JACKSON CONSULTING ENGINEERS

Preliminary Drainage Strategy

Project:

Residential Development to Land East of St Margarets, Quadring

Client:

Seagate Homes Ltd

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Introduction

Jackson consulting engineers have been appointed by Seagate Homes Ltd, to undertake a drainage strategy, forming part of the supporting evidence of the planning submission for the proposed residential development to land East of St Margarets, Quadring, PE11 4PR. 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 flood risk as a result of the development.

Existing Site

The site lies to the east of St Margarets. An aerial location plan is shown in figure 1.



Figure 1 - Site Location

Site Topography

A site survey has been carried out by Axis Surveys Ltd, drawing number AS2513/1, which can be found in appendix A.

The development sits just north of the A152, known as 'Main Road' which is separated by a single row of houses, the development backs up to their back gardens. There is a thin access point which connects the site to the junction of Sarah Gate/Main Road. The eastern boundary of the site is bounded by Sarah Gate, with the north section of the eastern boundary being a ditch. The northern boundary has a ditch which is assumed to be riparian and draining in a westerly direction. At the time of survey, the ditch had a water level of 2.19m AOD. This ditch leads into an IDB drain. To the north of this boundary

is open fields. The western boundary is mainly bounded by existing gardens, with the middle section being existing public open space. The total area of the site is approximately 1.08 hectares.

The site is accessed from the eastern arm of St Margarets. The existing road leading into the access is generally longitudinally flat with a camber falling either side of the central crown. The road does not have kerb upstands (some concrete channels laid level/below the surfacing appear to be visible) and any surface water discharges into the PoS to the north or soft landscaping to the south. The crown level of the road varies from 3.21 to 3.24m AOD along this stretch of road with a crossfall level reduction of approximately 70mm either side. The end of the road at the tie in point is approximately 3.19m AOD.

The site is greenfield with no evidence of existing or historic structures. The topography of the site is generally flat, with the majority of the site ranging between 2.74 to 3.35m AOD. The site subtly falls towards the north east and north ditch before eventually all flows enter the north ditch.

Ground Conditions

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

The bedrock geology of the site is described as 'Oxford Clay Formation'. This generally indicates that deeper infiltration devices, such as house soakaways are not viable for the site.

The superficial deposits are described as 'Tidal Flat Deposits', consisting of clays and silts. This type of strata can offer some limited infiltration but is generally restricted by the underlying clay. Additionally, the soaked CBR's of the silty clay are likely to be poor and not allow the use of items such as porous paving.

There is no borehole information in close enough proximity of the site to be relevant. Given experience with the local ground conditions, it is anticipated that the water level will be relatively close to the surface, potentially perched on top of the Oxford Clay strata. It is anticipated that these conditions will mean that deeper infiltration devices are not viable other than very shallow installations.

Both British Geological Survey definitions can be found respectively in figures 2 and 3 below.

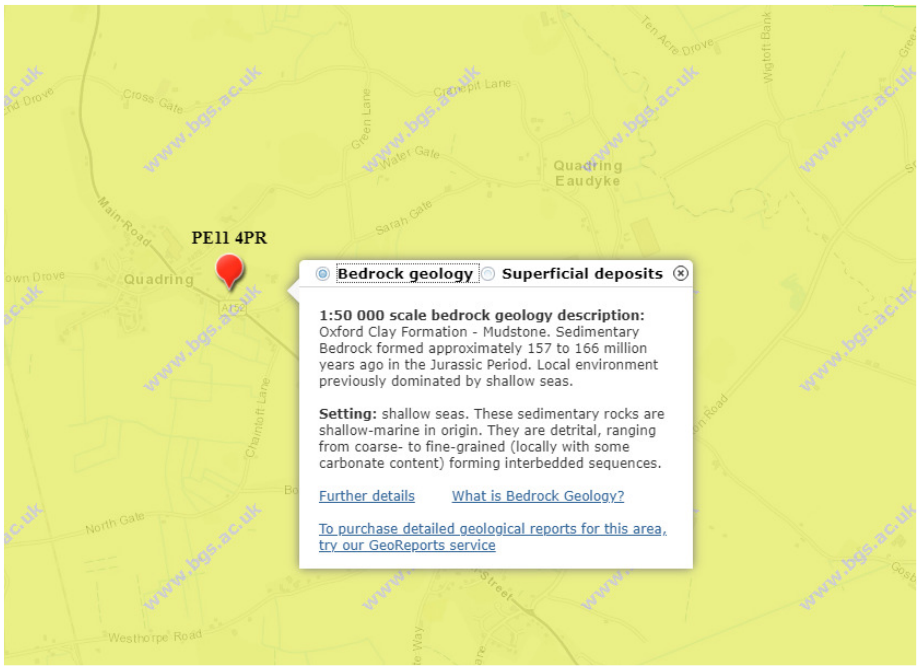


Figure 2 - BGS Bedrock Geology

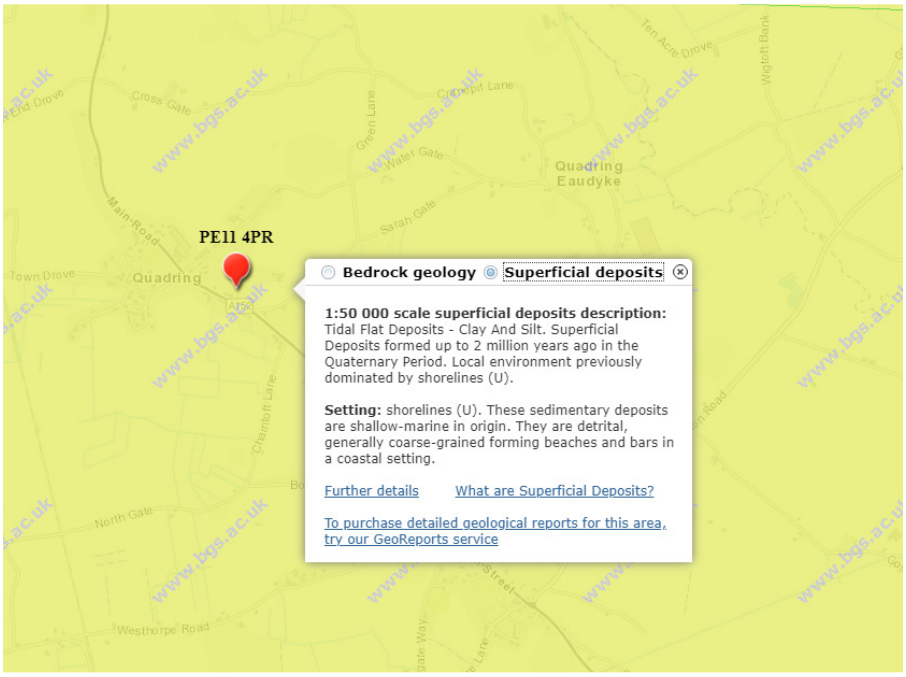


Figure 3 - BGS Superficial Deposits

Local Watercourses

There is a ditch which runs to the north of the site, and it is expected that this falls under riparian rights. Under any circumstance the natural gradients fall towards the ditch, and this would currently drain any greenfield run-off.

The watercourse/ditch to the north of the eastern boundary and the north boundary is generally in good condition, but would benefit from being cleared during the development to remove any silt and

overgrown vegetation. The northern ditch flows east where it joins into the Welland and Deepings IDB drain known as Watergate Dyke, which flows north into the overall IDB system. The IDB drain is approximately 70m from the north east corner of the site. The IDB drains are shown in figure 4.

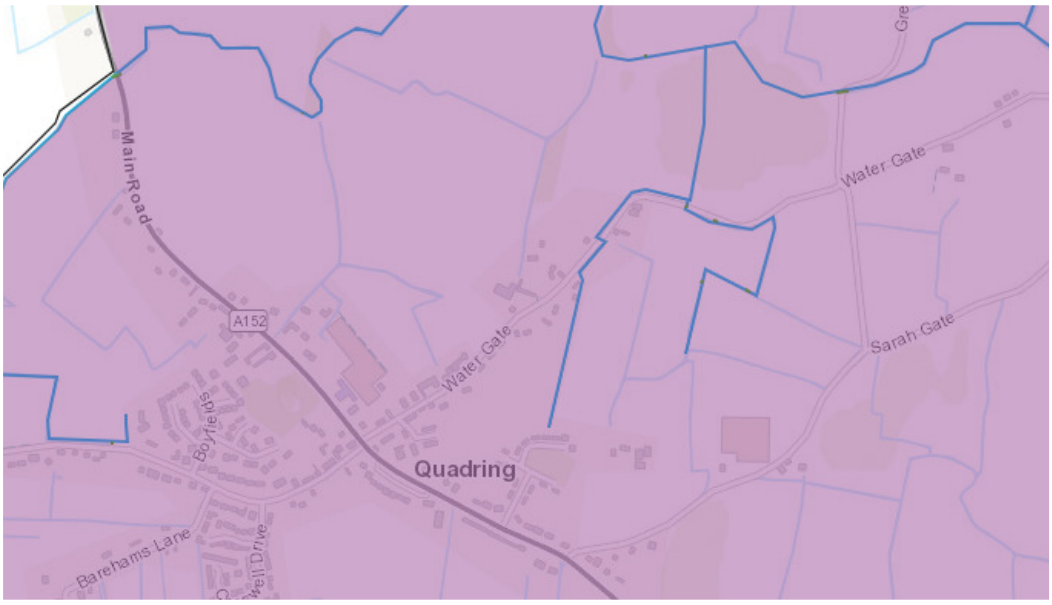


Figure 4 – Welland and Deepings IDB Drain Locations

Surface Water Flood Risk

The existing flood risk for the site has been assessed by a different consultant and can be found within report: ECL0543/SEAGATE Homes. The purpose of this document is to justify that flood risk as a result of the development will not be increased, and any discussions relating to the development proposals being suitable as a result of the existing flood risk assessment. The FRA has recommendations for the finished floor levels and other flood protection measures. For the purpose of the drainage strategy it is proposed that the FFL's are to be a minimum of 300mm above existing ground levels.

Development Proposals

The proposed development consists of creating 31 new dwelling consisting of a mixture of detached, semidetached and terraced housing, with the majority of the dwellings being semidetached. Each dwelling benefits from modest garden. Outline planning permission for the site was granted under South Holland District Council reference: H15-1039-17.

It is proposed that the main road running through the site and the main drainage will be put forward for adoption under a section 38 and section 104 agreement with Lincolnshire County Council and Anglian Water respectively. Because the site is flat and depths to both the surface and foul water outfalls are limited, parts of the drainage will be laid in private gardens (and remain private in places) to ensure the design solution is viable, especially in respect to gradient and cover depths of existing pipes.

In order to keep the FFL's in keeping with surrounding development and to achieve the minimum of 300mm above existing ground levels, both the surface and foul water will need to be pumped. An allowance for a dual – adoptable pumping station is provided to the north of the development.

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

Infiltration results are currently not available but giving to the underlying clay and anticipated high-water tables, it is assumed that infiltration is generally unsuitable for the site, although it could potentially be used in limited capacities. Additionally, the soaked CBR values are likely to not be suitable owing to the silty cray strata.

The entrance of the site has a section which has limited space to create suitable drainage features. In this location, it is proposed that 'over-the-edge' drainage will be used and discharge to the field to the north. This is a slightly informal arrangement, but the remainder of the existing road drains in this manner and there are no current drainage issues.

Watercourse

There is a watercourse located to the north of the development and the site naturally flows towards this, therefore any existing overland flows and run-off will be drained by the watercourse. The client has confirmed they have a right to connect into this.

As previously confirmed the northern ditch flows east where it joins into the Welland and Deepings IDB drain known as Watergate Dyke, which flows north into the overall IDB system. The IDB drain is approximately 70m from the edge of the site. The IDB drains are shown in figure 4.

This is the proposed method of waste surface water disposal from the site and given the strata on-site it is deemed that this method of surface water disposal mimics the existing greenfield drainage.

Surface Water Sewer

There are no identified surface water sewers in the vicinity of the site. A discussion with LCC has taken place regarding existing highways gullies which are located in proximity to the proposed development. These gullies are known to be blocked and the outfall is unknown.

As a watercourse connection is viable, then there is no need for a surface water sewer to be considered.

Run-off Rates

Existing Drainage Arrangements

As the site is greenfield it has no current surface water drainage. The natural topography of the site directs surface runoff into the existing watercourses to the east and north of the site and will either flow directly overland or laterally within the strata layers, certainly as the underlying strata, the Oxford Clay Formation, would be considered as being very slow draining to impermeable.

Existing greenfield run-off

The greenfield run-off for the site is summarised in figure 5, with a full copy of the report provided in appendix B.

Greenfield runoff rates

	Default	Edited
Q_{BAR} (l/s):	1.51	1.51
1 in 1 year (l/s):	1.31	1.31
1 in 30 years (l/s):	3.7	3.7
1 in 100 year (l/s):	5.37	5.37
1 in 200 years (l/s):	6.36	6.36

Figure 5 - Existing Greenfield Runoff Rates

The Q_{BAR} greenfield run-off for the site is calculated at 1.5 l/s, and this is in accordance with industry guidance with only large PoS areas being discounted for the calculation.

The IDB have their own allowable rate based on their pumped catchment, and this is set at 1.4l/s per hectare, which is roughly in line with the figures above. At present, the actual run-off from the site will not be increased above Q_{BAR} and flood risk as a result of the development is not increased.

Both LCC and DEFRA's standards allow the runoff from the site to not be increase above the like for like storm – both of these are inline with statutory standards. As explained within this document, there are restrictions on the surface water pump which require a minimum discharge – additionally AW will require a minimum discharge rate of 2l/s to be allowable for adoption (the pump rate is likely to be higher than this). At detailed discussion with the IDB, it will be discussed to increase the discharge rate from the site above Q_{BAR}, but still in line with the statutory stanadrds.

The HR Wallingford Greenfield runoff rate estimation for sites is an industry standard way to determine the greenfield run-off. The calculator uses the IH124 method to determine the runoff.

Existing Run Off Volume

The runoff volume for the 1 in 100, 360-minute storm is 108m^3, as shown in figure 6. This has been calculated using Causeway Flow.

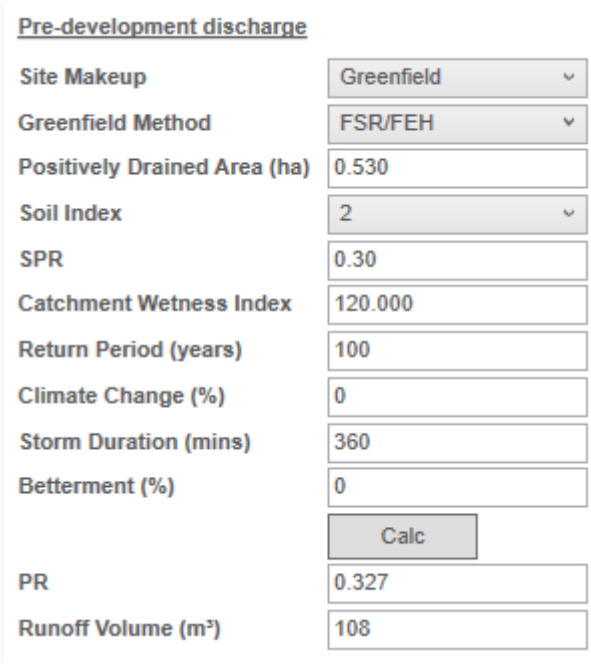


Figure 6 - Existing Greenfield Runoff Volume

Proposed Run-off

The greenfield run-off from the site should, as far as reasonably practical, not exceed the greenfield runoff rate for the Q_{BAR} event, which in this instance is 1.5 l/s. Therefore, the proposed discharge from the site will have a maximum runoff of 1.5 l/s, for all storms up to and including the 1 in 100 year + 40% climate change event. The post development run-off values are summarised in the table below. These figures represent the non-surcharged event. The full calculations can be found in appendix C.

Storm	Critical Flow
1 Year	1.3 l/s
2 Year	1.4 l/s
30 Year	1.5 l/s
100 Year	1.5 l/s
100 Year + 40% CC	1.5 l/s

Proposed Discharge Volume

The proposed discharge volume for the 1 in 100-year 6-hour duration winter event is 36.8m^3, which is substantially lower than the existing greenfield discharge volume for the same storm being 108m^3.

As the runoff into the existing ditch is not being increased above the current scenario, it has sufficient capacity. It should also be noted that the runoff velocity for the Q_{BAR} event is not being increased, therefore the run-off rate from the site for all storms approximately above the 1 in 2 year event is being reduced, given a betterment over the existing natural site for storms greater than this.

Drainage Proposals

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. All of the water from these sources will pass through at least one treatment device.

The buildings are being drained by traditional rainwater pipes, before connecting into below ground drainage pipes, before discharging into the swale or eventual basin.

The highway drainage generally falls into an under drained swale, providing suitable source control for this area. As the filter drain will receive domestic water it is proposed that the under draining pipe will be maintained by Anglian Water. The final arrangements will be determined at detailed design stage.

It is proposed that the domestic drives will drain to shallow aco drains which will then either connect above ground into the swale, or directly into the below ground drainage before discharging eventually into the basin.

As previously described, the start of the entrance road cannot be included within the proposed drainage system as the levels will not allow this to happen. This area will be drained via. a shallow side swale, with the water discharging into the existing top strata and topsoil. The proposed area to be drained is approximately 220m^2, which includes the swale area. The existing road drains in this fashion, and as the water will be directed towards the existing PoS, and any flooding will be confined to a small volume, dispersed over a large area, giving shallow water depths, or result in 'soggy' ground. This strategy has tentatively been discussed with LCC and at the time of writing this report we still await an official response.

In order to ensure the dwellings finished floor levels remain reasonable, the site uses a proposed adoptable pumping station. The pump is restricted to 5l/s with water being temporarily stored in an upstream basin. Given the limitations of pumping station flow rates, and to ensure the development does not increase flood risk, the pump will lift the water into a shallow second basin, which is restricted by a Hydrobrake at the rate of 1.5l/s.

This arrangement is necessary for the initial strategy due to the relatively shallow ditch, restrictions on the pump and finished floor levels. At detailed design, the IDB may agree for a higher discharge rate, and this will allow the removal of the Hydrobrake and the basin can be joined up. In the event the pond floods it will naturally discharge into the northern ditch.

The system has been designed so that there is no above ground flooding for the 1 in 30-year event and all flooding up to and including the 1 in 100 year + 40% climate change event is contained within the site. The calculations demonstrate that the 1 in 30-year storm event is stored below ground. For the 1 in 100-year + 40% climate change event, there is some minor flooding – a total of approximately 6m^3 located in rear gardens – this would easily dissipate into the surrounding area, or within the private, unmodelled drainage pipes.

Rear garden patios or pavements will be laid to soft landscaping or laid with open joints and mimic a like for like drainage arrangement.

To comply with the NTS, the site's drainage has been design considered a 40% climate change allowance, and to comply with LCC's requirements, an allowance of 10% for urban creep has been allowed for. At present, a global factor of 10% has been used, but at detailed design this will be incorporated for each individual area and only including private curtilages.

Most of the main drainage is proposed to be put forward for adoption to Anglian Water. Some of the private drainage will remain private as they will not meet the criteria for adoption. The provisional sizing of the Hydrobrake is 61mm, which may not be suitable for adoption, but is subject to further discussions with AW. In the event that it is not suitable then the Hydrobrake will be placed into a private management company. The basin generally meets the criteria for adoption, but this is dependent on confirmation of the existing FFL's and dependant on the adoption of the Hydrobrake. It will be beneficial to have a weir to ensure if the Hydrobrake was to fail, the site would discharge into the existing watercourse. Again, the agreement of the basin is subject to further discussion with AW and if not suitable for adoption it will be placed into a management company.

The proposed drainage layout is shown on JCE drawing 077-JCE-00-SI-DR-C-3000, which can be found in appendix D and a 3D schematic of the drainage model shown in figure 7.

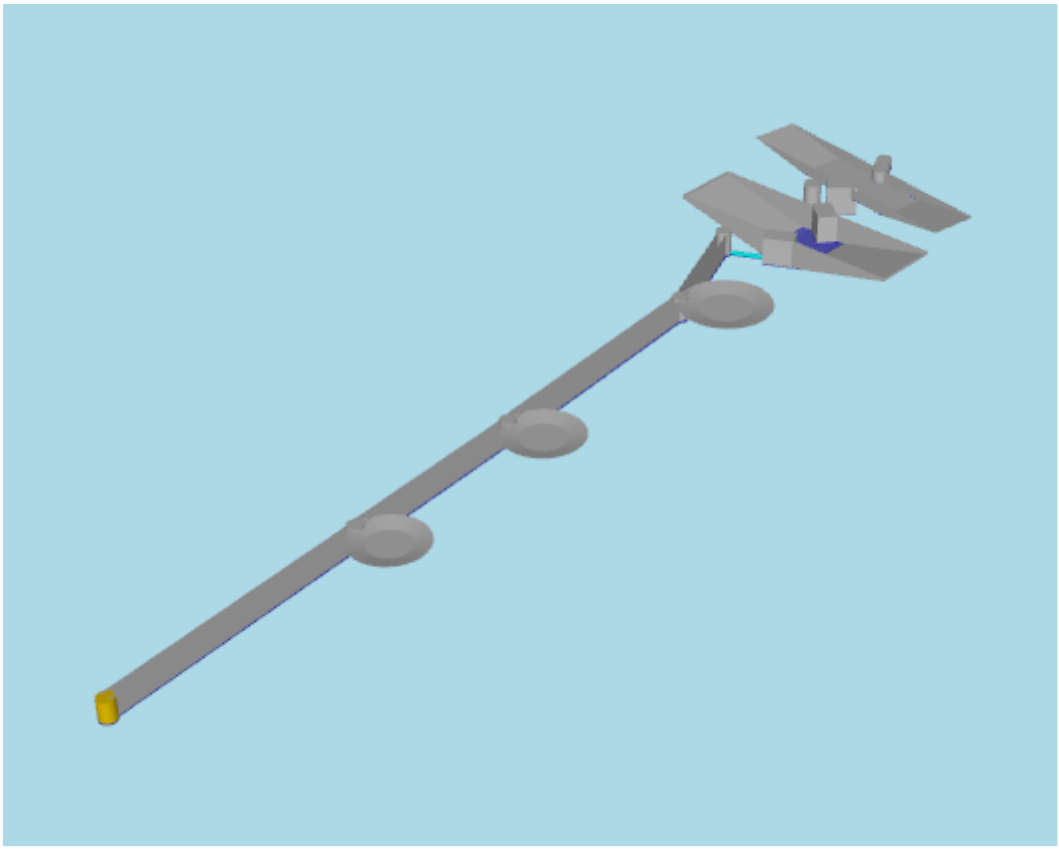


Figure 7 - 3D View of Drainage Model

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. A surcharged outfall has been included as part of the calculations. The field levels in relation to the water level in the ditch are only around 600mm and it would be expected that a natural freeboard of at least 300mm is available, therefore the maximum surcharge event is 300mm. A flap valve is to be fitted downstream of the Hydrobrake to ensure offsite water does not enter the site, subject to agreement with the adopting body.

The maximum water level in this event is 3.167m AOD, and this represents the worst case, highest pond level scenario. This gives an approximate water depth in the basin of 0.90m.

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. A weir overflow (constructed out of the earth at the top of bank) is to be considered at the basin/watercourse location to ensure exceedance flows leave the site at this point and not the uncontrolled within the site. The top of bank for the pond is set at 3.200m AOD, which is similar to the surrounding levels. The modelled water level in the pond during the critical 1 in 100 year + 40% storm event is 3.167m AOD.

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 - Low
- Residential parking - Low
- Cul de sac - 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 figures 8 and 9 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 8 - 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 9 - 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:

$$\text{Total SuDS mitigation index} = \text{mitigation index}_1 + 0.5\ (\text{mitigation index}_2)$$

Where:

$$\text{Mitigation index}_n = \text{mitigation index for component } n$$

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 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 generally enters the below ground pipework directly, although at detailed design some of this will enter the porous paving and filter into the system. Roof water will generally be considered to be clean, but will flow through the attenuation basin prior to discharging into the watercourse. The mitigation for this area is:

0.2 0.2 0.05 = Detention Basin 0.5 0.5 0.6

The mitigation provided by the detention basin creates enough water quality improvement for this aspect of the development.

Residential parking

The residential parking will either discharge into a swale and then the basin, or just into the basin. The mitigation for this area are:

0.5 0.4 0.4 = Swale 0.7 0.6 0.7 + ½ Detention Basin 0.25 0.25 0.3 = 0.95 0.85 1.0

0.5 0.4 0.4 = Swale 0.7 0.6 0.7

The mitigation provided by either option creates enough water quality improvement for this aspect of the development.

Cul-de-sac

The cul-de-sac will be drained by an under drained swale, with the overland flow from the carriageway discharging over a filter strip first. The mitigation for this area is:

0.5 0.4 0.4 = Filter Strip 0.4 0.4 0.5 + ½ Swale 0.35 0.3 0.35 + 1/2 Filter Drain 0.2 0.2 0.2 = 0.95 0.9 1.05

The combined SUDS features provide sufficient water quality for this aspect of the development.

All the methods above provide enough water quality in accordance with CIRIA's C753 document or a suitable compared to the existing drainage arrangements. Further evidence will need to be provided at detailed design stage.

Finished Floor Levels

The FRA highlights that the finished floor levels should be set at 3.800m AOD.

Flood Risk off-site

Most sites have to be raised in this area of development to protect against the flood zone flood risk form the site, and this site is no exception with the FFL's requiring being raised by up to a metre above existing ground levels, with the FFL's in places being raised event further to allow for a gravity drainage solution. This will mean that the finished floor levels are up to a metre higher than the existing FFL's.

If the sites drainage was to fail, or an exceedance event were to occur the flow route will be along the proposed swale location, which flow away from the proposed and existing buildings, which leads into the attenuation basin which will be fitted with a high level overflow to ensure the water levels do not reach existing property FFL's. The top of the bank of the basin is set at 3.2m AOD, with he minimum finished floor levels set at 3.3m AOD.

The rear gardens will be raised using permeable soil, and therefore rainwater in the rear gardens will filter back to the sites natural ground level. The driveways will fall towards the proposed carriageway, which has the flow routes as described above. The existing PoS is a large flat area, generally lower than the surrounding, existing FFL's and any flooding will naturally find its way to this point, which

would offer a high amount of storage. It should be noted that the PoS is not part of the drainage proposal.

As a result, the flood risk offsite has not been increased as a result of the development.

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.

The adopted areas will have their own maintenance requirements and when the site's elements become adopted it is expected that the maintenance requirements will revert back to the adopting bodies policy.

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

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.

Detention Basin

Maintenance schedule	Required action	Typical frequency
Regular maintenance	Remove litter and debris	Monthly
	Cut grass – for spillways and access routes	Monthly (during growing season), or as required
	Cut grass – meadow grass in and around basin	Half yearly (spring – before nesting season, and autumn)
	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 banksides, structures, pipework etc for evidence of physical damage	Monthly
	Inspect inlets and facility surface for silt accumulation. Establish appropriate silt removal frequencies.	Monthly (for first year), then annually or as required
	Check any penstocks and other mechanical devices	Annually
	Tidy all dead growth before start of growing season	Annually
	Remove sediment from inlets, outlet and forebay	Annually (or as required)
	Manage wetland plants in outlet pool – where provided	Annually (as set out in Chapter 23)
Occasional maintenance	Reseed areas of poor vegetation growth	As required
	Prune and trim any trees and remove cuttings	Every 2 years, or as required
	Remove sediment from inlets, outlets, forebay and main basin when required	Every 5 years, or as required (likely to be minimal requirements where effective upstream source control is provided)
Remedial actions	Repair erosion or other damage by reseeding or re-turfing	As required
	Realignment of rip-rap	As required
	Repair/rehabilitation of inlets, outlets and overflows	As required
	Relevel uneven surfaces and reinstate design levels	As required

Swale

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

Foul Proposals

The site will need to be served by foul drainage, and it is proposed that the main foul water sewers will be adopted by Anglian Water. Similar to the surface water, parts of the foul drainage will need to be positioned in private areas as they will not meet the criteria for adoption.

There is an existing Anglian Water connection point located at the junction of Sarah gate and Main Road, manhole reference 8905 - as shown in figure 10, and it is proposed that the site will connect into the sewer just downstream of this manhole – subject to confirmed suitable levels. An easement will need to be offered for the adopted foul drainage sewer where it passes through the non-adopted highway and the position of the connection ensure this will not be across third party land.

Given the depth of the existing foul drain and the site levels, similar to the surface water proposals, the foul sewer will need to be pumped.



Figure 10 - Anglian Water Asset Map Extract

It is not expected that the proposals will cause any capacity issues for the existing sewer, and even if they do, Anglian Water will be required to upgrade their system to provide capacity. A section 106 application will be submitted at detailed design stage, and upon receiving full planning permission.

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.
- Volume of run-off will not exceed pre-development greenfield run-off for the 1 in 100 year, 360 minute storm.
- The flow offsite will be restricted to a maximum of 1.5 l/s – although it is proposed at detailed design that the rate will be revised to not exceed the like for like storm, subject to detailed discussion with the IDB/pump supplier.
- Calculations have been included to represent a surcharged outfall.
- Maximum water depth in the basin is less than 0.90m.
- No flooding is currently anticipated for the 1 in 100 year + 40% climate change event.
- The proposed flow from site will not exceed the like for like greenfield event.
- 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 JCE's Preliminary Drainage Strategy drawing: 077-JCE-00-SI-DR-C-3000.
- Finished floor levels will be set at a minimum of 300mm above existing ground levels, in accordance with the FRA.

Appendices

Appendix A – Existing Topographical Survey



KEY:

- BUILDING
- FENCE
- HEDGE
- CHANNEL
- WALL
- BANK EDGE
- STORM DRAINAGE
- FOUL DRAINAGE
- CHANGE OF SURFACE
- OVERHEAD ELECTRIC
- OVERHEAD TELEPHONE

TREE BOREHOLE MANHOLE

CONTROL STATION

B	BOLLARD	RE	ROD EYE
BIN	BIN	RI	RIDGE
BT	BRITISH TELECOM	SP	SIGN POST
CR	PIPE CROWN	STAY	STAY
CTV	CABLE TELEVISION	SV	STOP VALVE
EA	EARTH	TBM	BENCH MARK
EL	ELECTRIC BOX	TH	THRESHOLD
EP	ELECTRIC POLE	TL	TRAFFIC LIGHT
EV	EAVES	TP	TELEGRAPH POLE
FH	FIRE HYDRANT	TS	TRAFFIC SIGNAL
G	GULLY	VC	VENT COLUMN
GA	GAS VALVE	VP	VENT PIPE
IC	INSPECTION CHAMBER	WL	WATER LEVEL
IL	INVERT LEVEL	WM	WATER METER
KO	KERB OFFLET	WT	WALL TOP
LH	LAMP HOLE		
LP	LAMP POST		
MP	MARKER POST		
PB	POST BOX		

NOTES:

1. Survey plotted to OS National Grid.
2. All levels related to OS Datum.
3. GPS/GLONASS ETRS89 observations transformed to OSGB36 using OSTN15 Transformation.
4. ETRS89 ellipsoid heights converted to heights above mean sea level using National Geoid Model OSGM15.

Initial Survey Date: 18 JAN 2021

CONTROL STATIONS

Stn:	Eastings:	Northings:	Height:
9000	522762.263	333092.064	3.122
9001	522784.719	333133.447	3.128
9002	522750.917	333119.846	3.048
9003	522768.725	333098.810	3.016
9004	522760.681	333088.192	3.157
9005	522756.035	333073.071	3.249
9006	522706.843	332973.384	3.608
9007	522843.895	332920.928	3.488
9008	522863.843	333025.594	2.914

Rev:	Amendments:	Date:

Client: SEAGATE HOMES

Project Title: ST.MARGARETS QUADRING LINCOLNSHIRE

Drawing Title: OGL SURVEY

Scale:	Date:
1:500	22 JAN 2021

Drawing No:	Rev:
AS2513/1	A1

C.A.D. Title: SEAGATEQUADRING180121

Drawn By: D.J.SMITH

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Appendix B – Greenfield Runoff

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Ben Jackson
Site name:	St Margarets
Site location:	Quadring

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:	52.88097° N
Longitude:	0.1758° W
Reference:	2870418528
Date:	May 03 2021 10:37

Runoff estimation approach

IH124

Site characteristics

Total site area (ha): 1.08

Methodology

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

Soil characteristics

	Default	Edited
SOIL type:	2	2
HOST class:	N/A	N/A
SPR/SPRHOST:	0.3	0.3

Hydrological characteristics

	Default	Edited
SAAR (mm):	558	558
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):	1.51	1.51
1 in 1 year (l/s):	1.31	1.31
1 in 30 years (l/s):	3.7	3.7
1 in 100 year (l/s):	5.37	5.37
1 in 200 years (l/s):	6.36	6.36

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

Appendix C – Proposed Drainage Calculations

Design Settings

Rainfall Methodology	FSR	Maximum Time of Concentration (mins)	30.00
Return Period (years)	2	Maximum Rainfall (mm/hr)	50.0
Additional Flow (%)	0	Minimum Velocity (m/s)	1.00
FSR Region	England and Wales	Connection Type	Level Soffits
M5-60 (mm)	20.000	Minimum Backdrop Height (m)	0.200
Ratio-R	0.400	Preferred Cover Depth (m)	1.200
CV	0.750	Include Intermediate Ground	✓
Time of Entry (mins)	5.00	Enforce best practice design rules	✓

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Width (mm)	Easting (m)	Northing (m)	Depth (m)
S1	0.144	5.00	3.000	1200		522829.371	333004.008	1.575
S2	0.112	5.00	3.000	1200		522844.113	333043.122	1.706
S3	0.091	5.00	3.000	1200		522854.690	333071.179	1.800
S4	0.120	5.00	3.000	1200		522868.087	333108.741	1.925
S5			3.000	1200		522869.325	333129.303	1.989
Headwall 1			3.200	2300	1500	522877.914	333129.449	2.224
Headwall 2	0.073	5.00	3.200	2300	1500	522882.554	333139.116	2.276
Pump			3.200	1350		522878.890	333147.223	2.304
Headwall 3			3.200	2300	1500	522883.322	333148.296	2.318
Headwall 4		5.00	3.200	1200		522887.876	333154.784	0.900
S9			3.200	1200		522887.843	333156.838	0.933
Headwall 5			3.200	1200		522887.876	333158.499	0.966

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	S1	S2	41.800	0.600	1.425	1.294	0.131	320.0	375	5.69	50.0
1.001	S2	S3	29.984	0.600	1.294	1.200	0.094	320.0	375	6.19	50.0
1.002	S3	S4	39.880	0.600	1.200	1.075	0.125	320.0	375	6.85	50.0
1.003	S4	S5	20.599	0.600	1.075	1.011	0.064	320.0	375	7.19	50.0
1.004	S5	Headwall 1	8.590	0.600	1.011	0.984	0.027	318.2	375	7.33	50.0
2.000	Headwall 2	Pump	8.897	0.600	0.924	0.896	0.028	320.0	375	5.15	50.0
2.001	Pump	Headwall 3	4.560	0.600	0.896	0.882	0.014	320.0	375	5.22	50.0
3.000	Headwall 4	S9	5.000	0.600	2.300	2.267	0.033	150.0	150	5.10	50.0
3.001	S9	Headwall 5	5.000	0.600	2.267	2.234	0.033	150.0	150	5.20	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)
1.000	1.007	111.2	19.5	1.200	1.331	0.144	0.0	106	0.764
1.001	1.007	111.2	34.7	1.331	1.425	0.256	0.0	143	0.893
1.002	1.007	111.2	47.0	1.425	1.550	0.347	0.0	170	0.967
1.003	1.007	111.2	63.3	1.550	1.614	0.467	0.0	203	1.038
1.004	1.010	111.6	63.3	1.614	1.841	0.467	0.0	203	1.041
2.000	1.007	111.2	9.9	1.901	1.929	0.073	0.0	75	0.630
2.001	1.007	111.2	9.9	1.929	1.943	0.073	0.0	75	0.630
3.000	0.818	14.5	0.0	0.750	0.783	0.000	0.0	0	0.000
3.001	0.818	14.5	0.0	0.783	0.816	0.000	0.0	0	0.000

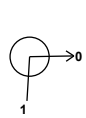
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Pipeline Schedule

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	41.800	320.0	375	Circular	3.000	1.425	1.200	3.000	1.294	1.331
1.001	29.984	320.0	375	Circular	3.000	1.294	1.331	3.000	1.200	1.425
1.002	39.880	320.0	375	Circular	3.000	1.200	1.425	3.000	1.075	1.550
1.003	20.599	320.0	375	Circular	3.000	1.075	1.550	3.000	1.011	1.614
1.004	8.590	318.2	375	Circular	3.000	1.011	1.614	3.200	0.984	1.841
2.000	8.897	320.0	375	Circular	3.200	0.924	1.901	3.200	0.896	1.929
2.001	4.560	320.0	375	Circular	3.200	0.896	1.929	3.200	0.882	1.943
3.000	5.000	150.0	150	Circular	3.200	2.300	0.750	3.200	2.267	0.783
3.001	5.000	150.0	150	Circular	3.200	2.267	0.783	3.200	2.234	0.816

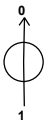
Link	US Node	Dia (mm)	Width (mm)	Node Type	MH Type	DS Node	Dia (mm)	Width (mm)	Node Type	MH Type
1.000	S1	1200		Manhole	Adoptable	S2	1200		Manhole	Adoptable
1.001	S2	1200		Manhole	Adoptable	S3	1200		Manhole	Adoptable
1.002	S3	1200		Manhole	Adoptable	S4	1200		Manhole	Adoptable
1.003	S4	1200		Manhole	Adoptable	S5	1200		Manhole	Adoptable
1.004	S5	1200		Manhole	Adoptable	Headwall 1	2300	1500	Manhole	Adoptable
2.000	Headwall 2	2300	1500	Manhole	Adoptable	Pump	1350		Manhole	Adoptable
2.001	Pump	1350		Manhole	Adoptable	Headwall 3	2300	1500	Manhole	Adoptable
3.000	Headwall 4	1200		Manhole	Adoptable	S9	1200		Manhole	Adoptable
3.001	S9	1200		Manhole	Adoptable	Headwall 5	1200		Manhole	Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Width (mm)	Connections	Link	IL (m)	Dia (mm)
S1	522829.371	333004.008	3.000	1.575	1200					
							0	1.000	1.425	375
S2	522844.113	333043.122	3.000	1.706	1200			1	1.000	1.294	375
								0	1.001	1.294	375
S3	522854.690	333071.179	3.000	1.800	1200			1	1.001	1.200	375
								0	1.002	1.200	375
S4	522868.087	333108.741	3.000	1.925	1200			1	1.002	1.075	375
								0	1.003	1.075	375
S5	522869.325	333129.303	3.000	1.989	1200			1	1.003	1.011	375
								0	1.004	1.011	375
Headwall 1	522877.914	333129.449	3.200	2.224	2300	1500		1	1.004	0.984	375

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Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Width (mm)	Connections	Link	IL (m)	Dia (mm)
Headwall 2	522882.554	333139.116	3.200	2.276	2300	1500				
							0	2.000	0.924	375
Pump	522878.890	333147.223	3.200	2.304	1350		1	2.000	0.896	375
										
							0	2.001	0.896	375
Headwall 3	522883.322	333148.296	3.200	2.318	2300	1500	1	2.001	0.882	375
										
Headwall 4	522887.876	333154.784	3.200	0.900	1200					
							0	3.000	2.300	150
S9	522887.843	333156.838	3.200	0.933	1200		1	3.000	2.267	150
										
							0	3.001	2.267	150
Headwall 5	522887.876	333158.499	3.200	0.966	1200		1	3.001	2.234	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 (m³/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	10	0

Node S9 Online Hydro-Brake® Control

Flap Valve	✓	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	2.267	Product Number	CTL-SHE-0061-1500-0800-1500
Design Depth (m)	0.800	Min Outlet Diameter (m)	0.075
Design Flow (l/s)	1.5	Min Node Diameter (mm)	1200

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Node Pump Online Pump Control

Flap Valve	x	Design Depth (m)	2.000	Switch off depth (m)	0.000
Replaces Downstream Link	✓	Design Flow (l/s)	5.0		
Invert Level (m)	0.896	Switch on depth (m)	0.100		

Depth (m)	Flow (l/s)
2.000	5.000

Node Headwall 2 Flow through Pond Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Main Channel Length (m)	10.500
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	0.924	Main Channel Slope (1:X)	200.0
Safety Factor	2.0	Time to half empty (mins)		Main Channel n	0.035

Inlets

Headwall 1

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	63.0	0.0	2.000	410.0	0.0

Node S2 Link Surround Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Link	1.000
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	1.294	Surround Shape	(Trench)
Safety Factor	2.0	Time to half empty (mins)	169	Diameter (mm)	700

Node S3 Link Surround Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Link	1.001
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	1.200	Surround Shape	(Trench)
Safety Factor	2.0	Time to half empty (mins)		Diameter (mm)	700

Node S4 Link Surround Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Link	1.002
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	1.075	Surround Shape	(Trench)
Safety Factor	2.0	Time to half empty (mins)		Diameter (mm)	700

Node S5 Link Surround Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Link	1.003
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	1.011	Surround Shape	(Trench)
Safety Factor	2.0	Time to half empty (mins)		Diameter (mm)	700

Node Headwall 4 Flow through Pond Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Main Channel Length (m)	8.000
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	2.300	Main Channel Slope (1:X)	200.0
Safety Factor	2.0	Time to half empty (mins)		Main Channel n	0.035

Inlets

Headwall 3

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Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	144.0	0.0	0.900	321.0	0.0

Node S2 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	2.700
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	23.0	0.0	0.300	76.0	0.0

Node S3 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	2.700
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	27.0	0.0	0.300	86.0	0.0

Node S4 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	2.700
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	37.0	0.0	0.300	117.0	0.0

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Results for 1 year Critical Storm Duration. Lowest mass balance: 87.02%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	S1	10	1.530	0.105	20.3	0.1192	0.0000	OK
720 minute winter	S2	555	1.500	0.206	3.3	0.9797	0.0000	OK
720 minute winter	S3	555	1.500	0.300	4.5	1.2258	0.0000	OK
720 minute winter	S4	555	1.500	0.425	5.4	2.2872	0.0000	SURCHARGED
720 minute winter	S5	555	1.500	0.489	4.9	1.8459	0.0000	SURCHARGED
720 minute winter	Headwall 1	555	1.499	0.523	4.6	0.2120	0.0000	OK
720 minute winter	Headwall 2	555	1.500	0.576	3.4	1.9864	0.0000	SURCHARGED
720 minute winter	Pump	555	1.500	0.604	1.5	0.8640	0.0000	SURCHARGED
1440 minute winter	Headwall 3	1470	2.585	1.703	1.5	5.8746	0.0000	OK
1440 minute winter	Headwall 4	1470	2.585	0.285	1.3	0.3221	0.0000	SURCHARGED
1440 minute winter	S9	1470	2.584	0.317	1.3	0.3589	0.0000	SURCHARGED
15 minute summer	Headwall 5	1	2.534	0.300	0.0	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)
15 minute winter	S1	1.000	S2	19.8	0.615	0.178	1.3765	
720 minute winter	S2	1.001	S3	3.3	0.387	0.030	2.3443	
720 minute winter	S3	1.002	S4	3.9	0.329	0.035	4.0829	
720 minute winter	S4	1.003	S5	4.9	0.325	0.044	2.2720	
720 minute winter	S5	1.004	Headwall 1	4.6	0.364	0.041	0.9475	
720 minute winter	Headwall 1	Flow through pond	Headwall 2	2.5	0.013	0.000	60.9632	
720 minute winter	Headwall 2	2.000	Pump	1.5	0.070	0.014	0.9813	
720 minute winter	Pump	Pump	Headwall 3	1.5				
1440 minute winter	Headwall 3	Flow through pond	Headwall 4	1.3	0.005	0.000	45.1121	
1440 minute winter	Headwall 4	3.000	S9	1.3	0.073	0.089	0.0880	
1440 minute winter	S9	Hydro-Brake®	Headwall 5	1.3				46.1

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Results for 2 year Critical Storm Duration. Lowest mass balance: 87.02%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
720 minute winter	S1	570	1.582	0.157	2.2	0.1780	0.0000	OK
720 minute winter	S2	570	1.582	0.288	3.9	1.4368	0.0000	OK
720 minute winter	S3	570	1.582	0.382	4.9	1.6383	0.0000	SURCHARGED
720 minute winter	S4	570	1.582	0.507	6.1	2.9769	0.0000	SURCHARGED
720 minute winter	S5	570	1.582	0.571	5.5	2.2969	0.0000	SURCHARGED
720 minute winter	Headwall 1	570	1.582	0.605	5.4	0.2842	0.0000	OK
720 minute winter	Headwall 2	570	1.582	0.658	4.0	2.2711	0.0000	SURCHARGED
720 minute winter	Pump	570	1.582	0.686	1.7	0.9821	0.0000	SURCHARGED
1440 minute winter	Headwall 3	1560	2.609	1.727	1.7	5.9588	0.0000	OK
1440 minute winter	Headwall 4	1560	2.609	0.309	1.5	0.3497	0.0000	SURCHARGED
1440 minute winter	S9	1560	2.609	0.342	1.4	0.3865	0.0000	SURCHARGED
15 minute summer	Headwall 5	1	2.534	0.300	0.0	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)
720 minute winter	S1	1.000	S2	2.2	0.319	0.020	2.8168	
720 minute winter	S2	1.001	S3	3.6	0.397	0.032	3.0171	
720 minute winter	S3	1.002	S4	4.2	0.329	0.038	4.3986	
720 minute winter	S4	1.003	S5	5.5	0.342	0.050	2.2720	
720 minute winter	S5	1.004	Headwall 1	5.4	0.377	0.048	0.9475	
720 minute winter	Headwall 1	Flow through pond	Headwall 2	2.9	0.012	0.000	74.6652	
720 minute winter	Headwall 2	2.000	Pump	1.7	0.049	0.015	0.9813	
720 minute winter	Pump	Pump	Headwall 3	1.7				
1440 minute winter	Headwall 3	Flow through pond	Headwall 4	1.5	0.004	0.000	49.9799	
1440 minute winter	Headwall 4	3.000	S9	1.4	0.078	0.095	0.0880	
1440 minute winter	S9	Hydro-Brake®	Headwall 5	1.4				53.1

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Results for 30 year Critical Storm Duration. Lowest mass balance: 87.02%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
600 minute winter	S1	570	1.939	0.514	4.5	0.5819	0.0000	SURCHARGED
600 minute winter	S2	570	1.939	0.645	7.5	4.4391	0.0000	SURCHARGED
600 minute winter	S3	570	1.939	0.739	9.6	4.2028	0.0000	SURCHARGED
600 minute winter	S4	570	1.939	0.864	13.0	6.3714	0.0000	SURCHARGED
600 minute winter	S5	570	1.939	0.928	12.4	4.2449	0.0000	SURCHARGED
600 minute winter	Headwall 1	570	1.939	0.962	12.0	0.7184	0.0000	OK
600 minute winter	Headwall 2	570	1.939	1.015	8.3	3.5029	0.0000	SURCHARGED
600 minute winter	Pump	570	1.939	1.043	2.7	1.4930	0.0000	SURCHARGED
1440 minute winter	Headwall 3	1680	2.777	1.895	2.5	6.5380	0.0000	OK
1440 minute winter	Headwall 4	1680	2.777	0.477	2.0	0.5397	0.0000	SURCHARGED
1440 minute winter	S9	1680	2.777	0.510	1.5	0.5763	0.0000	SURCHARGED
15 minute summer	Headwall 5	1	2.534	0.300	0.0	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)
600 minute winter	S1	1.000	S2	4.0	0.350	0.036	4.6104	
600 minute winter	S2	1.001	S3	6.8	0.405	0.061	3.3071	
600 minute winter	S3	1.002	S4	9.2	0.353	0.083	4.3986	
600 minute winter	S4	1.003	S5	12.4	0.418	0.111	2.2720	
600 minute winter	S5	1.004	Headwall 1	12.0	0.440	0.108	0.9475	
600 minute winter	Headwall 1	Flow through pond	Headwall 2	6.5	0.023	0.000	147.3977	
600 minute winter	Headwall 2	2.000	Pump	2.7	0.192	0.024	0.9813	
600 minute winter	Pump	Pump	Headwall 3	2.6				
1440 minute winter	Headwall 3	Flow through pond	Headwall 4	2.0	0.006	0.000	86.4891	
1440 minute winter	Headwall 4	3.000	S9	1.5	0.086	0.104	0.0880	
1440 minute winter	S9	Hydro-Brake®	Headwall 5	1.5				71.5

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Results for 100 year Critical Storm Duration. Lowest mass balance: 87.02%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
600 minute winter	S1	570	2.142	0.717	5.8	0.8107	0.0000	SURCHARGED
600 minute winter	S2	570	2.142	0.848	10.1	6.4433	0.0000	SURCHARGED
600 minute winter	S3	570	2.142	0.942	13.1	5.7071	0.0000	SURCHARGED
600 minute winter	S4	570	2.142	1.067	17.4	8.2945	0.0000	SURCHARGED
600 minute winter	S5	570	2.142	1.131	16.7	5.3493	0.0000	SURCHARGED
600 minute winter	Headwall 1	570	2.141	1.165	16.3	1.0521	0.0000	OK
600 minute winter	Headwall 2	570	2.142	1.218	9.7	4.2008	0.0000	SURCHARGED
600 minute winter	Pump	570	2.141	1.245	3.2	1.7822	0.0000	SURCHARGED
1440 minute winter	Headwall 3	1680	2.887	2.005	3.0	6.9186	0.0000	OK
1440 minute winter	Headwall 4	1680	2.887	0.587	5.0	0.6643	0.0000	SURCHARGED
1440 minute winter	S9	1680	2.887	0.620	1.5	0.7011	0.0000	SURCHARGED
15 minute summer	Headwall 5	1	2.534	0.300	0.0	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)
600 minute winter	S1	1.000	S2	5.6	0.353	0.051	4.6104	
600 minute winter	S2	1.001	S3	9.4	0.396	0.084	3.3071	
600 minute winter	S3	1.002	S4	12.5	0.384	0.113	4.3986	
600 minute winter	S4	1.003	S5	16.7	0.436	0.150	2.2720	
600 minute winter	S5	1.004	Headwall 1	16.3	0.462	0.146	0.9475	
600 minute winter	Headwall 1	Flow through pond	Headwall 2	6.9	0.024	0.000	198.3990	
600 minute winter	Headwall 2	2.000	Pump	3.2	0.199	0.029	0.9813	
600 minute winter	Pump	Pump	Headwall 3	3.1				
1440 minute winter	Headwall 3	Flow through pond	Headwall 4	5.0	0.007	0.000	113.4449	
1440 minute winter	Headwall 4	3.000	S9	1.5	0.107	0.104	0.0880	
1440 minute winter	S9	Hydro-Brake®	Headwall 5	1.5				76.4

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Results for 100 year +40% CC +10% A Critical Storm Duration. Lowest mass balance: 87.02%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	S1	12	2.737	1.312	99.4	1.4843	0.0000	FLOOD RISK
15 minute winter	S2	12	2.670	1.376	172.4	11.6709	0.0000	SURCHARGED
480 minute winter	S3	456	2.605	1.405	24.6	9.1488	0.0000	SURCHARGED
480 minute winter	S4	456	2.606	1.531	32.6	12.7071	0.0000	SURCHARGED
480 minute winter	S5	456	2.608	1.597	31.4	7.8934	0.0000	SURCHARGED
480 minute winter	Headwall 1	480	2.606	1.629	536.8	2.0589	0.0000	OK
480 minute winter	Headwall 2	472	2.591	1.667	233.4	5.7527	0.0000	SURCHARGED
720 minute winter	Pump	660	2.598	1.702	4.3	2.4349	0.0000	SURCHARGED
1440 minute winter	Headwall 3	1680	3.170	2.288	4.1	7.8932	0.0000	OK
1440 minute winter	Headwall 4	1680	3.166	0.866	2.8	0.9793	0.0000	FLOOD RISK
1440 minute winter	S9	1680	3.167	0.900	1.7	1.0177	0.0000	FLOOD RISK
15 minute summer	Headwall 5	1	2.534	0.300	0.0	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)
15 minute winter	S1	1.000	S2	95.2	0.863	0.855	4.6104	
15 minute winter	S2	1.001	S3	130.3	1.181	1.171	3.3071	
480 minute winter	S3	1.002	S4	23.6	0.456	0.212	4.3986	
480 minute winter	S4	1.003	S5	31.4	0.515	0.282	2.2720	
480 minute winter	S5	1.004	Headwall 1	30.6	0.549	0.274	0.9475	
480 minute winter	Headwall 1	Flow through pond	Headwall 2	-513.2	0.027	-0.006	336.1033	
480 minute winter	Headwall 2	2.000	Pump	9.8	0.089	0.088	0.9813	
720 minute winter	Pump	Pump	Headwall 3	4.3				
1440 minute winter	Headwall 3	Flow through pond	Headwall 4	2.8	0.005	0.000	192.8653	
1440 minute winter	Headwall 4	3.000	S9	1.7	0.098	0.120	0.0880	
1440 minute winter	S9	Hydro-Brake®	Headwall 5	1.5				81.1

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Appendix D – Proposed Drainage Plan



Drainage Key

- Proposed adoptable surface drainage (pipe size/gradient, manhole reference & size)
- Proposed adoptable foul drainage (pipe size/gradient, manhole reference & size)
- Proposed adoptable foul riser main
- Proposed private foul water sewer (indicative)
- Proposed private surface drainage (indicative)
- Existing Anglian Water surface sewer with references
- Existing Anglian Water foul sewer with references
- Proposed Anglian Water easement
- FFL 7.700 (7.55) — Finished Floor Level / Proposed Level / (Approx. Existing Level)
- Attenuation Basin — The main method of attenuation, restricting the site to greenfield run-off rates is via. a Hydrobrake which slows the flow and is stored within this basin.

  It is intended that this basin will be online, and as the site is generally designed in accordance with a SuDS treatment train it will be adopted by Anglian Water - to be determined at the detailed design phase.
- Denotes proposed surface flow routes.

The drainage strategy is preliminary and subject to full detail drainage and level design.

Discharge rate to be agreed with the IDB, and subject to written approval.

The existing Anglian Water foul sewer's invert level is to be confirmed, including the pipe diameter.

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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. This drawing is not to be scaled.
5. 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.Drawings to be read in conjunction with all other relevant drawings.
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 with 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 cover 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. The junctions from RWP's, SVP's and similar connections into main runs are to be determined on site, and in accordance with building regulations.
20. All RWP's, to be fitted with rodding access.
21. 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.
22. 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.
23. Backdrops might be required to ensure that building drainage above foundations are able to enter the PPIC - refer to details.
24. 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.
25. All RWP's and below ground foul drainage to be set out from the Architect's drawings.
26. Where drainage gradients have been stated, these represent the minimum gradients, and can be steeper - subject to a maximum gradient of 1:10.
27. If the contractor is unsure which manhole/PPIC reference is being used for the actual manhole/PPIC, ask!
28. Cover levels are approximate only and must suit the finished external levels.
29. Threshold drains are to the Architect's details.
30. Rear patios to fall towards soft landscaped areas.

Preliminary
For client approval

Rev	Description	Drn	Ck'd	Date
P03	Minor updates.	BJ	JCE	10/05/23
P02	Site plan and drainage strategy updated.	BJ	JCE	03/03/23
P01	Site plan updated.	BJ	JCE	03/09/21

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Land of St Margarets,
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Drawing Title:
Drainage General Arrangement

Drawing Status:
Preliminary

Scale: 1:500	Paper size: A1	Date:03/05/2021
Drawn: BJ	Checked: JCE	Approved: JCE

Drawing Number:
077-JCE-00-SI-DR-C-3000

Rev
P03