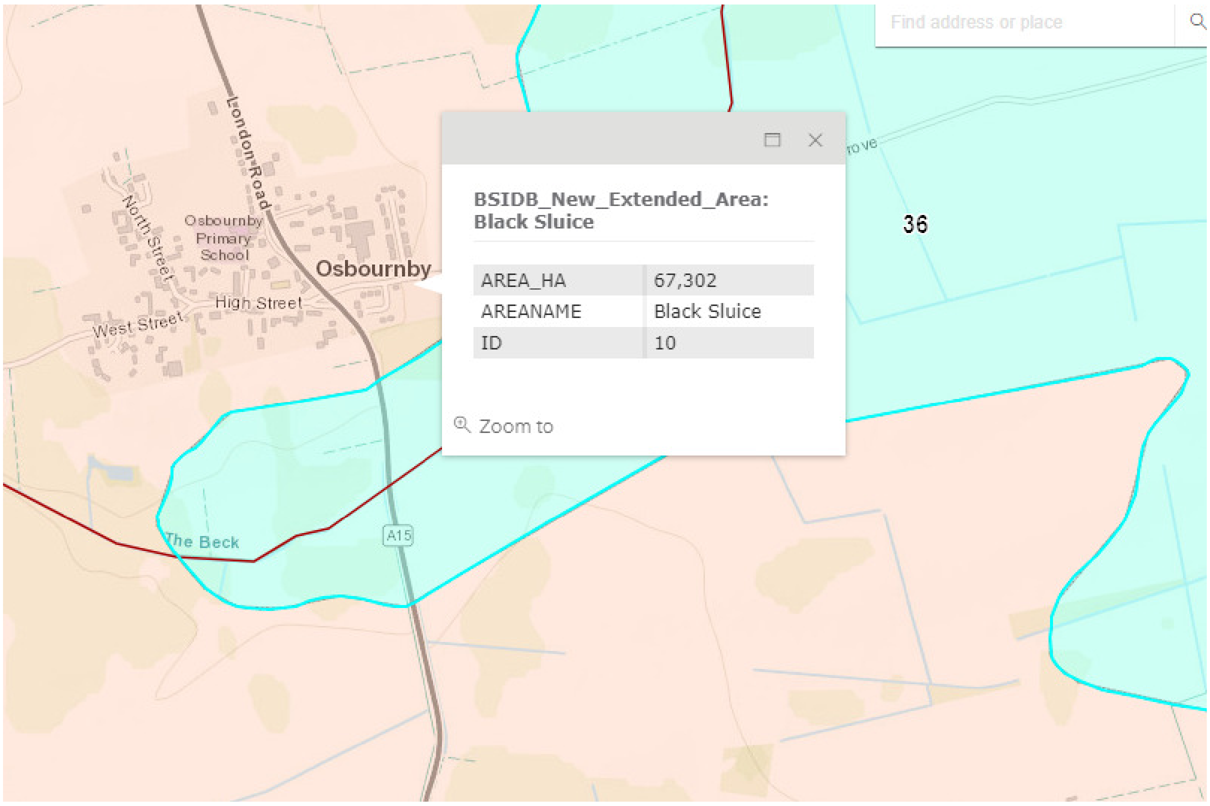


Figure 6 - IDB District

To the south of the site, the IDB's catchment is known as Scredington, ID 36. The red line running through the figure above shows that this is an EA Main River, known as 'The Beck'. There do not appear to be any IDB maintained watercourses leading into the EA river from the site. As the discharge into the EA river will be through riparian drains, the IDB may have a more relaxed approach to the runoff, although this will be determined through byelaw consent.

One item that is not definitively resolved is the connectivity of the ditch into The Beck, this is due to the overgrown nature of the site. The figure below shows the watercourses picked up from aerial views. The topo shows that there is a watercourse to the south and east of the site. The aerial views show that there is a watercourse to east of the southeast corner of the site, which then connects into the Beck. There is a unknown gap of approximately 80m. Given the size of the drain to the south of the site, it is assumed that there is a link between these two areas, it could be a heavily overgrown watercourse, or a culvert.

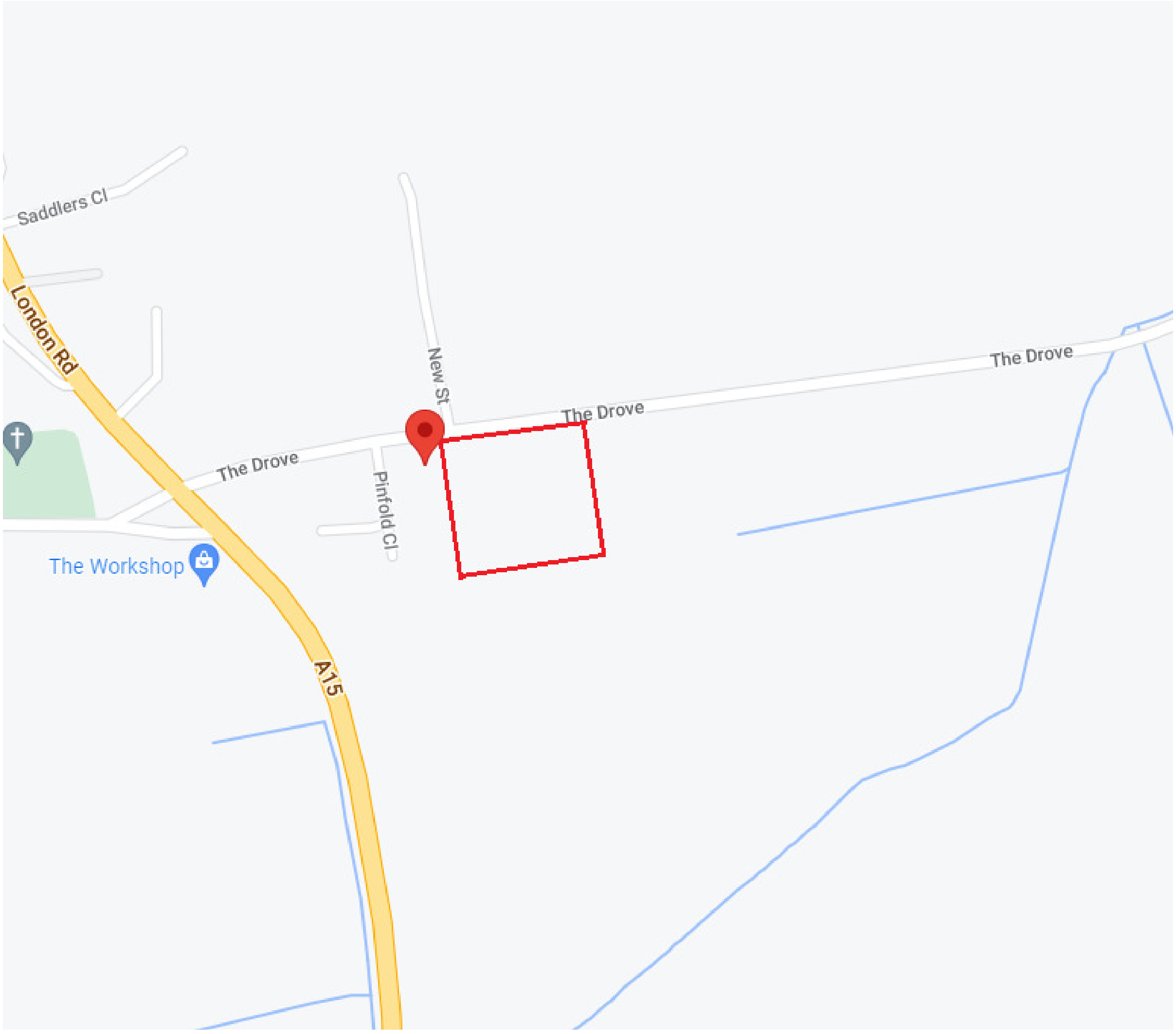


Figure 7 - Local Watercourses

Although the watercourse connectivity is not fully known, it is assumed, given the sizes and topography, that there is a link into The Beck. If there is not a link between these ditches, then given the relatively short distances involved, it is reasonable to assume that an agreement with the landowner can be reached to provide a connection. It is therefore proposed that the surface water from the site, will discharge into the watercourse towards the south east corner.

Additionally, as shown in the upcoming figure, there is an existing Anglian Water sewer and headwall entering the southern watercourse, upstream of our site. This would generally support that there is overall connectivity into the ditch and subsequent main river.

Surface Water Sewer

The Anglian water asset maps have been obtained for the development, as shown in the figure below, with the full asset map provided in appendix B. The map shows that there is a surface water sewer discharge into the southern watercourse – to the west of the site. Foul water being available in The Drove. As a viable connection into the watercourse has been established, then this method of surface water disposal does not need to be considered. Lower hierarchy methods of disposal will not be considered.

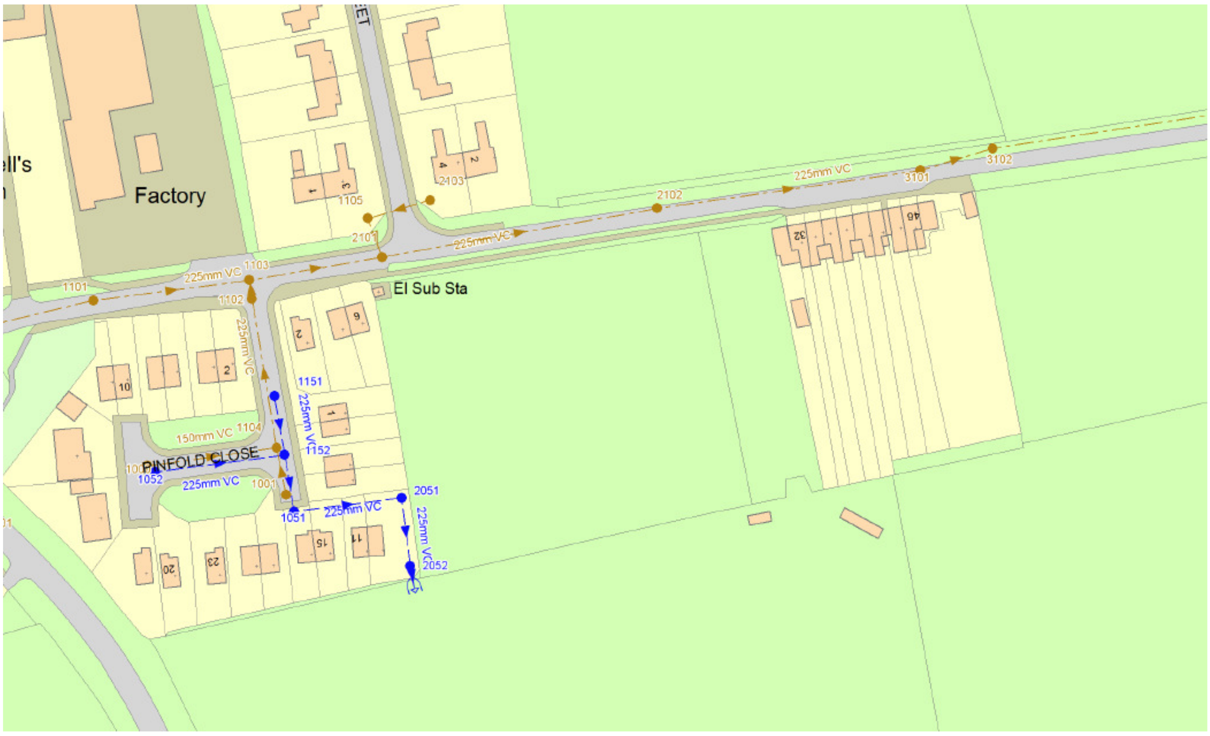


Figure 8 - Anglian Water Asset Map

Run-off Rates

Existing Drainage Arrangements

As the site is predominantly greenfield it has no current surface water drainage serving it. The natural gradient of the site falls towards the south watercourse and any greenfield flows will flow to that point.

Existing greenfield run-off Rate

The greenfield run-off for the site is summarised in the figure below, with a full copy of the report provided in appendix C. HR Wallignford method uses all site areas with the exception of large public open space. In this instance there is no PoS, but the gardens of the southern plots will discharge directly into the existing watercourse, and therefore this area has been discounted. The total contributing greenfield area is 5890m^2, which has been used in the calculation. The HR Wallingford Greenfield runoff rate estimation for sites is an industry standard method to determine the greenfield run-off. The above calculation uses the IH124 method to determine the runoff.

Greenfield runoff rates	Default	Edited
Q_{BAR} (l/s):	2.35	2.35
1 in 1 year (l/s):	2.04	2.04
1 in 30 years (l/s):	5.75	5.75
1 in 100 year (l/s):	8.35	8.35
1 in 200 years (l/s):	9.88	9.88

Figure 9 - Existing Greenfield Runoff Rates

The Q_{BAR} greenfield run-off for the site is calculated at 2.35 l/s, with the 1 in 1, 30 and 100 year events having a runoff of 2.04, 5.75 and 8.35 l/s respectively.

The LLFA have two approaches to greenfield run-off, the 1st is based on a variable discharge – matching the like for like greenfield storms. The second is to allow the post development runoff to have a peak discharge of Q_{BAR}. As the site is constrained, the attenuation space is limited, therefore it is proposed that approach 1 is used – matching the greenfield runoff.

Existing greenfield run-off Volume

The greenfield run-off for the site, based on an area of 5890m^2, is 183m^3, as shown in the figure below. The guidance set out in CIRIA 753 is that the post development runoff rate should not exceed the pre development runoff rate, for the 1 in 100 year 360 minute duration storm – this will be checked later in the document.

Pre-development discharge	
Site Makeup	Greenfield
Greenfield Method	FSR/FEH
Positively Drained Area (ha)	0.589
Soil Index	1
SPR	0.47
Catchment Wetness Index	120.000
Return Period (years)	100
Climate Change (%)	0
Storm Duration (mins)	360
Betterment (%)	0
	Calc
PR	0.497
Runoff Volume (m³)	183

Figure 10 - Pre-development Greenfield Runoff

Drainage Proposals

The ethos for the drainage design is to try and mimic natural drainage as far as possible, and to ensure that as a result of the development, flood risk offsite is not increased. The following section demonstrates how this will be achieved and outlines the general drainage strategy.

The proposed drainage layout is shown on Woodside drawing 20474-SK-3000, which can be found in appendix D, with the calculations, 20474-CAL-3000, being provided in appendix E.

Infiltration

As previously discussed, infiltration potential is not viable, this is also backed up by the greenfield runoff calculator's soil type being 4 (other than bedrock, this is the lowest permeability soil type), and therefore is discounted for the site.

Watercourses

Although the final connectivity of the riparian ditches is not known, it is reasonable to assume that the existing ditch on the site connects into the overall watercourse network, and into the EA main river which is approximately 250m to the east.

Although the natural greenfield runoff from the site is up to 8.35l/s, this exceeds the IDB's pumped catchment rate of 1.4l/s/ha. As the site is outside of the IDB's immediate catchment, they may have no concerns with this, subject to a nominal processing fee. If they deem it to need to fully comply with byelaw consent, then a development contribution will be payable. The proposals are such that the post-development discharge rates do not increase the pre-development greenfield rates and therefore no additional flood risk is being created from the development of the site.

Surface Water Conveyance and Attenuation

The ethos of the site is to comply with the SUDS management train as far as practical, although this is limited by the impermeable nature of the soil.

Given the scale of the development, it is proposed that both the foul and surface within the main road are to be adopted by Anglian Water under a section 104 agreement – constructed to the latest DCG. In order to satisfy this, it is proposed that the main adopted S104 surface sewer is sized at 600mm dia., this ensures that they system does not surcharge for the 1 in 2 year event, which satisfies the AW requirements.

To provide source control and attenuation, it is proposed that the shared drives are to be constructed using porous paving. The storage stone in these areas will be 600mm thick, will be wrapped in an impermeable membrane, and will be restricted by an orifice flow control device. The shared drives will be placed into a management company.

The individual domestic drives will be laid to porous paving which will feed into the main sewer. The plot drainage will also discharge into the domestic drives, but these smaller areas will not be attenuated, and discharge unrestricted into the main sewer, although this acts as source control.

As space on the site is limited, there is still a need for offline attenuation tanks, these are located in private plots. These have been placed in areas which are highly unlikely to have future development and will be put into a management company. The sizes of the attenuation tanks are shown on the drainage general arrangement.

The discharge from the site will be controlled using multiple hydrobrakes, this allows the discharge form he site to reflect the different flow rates from the comparative storms – this is known as a complex control. It is proposed that the flow control will be adopted by Anglian water. Part of the S104 drainage run is located outside of the adopted road, and will be subject to an easement.

The discharge from the flow control chamber will flow directly into the riparian ditch, where an insitu concrete headwall will be built.

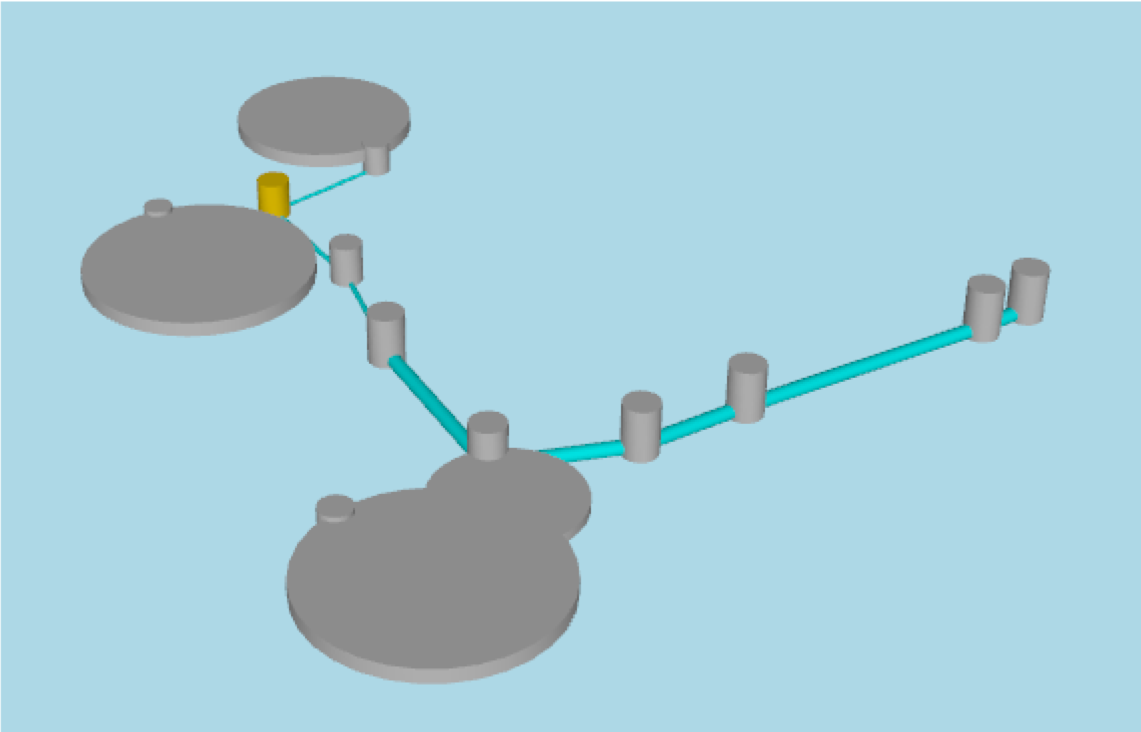


Figure 11 - 3D View of Drainage Model

There is an existing watercourse to the front of the site which appears to currently serve the highway drainage, with the carriageway draining into kerb outlets, which then appear to connect into the shallow ditch. It is assumed that this ditch is fully within the site's land, and therefore will be private or riparian. As a result of the development, part of this ditch will need to be partly culverted. The topo indicates that this ditch is small in size, and therefore, replacing its capacity should not be an issue. It is currently assumed that the existing north ditch connect to the eastern ditch, although the topo is not conclusive (due to growth). To ensure connectivity, the proposed culvert will be linked into the eastern culvert, which ultimately discharges to the southeast corner. Any works to the north watercourse will ensure that any lost volume/capacity is compensated for.

The site has been designed to incorporate source control as far as reasonably practical, and it is our opinion that SUDS has been fully considered for this type of development, meeting all of the appropriate standards.

Surcharged Outfall

During storm events, it is unknown if the water in the existing watercourse network will surcharge the outfall device, which needs to be considered. The ditch levels have a substantial fall, and appear to be located at the head of the system. At this stage of the design, it is reasonable to assume that the outfall will not be surcharged. Once the final connectivity of the overall watercourse network is established, the surcharged outfall criteria will be reassessed.

Post development Runoff Rates

The post development flow rates, against the existing greenfield flow rates are summarised in the figure below. As identified, none of the post development flow exceeds the predevelopment flow for the like for like storm – this is in accordance with the LLFA's policy, in particular approach 1.

Return Period	Pre-development Flow	Post-development Flow
1 Year	2.04 l/s	2.00 l/s
30 Year	5.75 l/s	5.20 l/s
100 Year	8.35 l/s	8.30 l/s (40% CC)

Post development Runoff Volume

The post development, 100 year, 360 minute storm discharge volume is 100.1m^3, which is 54% of the predevelopment flow, and therefore a significant betterment is being created.

Finished Floor Levels

As the site is in flood zone 1, and no significant risks of flooding have been identified, it is proposed that the finished floor levels are a minimum of 150mm above ground levels. Once a full topographical survey of the site has been reviewed, detailed level design can be undertaken.

Exceedance Flows

There is always the possibility that a device can become temporarily blocked or fail, resulting in the system flooding. Additionally, the site has been designed up to and including storms of 1 in 100 year + 40% climate change, any storms greater than this will cause above ground flooding. The proposed flood routing in storms of exceedance or device failure can be found on the drainage drawing.

Although detailed level design is not known, the site does fall from north west to south east. At detailed design, the final levels will compliment the existing topography.

Water Quality

The methods of surface water disposal mentioned above have included provisions for water quality. In accordance with CIRIA C753, the pollution hazard features for the drainage areas are:

- Residential roofs - Very Low
- Individual Property Driveways - Low
- Shared Driveways - Low
- Low traffic roads - Low

To remove the pollution risks, CIRA have developed 'Pollution hazard indices' and the 'mitigation indices' that the SuDS components provide, further details of these are found in the figures below. This simple approach is considered suitable for this type of development.

Land use	Pollution hazard level	Total suspended solids (TSS)	Metals	Hydro-carbons
Residential roofs	Very low	0.2	0.2	0.05
Other roofs (typically commercial/ industrial roofs)	Low	0.3	0.2 (up to 0.8 where there is potential for metals to leach from the roof)	0.05
Individual property driveways, residential car parks, low traffic roads (eg cul de sacs, homezones and general access roads) and non-residential car parking with infrequent change (eg schools, offices) ie < 300 traffic movements/day	Low	0.5	0.4	0.4
Commercial yard and delivery areas, non-residential car parking with frequent change (eg hospitals, retail), all roads except low traffic roads and trunk roads/motorways[1]	Medium	0.7	0.6	0.7
Sites with heavy pollution (eg haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways[1]	High	0.8[2]	0.8[2]	0.9[2]

Figure 12 - Pollution hazard indices for different land use classifications

	Mitigation indices[1]		
Type of SuDS component	TSS	Metals	Hydrocarbons
Filter strip	0.4	0.4	0.5
Filter drain	0.4[2]	0.4	0.4
Swale	0.5	0.6	0.6
Bioretention system	0.8	0.8	0.8
Permeable pavement	0.7	0.6	0.7
Detention basin	0.5	0.5	0.6
Pond[4]	0.7[3]	0.7	0.5
Wetland	0.8[3]	0.8	0.8
Proprietary treatment systems[5,6]	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.		

Figure 13 - Indicative SuDS mitigation indices for discharging to a surface water

To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each contaminant type) that equals or exceeds the pollution hazard index (for each contaminant type):

Total SuDS mitigation index ≥ pollution hazard index
(for each contaminant type) (for each contaminant type)

Where the mitigation index of an individual component is insufficient, two components (or more) in series will be required where:

Total SuDS mitigation index = mitigation index$_1$ + 0.5 (mitigation index$_2$)

Where:

Mitigation index$_n$ = mitigation index for component n

Provided the total SuDS mitigation index exceeds the pollution hazard indices, then sufficient water quality will be provided.

For the calculations below the hazards are represented by;

Total suspended solids = Red

Metals = Blue

Hydrocarbons = Green

The figures are presented to show the actual mitigation achieved in the right-hand side column. The right-hand side's total must be higher than the left-hand side.

Roof areas

The roof water enters the below ground pipework directly, before passing into a swale, before ultimately discharging into the attenuation pond. The mitigation for this area is:

0.2 0.2 0.05 = Porous Paving Sub-base* 0.35 0.3 0.35

*the mitigation index is halved as it enters the sub-base directly.

The mitigation provided by the filter strip alone create the required mitigation indices, and therefore the mitigation indices requirements has been substantially exceeded.

Individual property driveways

0.5 0.4 0.4 = Permeable Pavement 0.5 0.5 0.6

The mitigation provided by the permeable paving alone satisfies the mitigation index, therefore sufficient treatment has been provided.

Shared Driveways

0.5 0.4 0.4 = Permeable Pavement 0.5 0.5 0.6

The mitigation provided by the permeable paving alone satisfies the mitigation index, therefore sufficient treatment has been provided.

Adopted Road

0.5 0.4 0.4 = Filter Strip 0.4 0.4 0.5 + ½ Swale 0.5 0.6 0.6 – Total 0.65 0.7 0.7

The mitigation provided by the filter strip and swale provides the required water quality treatment required.

In summary all the methods above provide enough water quality in accordance with CIRIA's C753 requirements.

Flood Risk off-site

As the flow from site will be restricted to greenfield runoff rates, the flood risk offsite has not been increased as a result of the development. Any land level increases are minimal and will still create a flat surface, which will not generate high flows offsite.

Management/Maintenance

It is crucial that the elements mentioned in the drainage elements and water quality are maintained to a sufficient standard to ensure that the devices can still function as designed. Generally, the maintenance requirements are either from CIRIA 753, or manufacturer guidance. It is currently assumed that the site owners management team will maintain the SUDS devices. This can form a condition if the planning application is acceptable.

The devices outlined below are preliminary only and subject to detailed design.

Filter Strip

Operation and maintenance requirements for filter strips

Maintenance schedule	Required action	Typical frequency
Regular maintenance	Remove litter and debris	Monthly (or as required)
	Cut the grass – to retain grass height within specified design range	Monthly (during growing season), or as required
	Manage other vegetation and remove nuisance plants	Monthly (at start, then as required)
	Inspect filter strip surface to identify evidence of erosion, poor vegetation growth, compaction, ponding, sedimentation and contamination (eg oils)	Monthly (at start, then half yearly)
	Check flow spreader and filter strip surface for even gradients	Monthly (at start, then half yearly)
	Inspect gravel flow spreader upstream of filter strip for clogging	Monthly (at start, then half yearly)
	Inspect silt accumulation rates and establish appropriate removal frequencies	Monthly (at start, then half yearly)
Occasional maintenance	Reseed areas of poor vegetation growth; alter plant types to better suit conditions, if required	As required or if bare soil is exposed over > 10% of the filter strip area.
Remedial actions	Repair erosion or other damage by re-turfing or reseeding	As required
	Relevel uneven surfaces and reinstate design levels	As required
	Scarify and spike topsoil layer to improve infiltration performance, break up silt deposits and prevent compaction of the soil surface	As required
	Remove build-up of sediment on upstream gravel trench, flow spreader or at top of filter strip	As required
	Remove and dispose of oils or petrol residues using safe standard practices	As required

Swales

Maintenance schedule	Required action	Typical frequency
Regular maintenance	Remove litter and debris	Monthly, or as required
	Cut grass – to retain grass height within specified design range	Monthly (during growing season), or as required
	Manage other vegetation and remove nuisance plants	Monthly at start, then as required
	Inspect inlets, outlets and overflows for blockages, and clear if required	Monthly
	Inspect infiltration surfaces for ponding, compaction, silt accumulation, record areas where water is ponding for > 48 hours	Monthly, or when required
	Inspect vegetation coverage	Monthly for 6 months, quarterly for 2 years, then half yearly
	Inspect inlets and facility surface for silt accumulation, establish appropriate silt removal frequencies	Half yearly
Occasional maintenance	Reseed areas of poor vegetation growth, alter plant types to better suit conditions, if required	As required or if bare soil is exposed over 10% or more of the swale treatment area
Remedial actions	Repair erosion or other damage by re-turfing or reseeding	As required
	Relevel uneven surfaces and reinstate design levels	As required
	Scarify and spike topsoil layer to improve infiltration performance, break up silt deposits and prevent compaction of the soil surface	As required
	Remove build-up of sediment on upstream gravel trench, flow spreader or at top of filter strip	As required
	Remove and dispose of oils or petrol residues using safe standard practices	As required

Hydrobrake Maintenance

Maintenance

Normally, little maintenance is required as there are no moving parts within the Flow Control. Experience has shown that if blockages occur they do so at the intake, and the cause on such occasions has been due to a lack of attention to engineering detail such as approach velocities being too low, inadequate benching, or the use of units below the minimum recommended size. The Flow Control (where applicable) is fitted with a pivoting bypass door, which allows the manhole chamber to be drained down should blockage occur. The smaller conical units, below the minimum recommended size, are also supplied with rodding facilities or vortex suppressor pipes as standard.

Following installation of the Flow Control it is vitally important that any extraneous material i.e. building materials are removed from the unit and the chamber. After the system is made live, and assuming that the chamber design is satisfactory, it is recommended that each unit be inspected monthly for three months and thereafter at six monthly intervals with hose down if required. If problems are experienced, please do not hesitate to contact the company so that an investigation may be made.

All Flow Control units are typically manufactured from grade 304 Stainless Steel, and if required they can also be manufactured in grade 316 Stainless Steel. Both materials have an estimated life span in excess of the design life of drainage systems.

The sediment within the catchpit of the Hydrobrake is to be monitored at the same time as the Hydrobrake inspections (every 6 months) and the silt is to be removed as necessary.

Attenuation Crates

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

Catchpits

Catchpits are utilised to help prevent the ingress of heavy sediment and other debris from entering the system. Maintenance requirements are low, and it is recommended that catchpits are inspected every six months and any build-up of sediment removed.

Pipework

If sediment in the catchpits are above the incoming pipes, or if performance of the site is hampered, then the pipes are to be inspect and jetted as necessary. The condition of the pipes shall generally be checked at the catchpit inspections.

Foul Proposals

Unlike surface water drainage, the preference for foul water disposal is to connect into a sewer, and only where this is not a viable option should other means of drainage be considered.

Connection to a Sewer

Extracts of the Anglian Water asset maps, local to the site are shown in the figure below. As demonstrated, there is a 225 dia. pipe located directly to the front of the site. Given the size of the pipe, and the relatively small size of the development, it is not expected that capacity will be an issue. It will be proposed that a new foul manhole will be constructed on the existing run. The main foul drainage within the site will be designed to adoptable standards, and entered into a S104 agreement, for adoption by Anglina Water. The levels of the Anglian Water sewers are not known, and at present it is assumed the scheme will work under gravity. Once the levels have been obtained, the design will be reviewed and the use of a pump may be required.

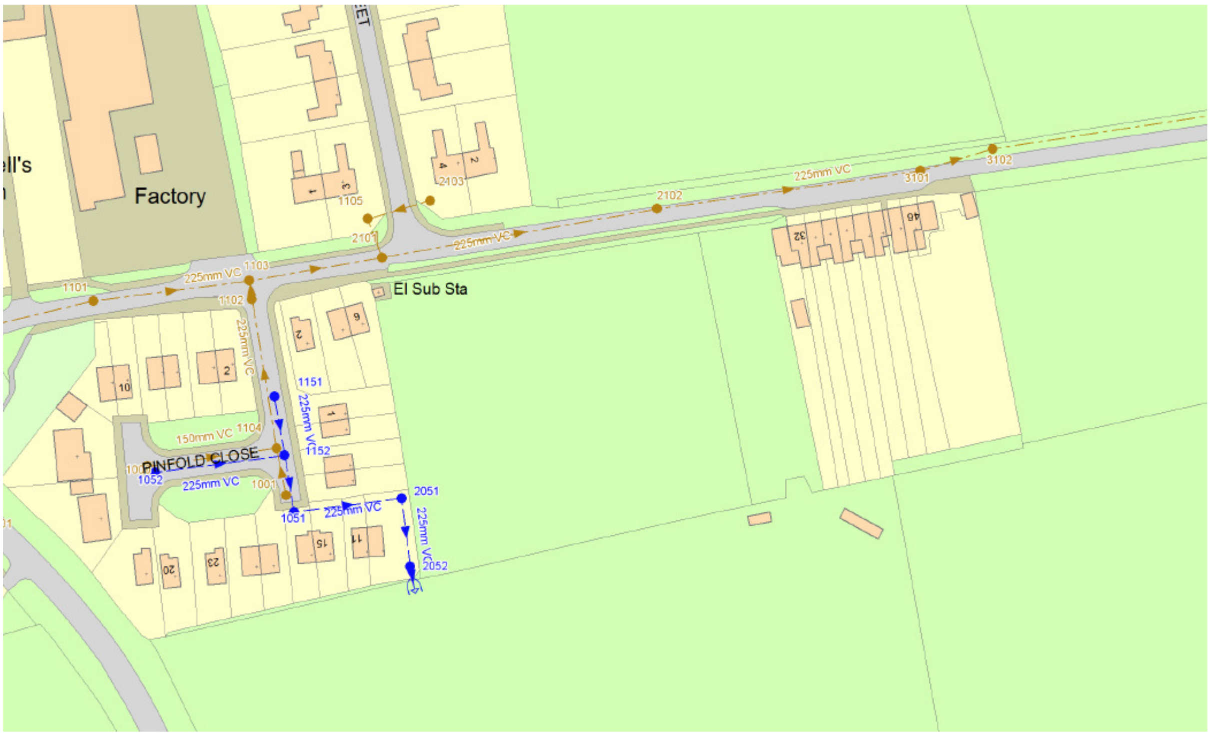


Figure 14 - Foul Connection into The Drove

Conclusion

- The above drainage strategy demonstrates that the site can be drained through the use of a Sustainable Drainage System at an appropriate level for the development.
- The flow offsite will be restricted to the like for like greenfield storm – in accordance with the LLFA's policies.
- Calculations demonstrate that all storms up to and including the 1 in 100 year + 40% climate change event are stored below ground.
- The proposed scheme does not increase the risk of flooding either on or off site as a result of the development.
- A full maintenance strategy will be developed at the detailed design stage.
- A preliminary drainage layout can be found on Woodsides's Preliminary Drainage Strategy drawing: 20474-SK-3000.
- The Black Sluice IDB will be consulted at the detailed application, and byelaw consent applied for if necessary.
- Finished floor levels will be raised a minimum of 150mm above ground levels (other than at level thresholds).
- The foul water will discharge into the existing adopted sewer in The Drove.

Appendices

Appendix A – Flood Map for Planning

Environment Agency

Flood map for planning

Your reference	Location (easting/northing)	Created
Osbournby	**507246/338115**	**29 Jul 2022 18:27**

Your selected location is in flood zone 1, an area with a low probability of flooding.

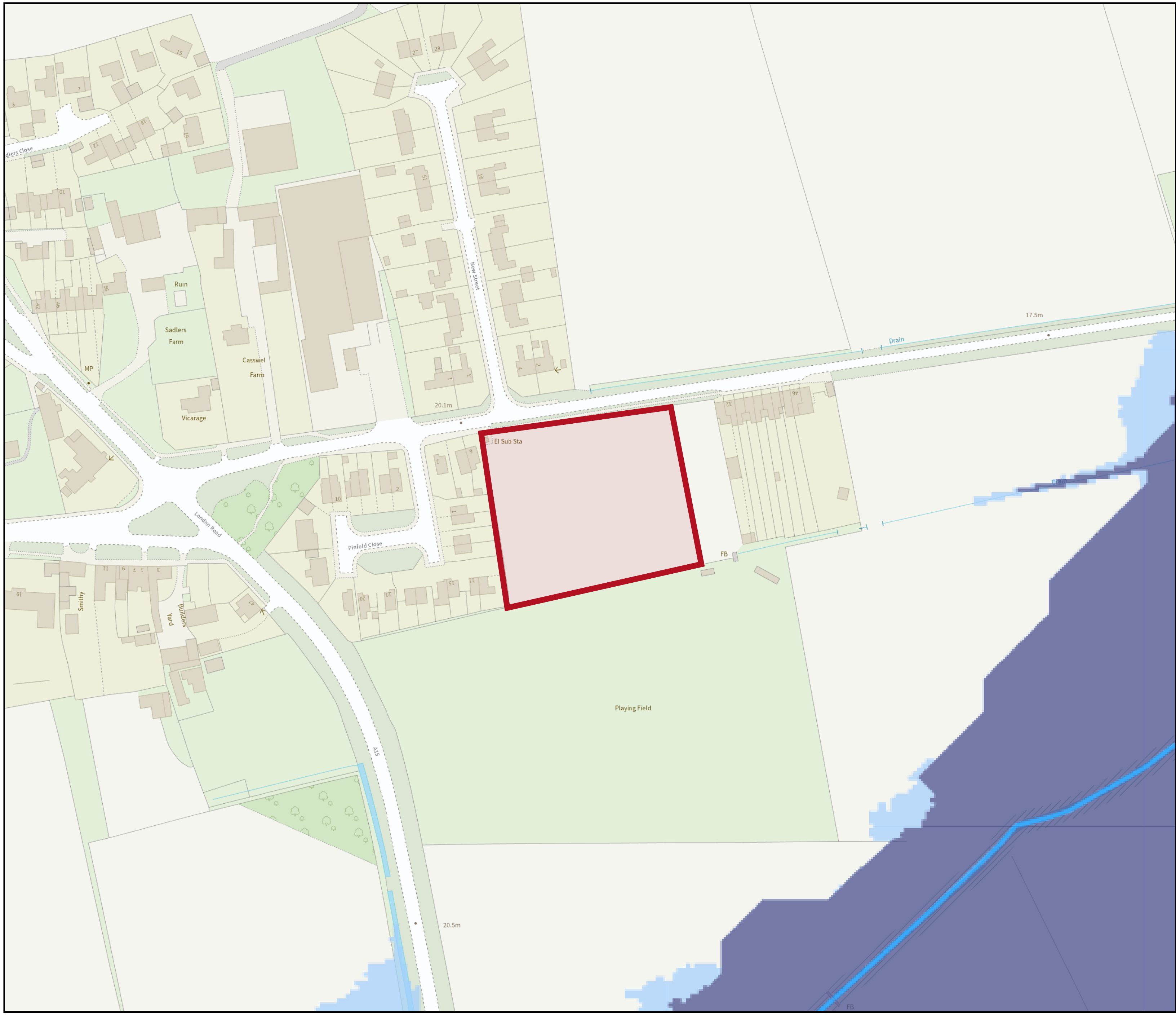
You will need to do a flood risk assessment if your site is **any of the following:**

- bigger that 1 hectare (ha)
- In an area with critical drainage problems as notified by the Environment Agency
- identified as being at increased flood risk in future by the local authority’s strategic flood risk assessment
- at risk from other sources of flooding (such as surface water or reservoirs) and its development would increase the vulnerability of its use (such as constructing an office on an undeveloped site or converting a shop to a dwelling)

Notes

The flood map for planning shows river and sea flooding data only. It doesn’t include other sources of flooding. It is for use in development planning and flood risk assessments.

This information relates to the selected location and is not specific to any property within it. The map is updated regularly and is correct at the time of printing.



Environment Agency

Flood map for planning

Your reference
Osbournby

Location (easting/northing)
507246/338115

Scale
1:2500

Created
29 Jul 2022 18:27

- Selected area
- Flood zone 3
- Flood zone 3: areas benefitting from flood defences
- Flood zone 2
- Flood zone 1
- Flood defence
- Main river
- Water storage area

0 20 40 60m

Appendix B – Anglian Water Asset Map

(c) Crown copyright and database rights 2016 Ordnance Survey 100022432	Date: 15/02/16	Scale: 1:1000	Map Centre: 507255,338127	Data updated: 04/12/15	Our Ref: 175202 - 1	Wastewater Plan A3

Foul Sewer
Surface Sewer
Combined Sewer
Final Effluent
Rising Main
(Colour denotes effluent type)
Private Sewer
(Colour denotes effluent type)
Decommissioned Sewer
(Colour denotes effluent type)

Outfall
(Colour denotes effluent type)
Inlet
(Colour denotes effluent type)
Manhole
(Colour denotes effluent type)
Sewage Treatment Works
Pumping Station

admin@g30consulting.com

05-0024 - The Drove

love every drop anglianwater

Manhole Reference	Liquid Type	Cover Level	Invert Level	Depth to Invert
0001	F	-	-	0.53
0002	F	-	-	0.76
1000	F	21.019	19.879	1.14
1001	F	20.984	19.774	1.21
1101	F	-	-	1.3
1102	F	20.389	19.519	0.87
1103	F	20.359	19.119	1.24
1104	F	20.394	19.704	0.69
1105	F	-	-	-
2101	F	-	-	1.14
2102	F	-	-	0.91
2103	F	-	-	-
3101	F	-	-	0.91
3102	F	-	-	0.53
1051	S	21.029	18.689	2.34
1052	S	21.089	19.299	1.79
1151	S	20.709	18.879	1.83
1152	S	20.8	18.77	2.03
2051	S	20.849	18.549	2.3
2052	S	20.229	18.259	1.97

Manhole Reference	Liquid Type	Cover Level	Invert Level	Depth to Invert

Manhole Reference	Liquid Type	Cover Level	Invert Level	Depth to Invert

Appendix C – Greenfield Runoff Rates

Print | Close Report

HR Wallingford
Working with water

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Ben Jackson
Site name:	The Drove
Site location:	Osbournby

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

Site Details

Latitude:	52.92979° N
Longitude:	0.40606° W
Reference:	1031402798
Date:	Jul 27 2022 20:44

Runoff estimation approach IH124

Site characteristics

Total site area (ha): 0.589

Methodology

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

Soil characteristics

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

Hydrological characteristics

	Default	Edited
SAAR (mm):	594	594
Hydrological region:	5	5
Growth curve factor 1 year:	0.87	0.87
Growth curve factor 30 years:	2.45	2.45
Growth curve factor 100 years:	3.56	3.56
Growth curve factor 200 years:	4.21	4.21

Notes

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

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

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

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

(3) Is SPR/SPRHOST ≤ 0.3?

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

Greenfield runoff rates

	Default	Edited
Q_{BAR} (l/s):	2.35	2.35
1 in 1 year (l/s):	2.04	2.04
1 in 30 years (l/s):	5.75	5.75
1 in 100 year (l/s):	8.35	8.35
1 in 200 years (l/s):	9.88	9.88

Appendix D – Drainage Plan

Drainage Key

- Proposed adoptable surface drainage
- Proposed private surface drainage
- Proposed adoptable foul drainage
- Proposed private foul water sewer
- Existing Anglian Water surface sewer with references

Adoptable Swale

The main drainage for the highways. At present it is likely that this is only receiving highway water and will therefore be adopted by LCC highways.

The final arrangements and adoption bodies are likely to change, but the main ethos that the swale drains the highway will not.

Private Attenuation Tanks

Providing the main attenuation volumes for all storms up to and including the 1 in 100 year + 40% climate change event. Preliminary sizes are shown on the plan. To be fitted with a vent pipe, and placed in a private management company.

Private Permeable Domestic Drives

Acting as source control. To act as water quality only, although some natural evaporation, and attenuation will occur. These areas will be maintained by the property owner.

Private Permeable Shared Drives

Acting as source control. To act as water quality only, although some natural evaporation, and attenuation will occur. These areas will be maintained by a management company. Preliminary calculations show that a sub-base thickness of 600mm is required.

Exceedence route flows

Drawing must be printed in colour. This text will be red if a coloured drawing.

Notes:

1. All drawings to be read in conjunction with Structural Engineers Drawings.
2. If in doubt - ASK!
3. Any discrepancies are to be reported back to the engineer immediately.
4. All dimensions are in metres unless noted otherwise (pipe diameters are generally shown in milimetres, unless noted otherwise).
5. Drawings marked Preliminary, Information or Tender are for guidance/ approval only, i.e. NOT for Construction.
6. It is likely that existing services will be present within the existing site. Contractor to ensure all necessary precautions have been taken before any excavation takes place.
7. It is the contractors responsibility to locate existing services where they may be affected by proposed works and protect them as necessary and agreed by the utility provider.
8. Drawings to be read in conjunction with all relevant drawings.
9. Setting out to be in accordance with the Architectural plans.
10. All proprietary items to be installed in strict accordance with the manufacturers instructions and recommendations.
11. All works to be carried out in accordance wit the current British Standards, Codes for practice and Building Regulations.
12. Manhole covers are to be D400 in adoptable areas and the main access way, C250 for shared parking areas, B125 for private domestic car parking and A15 for soft landscaped and non-vehicle areas.
13. For private drainage, the pipework does not need concrete pipe protection if the following depths are exceeded:
    - 13.1. For PVC - 0.9m in vehicular areas or 0.6m in soft landscaping.
    - 13.2. For clayware or concrete - 1.2m in vehicular areas or 0.6m in soft landscaping.
14. Pipe materials to be the following;
    - 14.1. Clayware to be Class 120
    - 14.2. Concrete to be Class M
    - 14.3. Thermoplastic to have a nominal ring stiffness of SN4.
15. PPIC's deeper than 1.2m are to be fitted with a non-access cover.
16. Unused PPIC connections are to be installed with caps.
17. Excavations near foundations need to be carefully considered. Refer to building regulations, and consult with the structural engineer where appropriate.
18. No work is to be started until all necessary approvals have been obtained.
19. Drainage has been design to store all storms up to and including the 1 in 100 year + 40% climate change within the site.
20. The junctions from RWP's, fin drains, SVP's and similar connections into main runs are to be determined on site, and in accordance with building regulations.
21. All RWP's, SVP's, etc., to be fitted with rodding access.
22. For PPIC's the orientation of the main channel is to generally connect upstream and downstream pipes with stated inverts. Non stated inverts such as RE's, R's and F's to generally connect into side inlets.
23. Drainage generally designed to incorporate a side step in preformed bases down into the main channel invert of of 100mm. Details vary dependant on the preformed base and manufacturer selected.
24. Backdrops might be required to ensure that building drainage above foundations are able to enter the PPIC - refer to details.
25. Unless noted , rain water pipes (R's) and below ground foul drainage pipes (F's) to be 100mmØ . R's to have a minimum fall of 1:100, and F's to have a minimum fall of 1:40, unless a WC is connected when the minimum fall shall be 1:80.
26. All RWP's and below ground foul drainage to be set out from the architectural drawings.
27. Where drainage gradients have been stated, these represent the minimum gradients, and can be steeper - subject to a maximum gradient of 1:10.
28. If the contractor is unsure which manhole/PPIC reference is being used for the actual manhole/PPIC, ask!
29. Cover levels are approximate only and must suit the finished external levels.
30. All tie in levels with existing items are to be confirmed prior to construction.
31. RWP positions have currently been assumed and must be confirmed by the architect.
32. Above ground foul drainage has currently been assumed and must be confirmed by the architect.

PLANNING

For planning authority approval

Rev	Description	Drn	Ck'd	Date

Client:
Y6 Architectural

Woodside Consulting Engineers Ltd
53 Brethergate, Westwoodside, Doncaster, DN9 2AA
email: paul@woodsideconsultingengineers.co.ukTel: 07914950587

Project Title:

Proposed Residential Development,
Land off The Drove,
Osbournby, Sleaford

Drawing Title:

Proposed Drainage General Arrangement

Drawing Status:
Preliminary

Scale: 1:250	Paper size: A1	Date: 15/07/2022
Drawn: WCE	Checked: WCE	Approved: WCE

Drawing Number:	Rev
20474-SK-3000	-

Appendix E – Drainage Calculations

Design Settings

Rainfall Methodology	FSR
Return Period (years)	1
Additional Flow (%)	0
FSR Region	England and Wales
M5-60 (mm)	20.000
Ratio-R	0.400
CV	0.750
Time of Entry (mins)	5.00
Maximum Time of Concentration (mins)	30.00
Maximum Rainfall (mm/hr)	50.0
Minimum Velocity (m/s)	1.00
Connection Type	Level Soffits
Minimum Backdrop Height (m)	0.200
Preferred Cover Depth (m)	1.200
Include Intermediate Ground	✓
Enforce best practice design rules	✓

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Depth (m)
S1			19.500	1350	1.627
S2	0.060	5.00	19.500	1350	1.700
S3	0.014	5.00	19.500	1500	2.141
S4			19.500	1500	2.170
S5	0.029	5.00	19.500	1500	2.190
S6	0.030	5.00	19.500	1500	2.207
S7	0.056	5.00	19.500	1500	2.246
Headwall			19.500	1500	2.254
PS1	0.055	5.00	19.500	1200	1.450
PS4	0.060	5.00	19.500	1350	1.575
PS1a	0.049	5.00	19.500	1200	1.350

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)
1.000	PS1a	S1	10.786	0.600	18.150	17.948	0.202	53.4	150	5.13	50.0
2.000	PS1	S1	10.173	0.600	18.050	17.948	0.102	100.0	150	5.17	50.0
1.001	S1	S2	12.360	0.600	17.873	17.800	0.073	170.0	225	5.37	50.0
1.002	S2	S3	11.193	0.600	17.800	17.734	0.066	170.0	225	5.56	50.0
1.003	S3	S4	16.947	0.600	17.359	17.330	0.029	580.0	600	5.84	50.0
3.000	PS4	S4	14.518	0.600	17.925	17.780	0.145	100.0	150	5.24	50.0
1.004	S4	S5	11.455	0.600	17.330	17.310	0.020	580.0	600	6.03	50.0
1.005	S5	S6	9.669	0.600	17.310	17.293	0.017	580.0	600	6.19	49.6
1.006	S6	S7	22.778	0.600	17.293	17.254	0.039	580.0	600	6.57	48.2
1.007	S7	Headwall	4.740	0.600	17.254	17.246	0.008	580.0	600	6.65	47.9

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)
1.000	1.379	24.4	6.6	1.200	1.402	0.049	0.0	54	1.179
2.000	1.005	17.8	7.5	1.300	1.402	0.055	0.0	68	0.961
1.001	1.000	39.7	14.1	1.402	1.475	0.104	0.0	93	0.917
1.002	1.000	39.7	22.2	1.475	1.541	0.164	0.0	120	1.026
1.003	1.004	283.8	24.1	1.541	1.570	0.178	0.0	117	0.621
3.000	1.005	17.8	8.1	1.425	1.570	0.060	0.0	71	0.983
1.004	1.004	283.8	32.3	1.570	1.590	0.238	0.0	135	0.675
1.005	1.004	283.8	35.9	1.590	1.607	0.267	0.0	143	0.696
1.006	1.004	283.8	38.8	1.607	1.646	0.297	0.0	149	0.711
1.007	1.004	283.8	45.9	1.646	1.654	0.353	0.0	162	0.746

Pipeline Schedule

Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
1.000	10.786	53.4	150	Circular	19.500	18.150	1.200	19.500	17.948	1.402
2.000	10.173	100.0	150	Circular	19.500	18.050	1.300	19.500	17.948	1.402
1.001	12.360	170.0	225	Circular	19.500	17.873	1.402	19.500	17.800	1.475
1.002	11.193	170.0	225	Circular	19.500	17.800	1.475	19.500	17.734	1.541
1.003	16.947	580.0	600	Circular	19.500	17.359	1.541	19.500	17.330	1.570
3.000	14.518	100.0	150	Circular	19.500	17.925	1.425	19.500	17.780	1.570
1.004	11.455	580.0	600	Circular	19.500	17.330	1.570	19.500	17.310	1.590
1.005	9.669	580.0	600	Circular	19.500	17.310	1.590	19.500	17.293	1.607
1.006	22.778	580.0	600	Circular	19.500	17.293	1.607	19.500	17.254	1.646
1.007	4.740	580.0	600	Circular	19.500	17.254	1.646	19.500	17.246	1.654

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	PS1a	1200	Manhole	1 Adoptable	S1	1350	Manhole	1 Adoptable
2.000	PS1	1200	Manhole	1 Adoptable	S1	1350	Manhole	1 Adoptable
1.001	S1	1350	Manhole	1 Adoptable	S2	1350	Manhole	1 Adoptable
1.002	S2	1350	Manhole	1 Adoptable	S3	1500	Manhole	1 Adoptable
1.003	S3	1500	Manhole	1 Adoptable	S4	1500	Manhole	1 Adoptable
3.000	PS4	1350	Manhole	1 Adoptable	S4	1500	Manhole	1 Adoptable
1.004	S4	1500	Manhole	1 Adoptable	S5	1500	Manhole	1 Adoptable
1.005	S5	1500	Manhole	1 Adoptable	S6	1500	Manhole	1 Adoptable
1.006	S6	1500	Manhole	1 Adoptable	S7	1500	Manhole	1 Adoptable
1.007	S7	1500	Manhole	1 Adoptable	Headwall	1500	Manhole	1 Adoptable

Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
S1	19.500	1.627	1350	1	2.000	17.948	150
				2	1.000	17.948	150
				0	1.001	17.873	225
S2	19.500	1.700	1350	1	1.001	17.800	225
				0	1.002	17.800	225
S3	19.500	2.141	1500	1	1.002	17.734	225
				0	1.003	17.359	600
S4	19.500	2.170	1500	1	3.000	17.780	150
				2	1.003	17.330	600
				0	1.004	17.330	600
S5	19.500	2.190	1500	1	1.004	17.310	600
				0	1.005	17.310	600
S6	19.500	2.207	1500	1	1.005	17.293	600
				0	1.006	17.293	600

Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
S7	19.500	2.246	1500	1	1.006	17.254	600
				0	1.007	17.254	600
Headwall	19.500	2.254	1500	1	1.007	17.246	600
PS1	19.500	1.450	1200	0	2.000	18.050	150
PS4	19.500	1.575	1350	0	3.000	17.925	150
PS1a	19.500	1.350	1200	0	1.000	18.150	150

Simulation Settings

Rainfall Methodology	FSR	Analysis Speed	Normal
FSR Region	England and Wales	Skip Steady State	x
M5-60 (mm)	20.000	Drain Down Time (mins)	240
Ratio-R	0.400	Additional Storage (m^3/ha)	0.0
Summer CV	0.750	Check Discharge Rate(s)	x
Winter CV	0.840	Check Discharge Volume	x

Storm Durations

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

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
1	0	0	0
2	0	0	0
30	0	0	0
100	0	0	0
100	40	0	0

Node S7 Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	17.254	Product Number	CTL-SHE-0072-2000-0650-2000
Design Depth (m)	0.650	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	2.0	Min Node Diameter (mm)	1200

Node PS1 Online Orifice Control

Flap Valve	x	Invert Level (m)	18.050	Discharge Coefficient	0.600
Replaces Downstream Link	✓	Diameter (m)	0.015		

Node PS4 Online Orifice Control

Flap Valve	x	Invert Level (m)	17.925	Discharge Coefficient	0.600
Replaces Downstream Link	✓	Diameter (m)	0.015		

Node PS1a Online Orifice Control

Flap Valve	x	Invert Level (m)	18.150	Discharge Coefficient	0.600
Replaces Downstream Link	✓	Diameter (m)	0.020		

Node S7 Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	17.900	Product Number	CTL-SHE-0090-3000-0500-3000
Design Depth (m)	0.500	Min Outlet Diameter (m)	0.150
Design Flow (l/s)	3.0	Min Node Diameter (mm)	1200

Node S7 Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	18.100	Product Number	CTL-SHE-0055-1000-0400-1000
Design Depth (m)	0.400	Min Outlet Diameter (m)	0.075
Design Flow (l/s)	1.0	Min Node Diameter (mm)	1200

Node PS1 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	18.700
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	215.0	0.0	0.600	215.0	0.0	0.601	0.0	0.0

Node PS4 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	18.700
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	230.0	0.0	0.600	230.0	0.0	0.601	0.0	0.0

Node S4 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	17.950
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	168

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	85.0	0.0	0.400	85.0	0.0	0.401	0.0	0.0

Node S4 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	17.900
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	204

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	52.0	0.0	0.400	52.0	0.0	0.401	0.0	0.0

Node PS1a Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	18.700
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	143.0	0.0	0.600	143.0	0.0	0.601	0.0	0.0

Results for 1 year Critical Storm Duration. Lowest mass balance: 93.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
240 minute winter	S1	232	17.907	0.034	1.0	0.0488	0.0000	OK
240 minute winter	S2	236	17.907	0.107	2.7	0.1531	0.0000	OK
240 minute winter	S3	232	17.907	0.548	3.1	0.9681	0.0000	OK
240 minute winter	S4	232	17.907	0.577	3.2	1.3906	0.0000	OK
240 minute winter	S5	232	17.904	0.594	3.4	1.0497	0.0000	OK
240 minute winter	S6	232	17.907	0.614	5.3	1.0851	0.0000	SURCHARGED
240 minute winter	S7	232	17.901	0.647	3.2	1.1441	0.0000	SURCHARGED
15 minute summer	Headwall	1	17.246	0.000	2.0	0.0000	0.0000	OK
120 minute winter	PS1	116	18.765	0.715	2.5	5.0539	0.0000	SURCHARGED
120 minute winter	PS4	116	18.763	0.838	2.8	5.6017	0.0000	SURCHARGED
60 minute winter	PS1a	55	18.764	0.614	3.6	3.4566	0.0000	SURCHARGED

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
180 minute winter	S1	1.001	S2	1.1	0.358	0.027	0.1232	
15 minute winter	S2	1.002	S3	9.3	0.794	0.234	0.1313	
15 minute summer	S3	1.003	S4	9.2	0.418	0.032	1.8229	
15 minute summer	S4	1.004	S5	4.3	0.293	0.015	1.3997	
240 minute winter	S5	1.005	S6	5.1	0.228	0.018	2.7214	
180 minute winter	S6	1.006	S7	3.8	0.142	0.013	6.4160	
15 minute summer	S7	Hydro-Brake®	Headwall	2.0				20.4
240 minute winter	S7	Hydro-Brake®	Headwall	0.0				0.0
15 minute summer	S7	Hydro-Brake®	Headwall	0.0				0.0
120 minute winter	PS1	Orifice	S1	0.4				
120 minute winter	PS4	Orifice	S4	0.4				
60 minute winter	PS1a	Orifice	S1	0.6				

Results for 2 year Critical Storm Duration. Lowest mass balance: 93.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
240 minute winter	S1	184	17.951	0.078	1.1	0.1114	0.0000	OK
240 minute winter	S2	184	17.951	0.151	3.1	0.2159	0.0000	OK
240 minute winter	S3	184	17.951	0.592	3.6	1.0457	0.0000	OK
240 minute winter	S4	184	17.951	0.621	3.5	3.7272	0.0000	SURCHARGED
240 minute winter	S5	184	17.951	0.641	4.6	1.1328	0.0000	SURCHARGED
240 minute winter	S6	184	17.949	0.656	4.4	1.1598	0.0000	SURCHARGED
240 minute winter	S7	184	17.950	0.696	4.2	1.2301	0.0000	SURCHARGED
15 minute summer	Headwall	1	17.246	0.000	2.0	0.0000	0.0000	OK
120 minute winter	PS1	118	18.792	0.742	3.2	6.7748	0.0000	SURCHARGED
180 minute winter	PS4	168	18.790	0.865	2.6	7.4516	0.0000	SURCHARGED
60 minute winter	PS1a	58	18.792	0.642	4.5	4.7050	0.0000	SURCHARGED

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
120 minute winter	S1	1.001	S2	1.5	0.321	0.037	0.2418	
15 minute winter	S2	1.002	S3	11.7	0.843	0.294	0.1553	
15 minute summer	S3	1.003	S4	10.0	0.421	0.035	2.5572	
480 minute summer	S4	1.004	S5	6.8	0.237	0.024	3.2192	
360 minute summer	S5	1.005	S6	6.1	0.233	0.022	2.7235	
240 minute summer	S6	1.006	S7	4.0	0.144	0.014	6.4160	
240 minute winter	S7	Hydro-Brake®	Headwall	2.1				52.0
240 minute winter	S7	Hydro-Brake®	Headwall	1.1				4.9
15 minute summer	S7	Hydro-Brake®	Headwall	0.0				0.0
120 minute winter	PS1	Orifice	S1	0.4				
180 minute winter	PS4	Orifice	S4	0.4				
60 minute winter	PS1a	Orifice	S1	0.7				

Results for 30 year Critical Storm Duration. Lowest mass balance: 93.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
180 minute winter	S1	140	18.086	0.213	1.2	0.3048	0.0000	OK
180 minute winter	S2	140	18.086	0.286	5.6	0.4091	0.0000	SURCHARGED
180 minute winter	S3	140	18.086	0.727	6.5	1.2839	0.0000	SURCHARGED
180 minute winter	S4	140	18.086	0.756	11.7	21.5175	0.0000	SURCHARGED
180 minute winter	S5	148	18.086	0.776	6.2	1.3703	0.0000	SURCHARGED
180 minute winter	S6	140	18.087	0.794	5.6	1.4027	0.0000	SURCHARGED
180 minute winter	S7	140	18.087	0.833	5.9	1.4712	0.0000	SURCHARGED
15 minute summer	Headwall	1	17.246	0.000	2.1	0.0000	0.0000	OK
180 minute winter	PS1	176	18.918	0.868	4.4	15.0763	0.0000	SURCHARGED
240 minute winter	PS4	236	18.922	0.997	3.9	16.7445	0.0000	SURCHARGED
120 minute winter	PS1a	116	18.934	0.784	5.3	10.9287	0.0000	SURCHARGED

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
60 minute winter	S1	1.001	S2	-2.2	0.350	-0.055	0.4643	
15 minute winter	S2	1.002	S3	21.1	0.980	0.531	0.4044	
30 minute winter	S3	1.003	S4	22.8	0.383	0.080	4.7736	
30 minute winter	S4	1.004	S5	-28.5	0.311	-0.100	3.2266	
30 minute winter	S5	1.005	S6	-24.5	0.303	-0.086	2.7235	
30 minute winter	S6	1.006	S7	-17.1	0.139	-0.060	6.4160	
180 minute winter	S7	Hydro-Brake®	Headwall	2.2				50.1
60 minute winter	S7	Hydro-Brake®	Headwall	3.0				19.0
15 minute summer	S7	Hydro-Brake®	Headwall	0.0				0.0
240 minute winter	PS1	Orifice	S1	0.4				
480 minute winter	PS4	Orifice	S4	0.5				
120 minute winter	PS1a	Orifice	S1	0.7				

Results for 100 year Critical Storm Duration. Lowest mass balance: 93.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
180 minute winter	S1	140	18.192	0.319	1.2	0.4560	0.0000	SURCHARGED
180 minute winter	S2	140	18.192	0.392	7.2	0.5603	0.0000	SURCHARGED
180 minute winter	S3	140	18.191	0.832	8.5	1.4705	0.0000	SURCHARGED
180 minute winter	S4	140	18.191	0.861	14.9	35.4484	0.0000	SURCHARGED
180 minute winter	S5	140	18.191	0.881	7.3	1.5560	0.0000	SURCHARGED
180 minute winter	S6	140	18.192	0.899	7.1	1.5884	0.0000	SURCHARGED
180 minute winter	S7	140	18.191	0.937	6.8	1.6561	0.0000	SURCHARGED
15 minute summer	Headwall	1	17.246	0.000	2.1	0.0000	0.0000	OK
240 minute winter	PS1	236	19.009	0.959	4.6	21.0655	0.0000	SURCHARGED
240 minute winter	PS4	236	19.014	1.089	5.0	23.2647	0.0000	SURCHARGED
120 minute winter	PS1a	118	19.036	0.886	7.0	15.4590	0.0000	SURCHARGED

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
60 minute winter	S1	1.001	S2	-2.3	0.247	-0.058	0.4916	
15 minute winter	S2	1.002	S3	27.2	1.029	0.684	0.4450	
15 minute winter	S3	1.003	S4	32.2	0.459	0.114	4.7736	
15 minute winter	S4	1.004	S5	-52.2	0.312	-0.184	3.2266	
15 minute winter	S5	1.005	S6	-42.0	0.288	-0.148	2.7235	
15 minute winter	S6	1.006	S7	-32.7	0.127	-0.115	6.4160	
180 minute winter	S7	Hydro-Brake®	Headwall	2.4				52.2
30 minute summer	S7	Hydro-Brake®	Headwall	3.0				16.7
180 minute winter	S7	Hydro-Brake®	Headwall	1.0				5.0
360 minute winter	PS1	Orifice	S1	0.5				
480 minute winter	PS4	Orifice	S4	0.5				
180 minute winter	PS1a	Orifice	S1	0.8				

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 93.99%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
180 minute winter	S1	136	18.821	0.948	1.4	1.3571	0.0000	SURCHARGED
180 minute winter	S2	140	18.821	1.021	9.7	1.4609	0.0000	SURCHARGED
180 minute winter	S3	140	18.820	1.461	11.5	2.5822	0.0000	SURCHARGED
180 minute winter	S4	140	18.821	1.491	22.5	54.8241	0.0000	SURCHARGED
180 minute winter	S5	140	18.820	1.510	11.4	2.6684	0.0000	SURCHARGED
180 minute winter	S6	140	18.821	1.528	7.3	2.6993	0.0000	SURCHARGED
180 minute winter	S7	140	18.820	1.566	9.2	2.7671	0.0000	SURCHARGED
15 minute summer	Headwall	1	17.246	0.000	2.2	0.0000	0.0000	OK
480 minute winter	PS1	464	19.179	1.129	3.7	32.1720	0.0000	SURCHARGED
360 minute winter	PS4	352	19.188	1.263	5.2	35.5206	0.0000	SURCHARGED
180 minute winter	PS1a	176	19.240	1.090	7.2	24.4151	0.0000	FLOOD RISK

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	S1	1.001	S2	-2.5	0.177	-0.063	0.4916	
15 minute summer	S2	1.002	S3	35.7	1.057	0.898	0.4452	
15 minute winter	S3	1.003	S4	40.4	0.503	0.142	4.7736	
15 minute summer	S4	1.004	S5	-66.9	0.298	-0.236	3.2266	
30 minute summer	S5	1.005	S6	-48.7	0.278	-0.171	2.7235	
15 minute winter	S6	1.006	S7	-29.7	0.145	-0.105	6.4160	
180 minute winter	S7	Hydro-Brake®	Headwall	3.0				57.0
180 minute winter	S7	Hydro-Brake®	Headwall	4.0				61.2
180 minute winter	S7	Hydro-Brake®	Headwall	1.3				12.3
720 minute winter	PS1	Orifice	S1	0.5				
720 minute winter	PS4	Orifice	S4	0.5				
180 minute winter	PS1a	Orifice	S1	0.8				