JACKSON
CONSULTING ENGINEERS

Proposed Drainage Strategy

Project:

Proposed Commercial Development, Cresswell Close, Pinchbeck

Client:

Industro Tech Ltd

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Introduction

Jackson consulting engineers have been appointed by Industro Tech Ltd, to undertake a drainage strategy forming part of the supporting evidence for planning purposed for the proposed industrial re-development, to land off unit 5, Cresswell Close, Pinchbeck, PE11 3TY. 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 at the end of Cresswell Close, located to the north. The site is located in an existing business park in Pinchbeck. An aerial location plan, with approximate site boundary is shown in figure 1, noting that the redline boundary does not constitute as the planning boundary. The site is currently brownfield, with a total site area of approximately 0.21 hectares.



Figure 1 - Site Location

The architectural consultant has undertaken a brief level survey for this project, which will be supplied as part of the supporting information for the project (by the third party). As expected in this area, the site is relatively flat, although the survey is to a temporary datum – set at 10m, it is expected the ground level is roughly 3m AOD.

The existing concrete yard drains to gullies, and the roof to traditional gutters and rain water pipes. Approximately three quarters of the site is impermeable, positively drained, with the rest of the site being laid to gravel and hardcore.

Flood Risk

Requirement of a Flood Risk Assessment

As part of the due process of planning permission, a flood risk assessment has been undertaken by a separate consultant. The FRA will be provided as part of the overall planning package.

Development Proposals

The development involves the creation of a proposed commercial building and the increase of the car parking/vehicle delivery area.

Proposed Drainage Strategy

The following section provides narrative on the principles behind the drainage strategy and has been carried out in general accordance with 'CIRIA's C753 – the SUDS Manual' and DEFRA's 'non-technical standards', 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 PPG, 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 is not available for the site. A desktop study via a review of BGS mapping shows that the bedrock geology is 'OXFORD CLAY FORMATION - MUDSTONE', with the superficial deposits being described as 'TIDAL FLAT DEPOSITS – CLAY AND SILT'. Evidence of these descriptions are shown in the respective figures below.

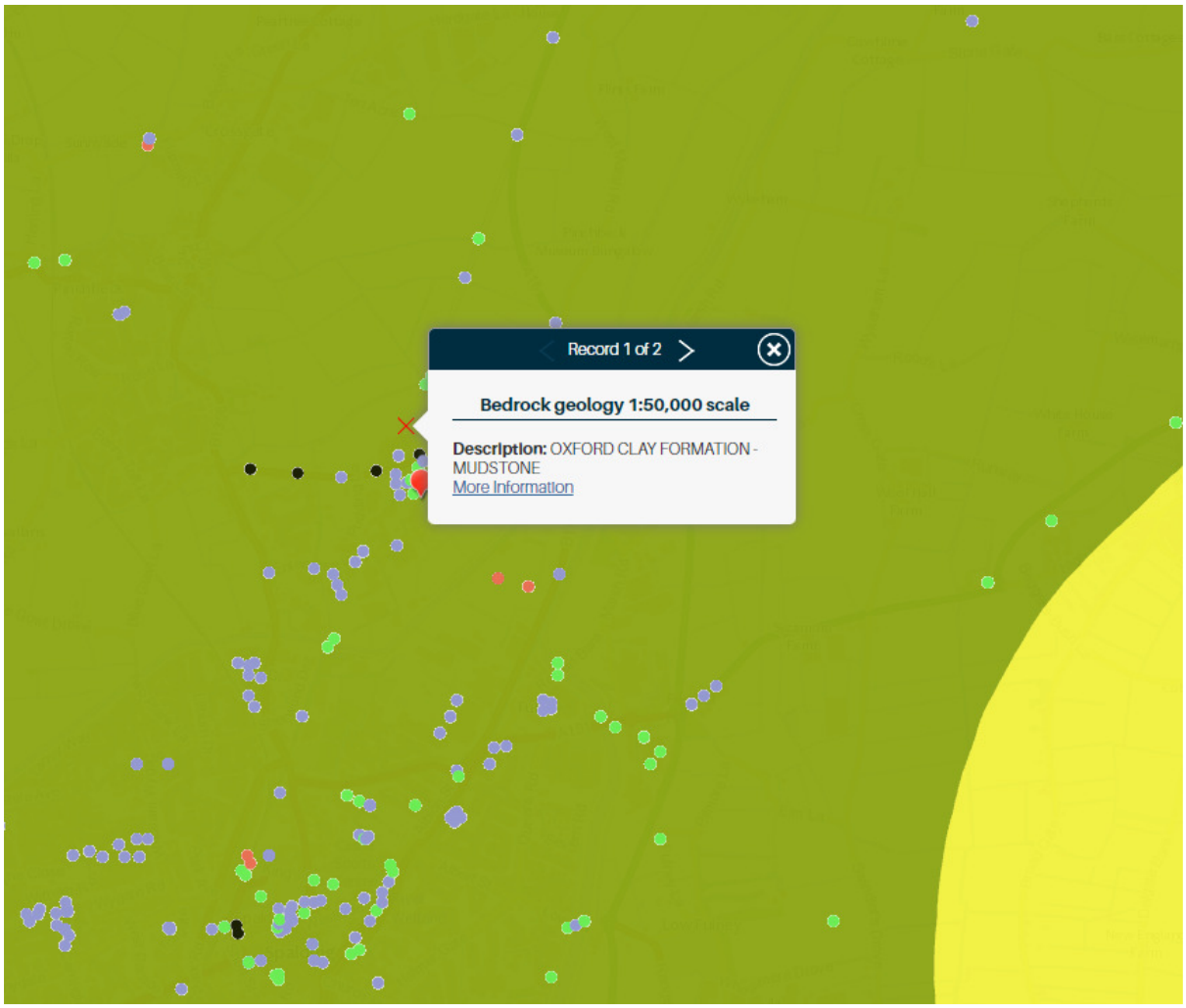


Figure 2 - Bedrock Geology



Figure 3 - Superficial Deposits

Additionally, as seen in the figures, there are a large number of borehole scans located in close proximity. The logs match the descriptions, and the closest borehole shows the ground to be 'very sandy clay, high silt content' – this description would tend to indicate a poor infiltration rate, with substantial long term issues.

More fundamentally, water was struck at 1.90m, with the water rising to 1.10 over the 5 hours the borehole was exposed. Soakaways need to have 1m from base of device to the top of the water-table, something that it does not achieve.

At present, infiltration would be ruled out due to the proximity of the ground water to the surface, and therefore is discounted as part of this strategy. It is expected that this is fully explored through the phase 2 ground investigation report.

Watercourse

There are no watercourses located in the vicinity of the site and as a result of the watercourses not being readily accessible, they have been ruled out. It is generally accepted that the surface water will eventually discharge into a drain or watercourse given the nature of this area of Lincolnshire.

Surface Water Sewer

As shown in the figure below, there are both foul and surface water sewers in the vicinity of the site. It is assumed that these sewers were designed to accommodate the overall business park. These manholes have significant size and depth.

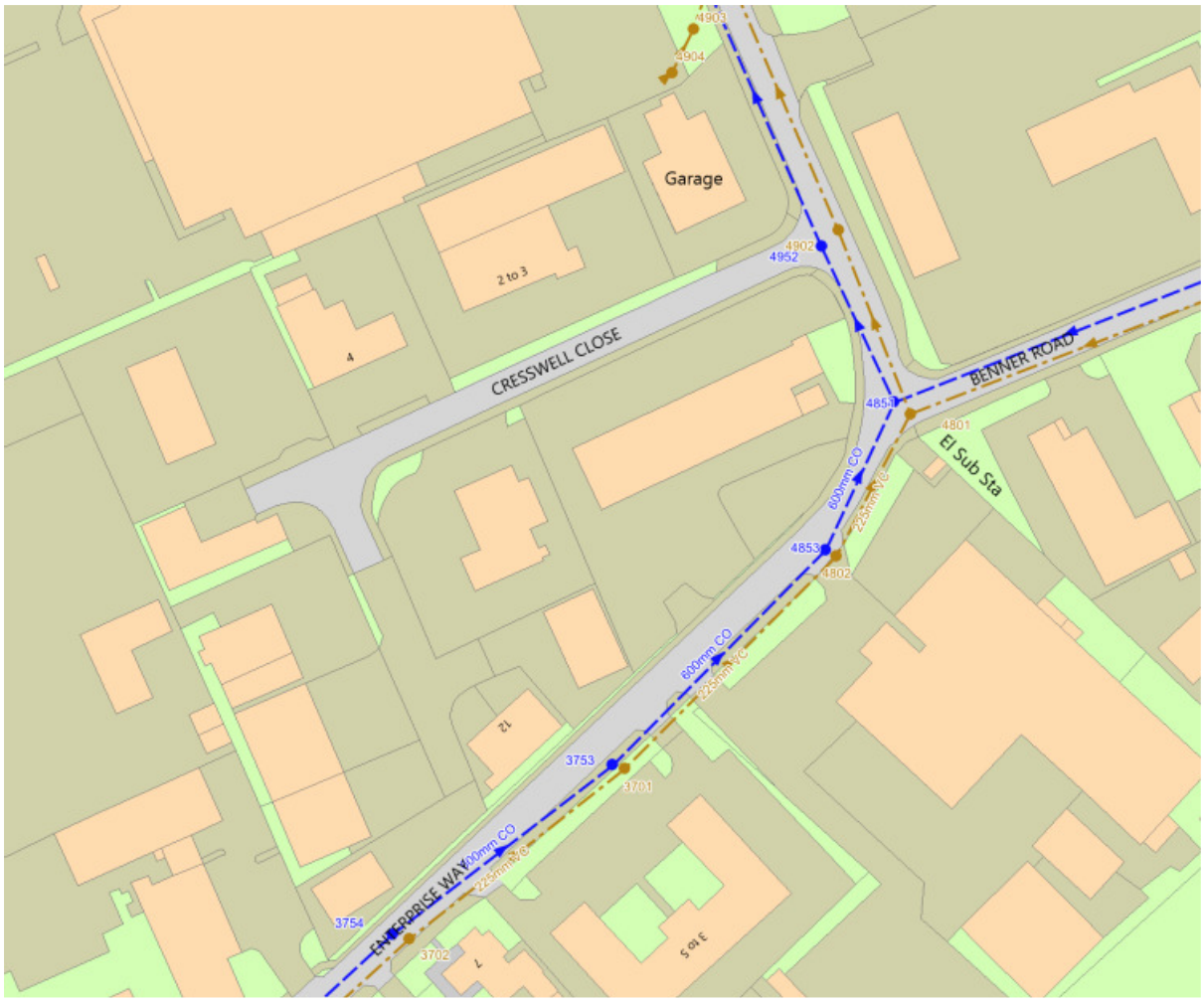


Figure 4 - Anglian Water Assets

Cresswell Close itself has both foul and surface water manholes which enter into 4902 and 4952 respectively. These have not been identified on the map as being adopted by AW, but given the age of the sewers, it is assumed that these would have been transferred to AW as part of the transfer of sewers. These manholes/sewers have been confirmed onsite.

The existing sites drainage has been briefly surveyed, and both the foul and surface water sewers have been identified to head into the existing sewers in Cresswell Close. The drainage from the site does

not connect directly into the manhole and instead connect via. junctions. Therefore the existing site has existing positive drainage.

Existing Greenfield Run Off Rates

Utilising HR Wallingford's greenfield runoff calculator, for a site are of 2100m^2, the discharge rates is shown in the figure below. For the sites size, this is generally deemed to be too small flow, and it is proposed than a peak discharge from site is 2l/s, to reduce the risk of device blockage – this is inline with AW's policy. The existing runoff rates are shown in the below figure.

Greenfield runoff rates	Default	Edited
Q_{BAR} (l/s):	0.29	0.29
1 in 1 year (l/s):	0.26	0.26
1 in 30 years (l/s):	0.72	0.72
1 in 100 year (l/s):	1.05	1.05
1 in 200 years (l/s):	1.24	1.24

Figure 5 - Greenfield Runoff Rates

Furthermore, as the site is currently positively drained, there may be scope to keep the flows even higher than greenfield runoff, but still in line with DEFRA's requirements. This is subject to a negotiated discharge rate allowance from AW.

Existing Brownfield Run Off Rates

The existing site is served by a 100 dia. surface water pipe. It is deemed that this existing pipe would tend not to have capacity for the existing site, and it would be expected that some flooding would occur – although this would be stored above ground before entering the system. Again, the small size of the pipe is due to the age of the site, but not flooding issues are presently recorded, and any flooding would likely on cause minor surface inconvenience and discharge quickly.

It is estimated that 1400m^2 of the existing hardstanding is impermeable and positively drained. A crude drainage model has been designed to determine the existing flows. As suspected, the 100mm dia. pipe is at capacity for all design storms, having an estimated flow of 15.8l/s. The flow rate is the same for all design storms as the pipe is at capacity and flooding is present (the maximum head is achieved, and this cannot increase due to the flooding). As a note, the pipe gradient is based on 1 in 100, which is likely to be steeper than this, and return a higher existing discharge rate. As the junction level is not known, the flattest allowable sewer gradient has been assumed.

The simple existing drainage model can be found in appendix A.

Proposed Run Off Rates

In accordance with DEFRA's non-technical standards, in particular standard S3, the following is expected:

> **S3** For developments which were previously developed, the peak runoff rate from the development to any drain, sewer or surface water body for the 1 in 1 year rainfall event and the 1 in 100 year rainfall event must be as close as reasonably practicable to the greenfield runoff rate from the development for the same rainfall event, but should never exceed the rate of discharge from the development prior to redevelopment for that event.

Figure 6 - DEFRA Peak Flow Control Requirements

As previously stated, it is likely that AW will only accept a flow rate or 2l/s. Given the nature of flow control devices, the discharge rate from site will be the same for lower storm return periods. This is due to the low hydrostatic head for the development. As the site is being restricted to no greater than 2l/s for all storms up to and including the 1 in 100 year + 40% climate change event, the summary of different flow rates for the return periods are:

Storm	Critical Flow
1 Year	2.0 l/s
2 Year	2.0 l/s
30 Year	2.0 l/s
100 Year	2.0 l/s
100 Year + 40% CC	2.0 l/s

Figure 7 - Post Development Flow Rates

As a result, the post development has a betterment of 88% - based on a post development discharge of 2l/s. Even if a higher rate, or say 5l/s can be negotiated with AW, this represents a betterment of 68%. Both are significant savings against the existing arrangements.

Additionally, the existing model suggest the site floods from the 1 in 1 storm through to the maximum design storm of 1 in 100 year + 40% climate change, showing flooding of approximate $1m^3$ to $28m^3$ respectively. The post development scheme shows no flooding for all storms up to and including the 1 in 100 year + 40% climate change event.

In summary, although the greenfield run off rate is not realistically achievable, the site is being heavily reduced compared to the current scenario, with a betterment of 88% being achieved. This is fully compliant with the non technical standards.

Proposed Drainage Design

The proposed drainage layout is shown on JCE drawing 0288-JCE-00-SI-SK-C-3000, which is available as part of the overall package of information.

It is proposed that the main service yard will be constructed out of heavy duty concrete – this is owing to the heavy usage of the yard. The concrete yard will be laid towards the porous paved parking bays.

The parking bays will be limited to lightweight vehicles, and this will be suitable for porous paving. In these instances, it is proposed that the concrete yard will discharge towards the permeable paving in a maximum ratio of 1:2 (permeable paving:impermeable paving).

Dependant on the final level design, some of the 1:2 ratios may be exceeded. Where exceeded, it is proposed that additional drainage features, such as filter drains or tree pits will be put forward. – these have been indicatively shown on the drainage plan.

The front elevation roof water will discharge into the porous sub-base via. catchpit and diffuser boxes. The rear of the roof will discharge into an under drained filter drain.

To provide the necessary storage required, it is proposed that the entire yard area – both concrete and porous paving, will be installed to a porous sub-base. The voided sub-base provides 30% void ratio. Preliminary calculations show that a minimum thickness of 300mm is required to provide the necessary attenuation. At detailed design and through the tendering process, it may be more cost effective to provide porous sub-base to the porous parking bays only, and add attenuation tanks to areas. This ethos would still be in line with the current strategy. The formation will generally be flat, so the minimum 300mm thickness will be increased locally to generate the surface falls.

All of the above elements will then collect into a below ground piped system, and attenuated via. a flow control – provisionally a Hydrobrake device, before entering the suspected AW sewer.

The preliminary calculations can be found in appendix B.

Flood Routing

The scheme has been designed so that no above ground flooding is created in the 1 in 100 year + 40% storm event, and therefore a plan is not required to represent the volumes, depths, velocities or extents of flooding. Typical surface water flow routes have been shown on the drainage drawing.

The FFL's will be elevated above existing ground and the proposed hardstanding falls away from the building – which mimics the existing levels. If the device were to fail, or the design storm exceeded, the water will flow away from the buildings towards the carriageway.

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:

- Industrial Roofs - Low
- Non-residential parking - Medium
- Commercial Yard and delivery area - Medium

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.

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:

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 achieved in the right-hand side column. The right-hand side's total must be higher than the left-hand side.

Roofs – Porous Paving

0.3 0.2 0.05 = Porous Paving Subbase 0.7 0.6 0.7

The roof water to the front elevation will discharge into the porous paving sub-base before entering the piped system, this will provide adequate pollution mitigation and is acceptable.

Non-residential parking

0.7 0.6 0.7 = Porous Paving 0.7 0.6 0.7

Porous paving provides sufficient water quality for the proposed areas and is deemed sufficient.

Commercial yard (porous paving)

0.7 0.6 0.7 = Porous Paving 0.7 0.6 0.7

Porous paving provides sufficient water quality for the proposed areas and is deemed sufficient.

All the methods above provide enough water quality in accordance with CIRIA's C753 document.

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.

Porous Paving

Maintenance schedule	Required action	Typical frequency
Regular maintenance	Brushing and vacuuming (standard cosmetic sweep over whole surface)	Once a year, after autumn leaf fall, or reduced frequency as required, based on site-specific observations of clogging or manufacturer's recommendations – pay particular attention to areas where water runs onto pervious surface from adjacent impermeable areas as this area is most likely to collect the most sediment
Occasional maintenance	Stabilise and mow contributing and adjacent areas	As required
	Removal of weeds or management using glyphospate applied directly into the weeds by an applicator rather than spraying	As required – once per year on less frequently used pavements
Remedial Actions	Remediate any landscaping which, through vegetation maintenance or soil slip, has been raised to within 50 mm of the level of the paving	As required
	Remedial work to any depressions, rutting and cracked or broken blocks considered detrimental to the structural performance or a hazard to users, and replace lost jointing material	As required
	Rehabilitation of surface and upper substructure by remedial sweeping	Every 10 to 15 years or as required (if infiltration performance is reduced due to significant clogging)
Monitoring	Initial inspection	Monthly for three months after installation
	Inspect for evidence of poor operation and/or weed growth – if required, take remedial action	Three-monthly, 48 h after large storms in first six months
	Inspect silt accumulation rates and establish appropriate brushing frequencies	Annually
	Monitor inspection chambers	Annually

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.

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

Similar to the surface water, there is an existing foul sewer to the south of the site, which is suspected to be an undocumented AW sewer. It is proposed that the proposed site will discharge into this existing sewer, via. an existing manhole.

The 2 industrial units are unlikely to significantly increase the foul load to the sewer, and therefore, it is not expected that AW will object to the proposals.

At detailed design, once the internal foul requirements are known, a detailed application will be submitted to AW via. a S106 application.

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 ensure they comply with DEFRA's, Anglian Water and LLFA standards.
- No above ground flooding is present for the 1 in 100 year + 40% climate change event.
- The proposed scheme does not increase the risk of flooding either on or off site as a result of the development, as storm water for all design storms are kept on site.
- 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: 0288-JCE-00-SI-SK-C-3000.
- Finished floor levels will be set at a minimum of 150mm above ground levels.
- It is proposed at detailed design a flow rate of 5l/s will be requested from AW, although the preliminary calcs have been designed to suit 2l/s. Both options have betterments of 68% and 88% respectively.
- The existing drainage model shows flooding in all design storm events, although no nuisance flooding has been reported. It is likely that the flooding is limited to the parking areas/soft landscaping and dissipates quickly.

Appendices

Appendix A – Existing Drainage Calculations

Appendix B – Proposed Drainage Calculations

Design Settings

Rainfall Methodology	FSR	Maximum Time of Concentration (mins)	30.00
Return Period (years)	1	Maximum Rainfall (mm/hr)	1.0
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)	Easting (m)	Northing (m)	Depth (m)
S1	0.140	5.00	10.000	1200	564296.684	394247.151	0.920
Ex. S1			10.000	1200	564307.684	394242.242	1.040

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	Ex. S1	12.046	0.600	9.080	8.960	0.120	100.0	100	5.26	1.0

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	0.769	6.0	0.4	0.820	0.940	0.140	0.0	17	0.420

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	12.046	100.0	100	Circular	10.000	9.080	0.820	10.000	8.960	0.940

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	S1	1200	Manhole	1 Adoptable	Ex. S1	1200	Manhole	1 Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
S1	564296.684	394247.151	10.000	0.920	1200	 0	1.000	9.080	100
Ex. S1	564307.684	394242.242	10.000	1.040	1200	1 	1.000	8.960	100

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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
30	0	0	0
100	0	0	0
100	40	0	0

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Results for 1 year Critical Storm Duration. Lowest mass balance: 99.85%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	S1	10	10.000	0.920	19.7	1.0405	0.3990	FLOOD
15 minute summer	Ex. S1	9	9.055	0.095	15.8	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	S1	1.000	Ex. S1	15.8	2.015	2.611	0.0933	8.8

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Results for 30 year Critical Storm Duration. Lowest mass balance: 99.85%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	S1	7	10.000	0.920	48.3	1.0405	9.8256	FLOOD
15 minute summer	Ex. S1	7	9.055	0.095	15.8	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	S1	1.000	Ex. S1	15.8	2.015	2.611	0.0933	12.7

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Results for 100 year Critical Storm Duration. Lowest mass balance: 99.85%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	S1	12	10.000	0.920	48.7	1.0405	15.6256	FLOOD
15 minute summer	Ex. S1	6	9.055	0.095	15.8	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	S1	1.000	Ex. S1	15.8	2.015	2.611	0.0933	22.6

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Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.85%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	S1	11	10.000	0.920	68.2	1.0405	28.0331	FLOOD
15 minute summer	Ex. S1	5	9.055	0.095	15.8	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	S1	1.000	Ex. S1	15.8	2.015	2.611	0.0933	25.4

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Design Settings

Rainfall Methodology	FSR	Maximum Time of Concentration (mins)	30.00
Return Period (years)	1	Maximum Rainfall (mm/hr)	1.0
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)	Easting (m)	Northing (m)	Depth (m)
S1	0.177	5.00	10.000	1200	564296.684	394247.151	0.700
Ex. S1			10.000	1200	564307.684	394242.242	0.820

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	Ex. S1	12.046	0.600	9.300	9.180	0.120	100.0	100	5.26	1.0

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	0.769	6.0	0.5	0.600	0.720	0.177	0.0	19	0.456

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	12.046	100.0	100	Circular	10.000	9.300	0.600	10.000	9.180	0.720

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	S1	1200	Manhole	1 Adoptable	Ex. S1	1200	Manhole	1 Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
S1	564296.684	394247.151	10.000	0.700	1200	0	1.000	9.300	100
Ex. S1	564307.684	394242.242	10.000	0.820	1200	1	1.000	9.180	100

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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
30	0	0	0
100	0	0	0
100	40	0	0

Node S1 Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	9.300	Product Number	CTL-SHE-0072-2000-0700-2000
Design Depth (m)	0.700	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	2.0	Min Node Diameter (mm)	1200

Node S1 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	9.500
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	1065.0	0.0	0.300	1065.0	0.0	0.301	0.0	0.0

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Results for 1 year Critical Storm Duration. Lowest mass balance: 93.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
60 minute winter	S1	58	9.537	0.237	13.0	12.2856	0.0000	SURCHARGED
15 minute summer	Ex. S1	1	9.180	0.000	2.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
60 minute winter	S1	Hydro-Brake®	Ex. S1	2.0	19.0

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Results for 30 year Critical Storm Duration. Lowest mass balance: 93.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
120 minute winter	S1	118	9.629	0.329	19.2	41.7270	0.0000	SURCHARGED
15 minute summer	Ex. S1	1	9.180	0.000	2.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
120 minute winter	S1	Hydro-Brake®	Ex. S1	2.0	41.9

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Results for 100 year Critical Storm Duration. Lowest mass balance: 93.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
180 minute winter	S1	176	9.686	0.386	18.5	59.9529	0.0000	SURCHARGED
15 minute summer	Ex. S1	1	9.180	0.000	2.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
180 minute winter	S1	Hydro-Brake®	Ex. S1	2.0	47.6

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Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 93.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
240 minute winter	S1	236	9.790	0.490	20.8	93.4674	0.0000	FLOOD RISK
15 minute summer	Ex. S1	1	9.180	0.000	2.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
240 minute winter	S1	Hydro-Brake®	Ex. S1	2.0	50.2

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