JACKSON
CONSULTING ENGINEERS

Drainage Strategy

Project:

Leisure Development, Bourne Road, Pinchbeck

Client:

RDC

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Introduction

Jackson consulting engineers have been appointed by RDC on behalf of their client, to undertake a drainage strategy forming part of the supporting evidence for the proposed leisure development, to land off Bourne Road, Spalding, Lincolnshire, PE11 3LJ. 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 north of Bourne Road of the outskirts of Spalding. 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 greenfield with a total site area of approximately 1.4 hectares.



Figure 1 - Site Location

A full topographical survey is not available, but the LIDAR data has been assessed. The data shows that the main road has an approximate elevation of 3.9m AOD with the internal site having an average

level of approximately 2.35m AOD – the level appears to fluctuates around 300mm over the sites area with the range between 2.50m and 2.20m AOD.

There is a roadside ditch to the north of Bourne road, and a presumably riparian ditch to the north of the development boundary. The entirety of the site is greenfield.

A site walkover has been carried out. It was identified that there was a ditch to the northern boundary of the site. This ditch was dry, even after a period of rain, but it did not appear to have an outfall. There was an additional ditch beyond the northern boundary – this appeared to have a significant outlet into the IDB drain located to the west of the site. Both ditches would be accessible to the client, but the levels would need confirming.

Flood Risk

The FRA is being addressed by another consultant and will be provided as part of the overall planning application.

Development Proposals

The development consists of leisure development, consisting of a single large building, with access road, gravel parking and amenity areas.

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", 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 this stage of the project a phase 2 ground investigation report has not been undertaken, and therefore a desktop study is being undertaken to understand the prospect of infiltration.

BGS mapping shows that the bedrock of the area is unrecorded, and no assessment can be made of this.

The superficial deposits, as shown in the figure below, is shown as 'Tidal Flat Deposits – Clay and Silt'. It is very likely that the silt content will block any pores available through the clay and the clay would be generally seen as impermeable.

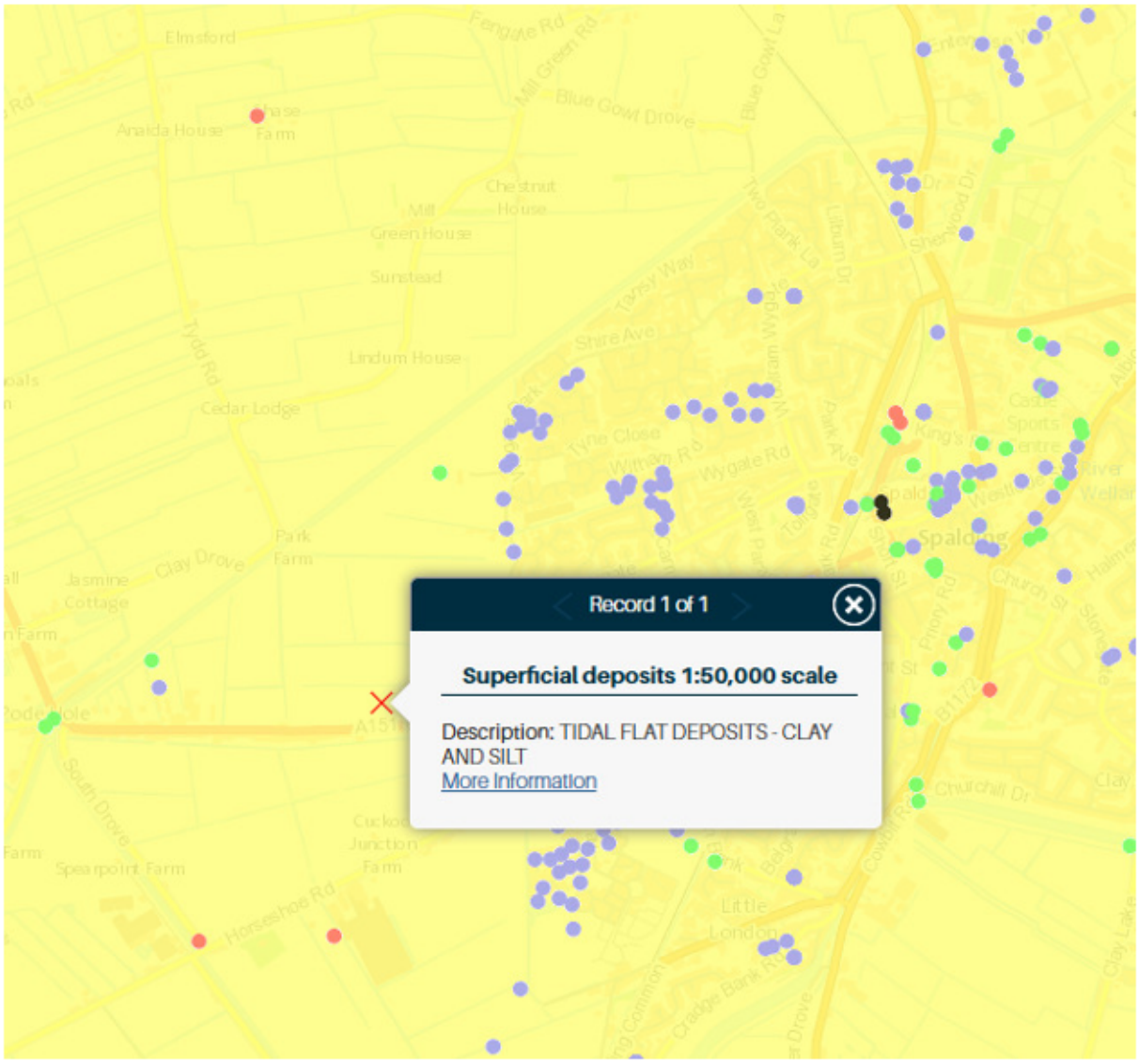


Figure 2 - Superficial Deposits

From the mapping there is limited local borehole information.

Local knowledge also understands that the water table would be relatively high owing to the low elevation of the site and infiltration had been ruled out for the development – although this will be checked for at detailed design.

Although a ground investigation report will be available at a later date, it is safe to assume infiltration will not be viable for the scheme.

Watercourse

As previously identified – there is an open watercourse to the north of the site. This flows to the west, towards an IDB drain – the 'Pennygate Drain'. The open watercourse will be riparian.

Bylaw consent will still be required from the IDB.

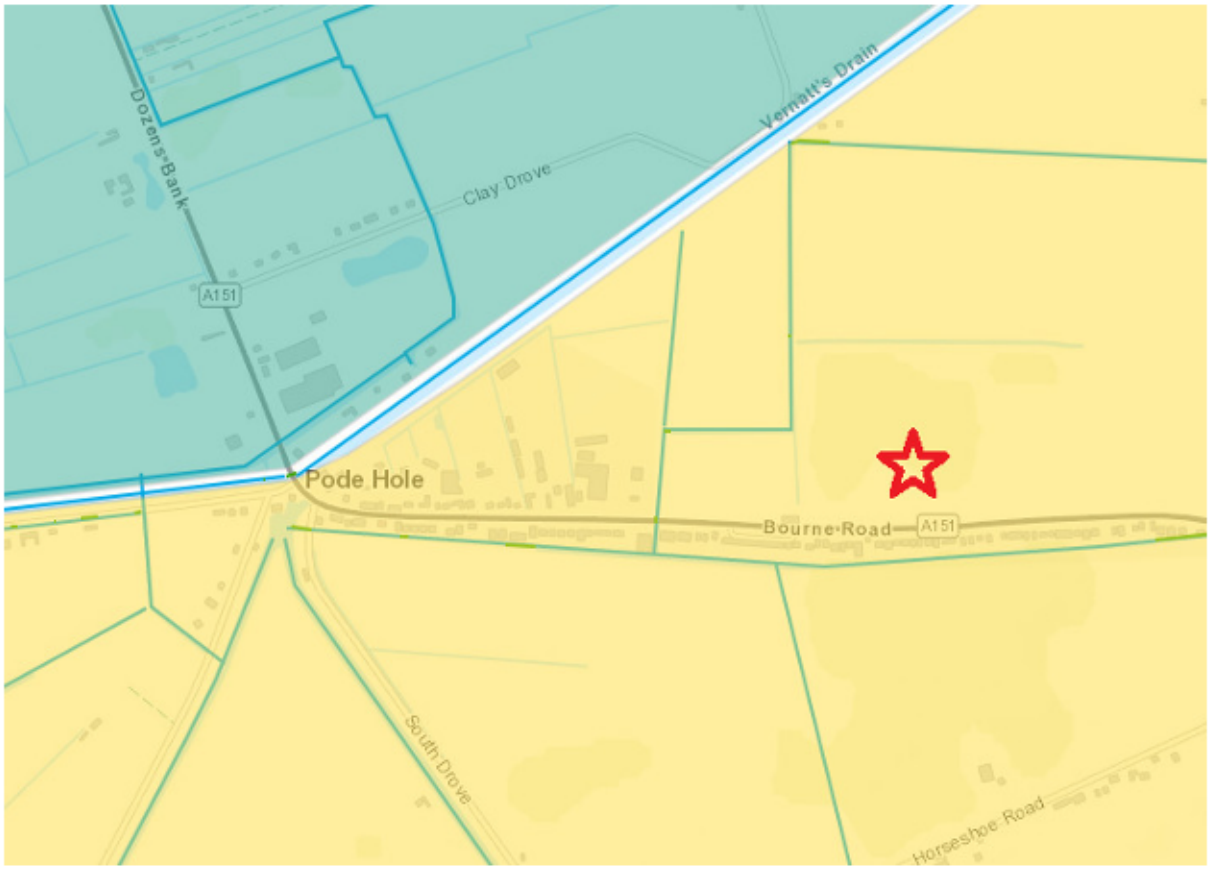


Figure 3 - Local IDB Features

As this is an adequate point of discharge, this is being considered as the discharge for the site.

Surface Water Sewer

There are no identified surface water sewers within close proximity of the site, and as a watercourse has been identified as a sufficient point of discharge then this is not being further explored.

Run-off Rates

Existing greenfield run-off

The existing run-off rates for the site for the 1 in 1 year, Q_{BAR}, 1 in 30 year, 1 in 100 year and 1 in 200 year events are summarised in the figure below. This is in accordance with the methodology in CIRIA C753 and the HR Wallingford Methods.

Results

Method	IH124
Flow rate 1 year (l/s)	0.5 l/s
Flow rate 2 year (l/s)	0.5 l/s
Flow rate 10 years (l/s)	1.0 l/s
Flow rate 30 years (l/s)	1.5 l/s
Flow rate 100 years (l/s)	2.1 l/s
Flow rate 200 years (l/s)	2.5 l/s

Figure 4 - Existing Greenfield Runoff Rates

Proposed Run-off

To comply with the LLFA, the site should be returned to greenfield runoff as far as practical. There are two options to achieve this, to match the runoff to a maximum of Q_{Bar}, or to have a variable outflow, matching to the like for like predevelopment greenfield runoff. To keep the design straight forward at the planning stage, it is proposed that the sites discharge rate will be restricted to Q_{Bar}, although at detailed design, and through consultation with the IDB, the approach may be modified to suit the like-for-like storms. This is noting that the discharge is fairly low and we wish to mitigate the risk of blockage as much as possible.

The current approach means that the discharge is comparable to the pumpable catchment rate for the Q_{BAR} return (this would equate to around 1.4l/s), so would be fully accounted for in the current IDB system, although it is noted the greenfield discharge for the undeveloped site would be higher.

With consent of the IDB and LLFA, during the detailed design process and application of byelaw consent, it would be requested that a minimum practical discharge rate of 5l/s is achieved, or a rate inbetween. The current design would allow for a very limited discharge and would increase a risk of blockage. The IDB drain seems to have good capacity so the risk of flooding will be limited.

Proposed Surface Water Drainage Design

The proposed drainage layout is shown on JCE drawing 0435-JCE-00-SI-SK-C-3000 & 3001, which has been provided as part of the overall planning submission.

All drainage models has been simulated for the 1 in 1, 30, 100 and 100 + 40% climate change events using FEH data, with a Cv value set to 1. No urban creep is included in this instance as it would be expected that any alterations in the future would be subject to planning permission.

A full copy of the preliminary drainage calculations can be found in appendix A of this document, and a graphical representation of the model is shown in the figure below. Please note, that some of the

drainage model has been simplified for this stage of the design. The final model will be fully represented and fine-tuned.

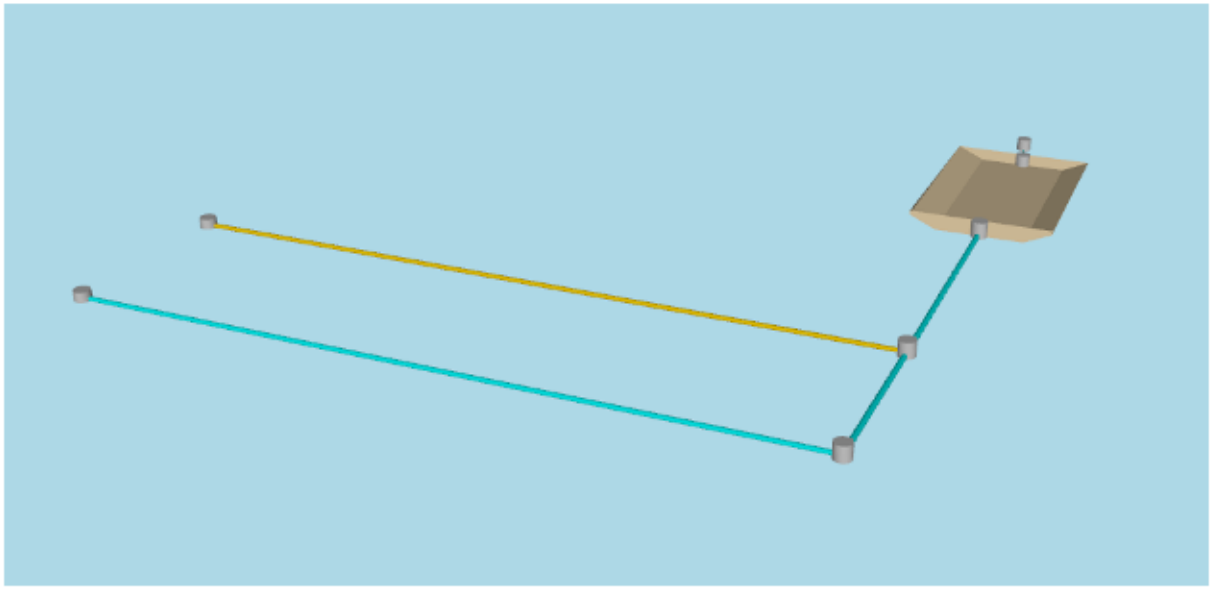


Figure 5 – Drainage Model

As outlined previously, it is assumed that infiltration is not feasible for this site, therefore the SuDS strategy for the surface water drainage will be achieved using the provision of surface water conveyance, cleansing and attenuation. The site layout has generally been designed to utilise a swale to collect highway and domestic water – leading into an attenuation basin, and eventual discharge into the existing ditch.

It is proposed the accessway will be collected through surface drainage such as an ACO channel, before discharging into below ground pipework and into the basin. The access road will be above the existing ground levels to ensure a gravity solution is feasible, and to provide acceptable gradients off the main road.

The car parking will be laid to gravel and be self-draining.

The roof water will discharging via. RWP's and into the below ground pipework, before discharging into the basin.

The basin will then pass through a Hydrobrake to limit the flows to the agreed rates.

Once a full phase 2 ground investigation has been undertaken, more will be known regarding the water table for the site. If the site has a high-water table, or at least if this is within or close to the base of the basin, it will be proposed it will be lined with an impermeable membrane to stop below ground lateral flow of the ground water – in essence to prevent it acting as a land drain.

Given the use of the attenuation basin, it is proposed that sufficient suds features are being used, and as a result the drainage from the development are to discharge into traditional pipework before entering the SUDs features as mentioned above.

The final discharge into the drain – subject to byelaw consent (as the catchment is into the Welland and Deepings IDB), will be via. a new insitu headwall.

The attenuation basin has been designed to store all storms, up to and including the 1 in 100 year + 40% climate change storm events. All storms up to and including the 1 in 100 year + 40% climate change event will be stored within the site. Some limited flooding is present for the 1 in 100 year + 40% event, but this will flow towards the gravel parking and remain on site.. This design ethos is in accordance with DEFRA guidance and LCC's requirements.

Given the elevation of the ditch and natural falls, it is not expected that surcharging will be an issue and therefore has not been included in the calculations.

Surface Water Drainage Summary

In summary, the surface water drainage will be to the following.

- Roofwater will discharge into a piped system before discharging into the attenuation basin.
- The access road will discharge into a piped system before discharging into the attenuation basin.
- The basin will discharge into the riparian drain to the north of the site, which in turn discharges into an IDB drain to the west.

Flood Routing

The scheme has been designed so that no above ground flooding is created in the 1 in 30 year event (other than to the basin) and manageable flooding in the 1 in 100 year + 40% climate change event. The design compliments the natural ground levels, and any flood water will flow towards the abundant land drainage.

Finished Floor Levels + Finished Levels

The FRA has requested that the FFL is set 500mm above the existing ground levels, this is something which is achievable.

The FFL is preliminary and subject to detailed design.

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:

- Other roofs - Low
- Low traffic roads, assumed less than 300 movements a day - 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 6 - 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 7 - 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.

Other Roofs - attenuation basin
0.3 0.5 0.05 = Attenuation Basin 0.5 0.5 0.6

The mitigation provided by the SUDs features exceeds all the pollution indices, therefore sufficient treatment has been provided.

Low traffic roads - attenuation basin
0.5 0.4 0.4 = Attenuation Basin 0.5 0.5 0.6

The mitigation provided by the SUDs features exceeds all the pollution indices, therefore sufficient treatment has been provided.

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

Maintenance

It is crucial that the elements mentioned in the drainage elements and water quality are maintained to a sufficient standard to ensure that the devices can still function as designed. Generally, the maintenance requirements are either from CIRIA 753, or manufacturer guidance.

It is not possible to give full details of the proposed maintenance responsibilities as the project has not been developed enough, but it is expected that a full maintenance/management strategy will be put forward at detailed design and that this can be conditioned.

The maintenance for the devices outlined below are preliminary only and subject to detailed design. The final design will be subject to a full maintenance plan. At present, it will be intended that the drainage for the scheme will remain private.

Attenuation Basin Maintenance

Operation and maintenance requirements for detention basins		
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

Foul Proposals

The preferred method of foul disposal is via. a sewer. In this instance, the nearest foul sewer is located in Broadway, or Primrose Way – this is between 6-700m of the site. There is a further connection located in Monkshouse Court – this is approximately 600m away. The connection in Monkshouse Court would be more favourable as it would be easier to manage in terms of traffic and will be away from the 4-way traffic lights. It is expected that AW will be consulted on the planning application and under this they will be able to confirm if the proposed connection into Monkshouse Court has capacity issues.

To assist in this the proposed maximum users of approximately 500 persons will generate a daily discharge of 20m3. Given the length it is assumed that this will be a pumped connection, with a discharge rate agreed by the water authority. The pump will have backup storage for 24 hours in line with building regulations. Once the foul sewer crosses Monks House Lane, it will likely need to be adopted by the water authority under a section 104. The possible route of the riser main (orange) and S104 sewer (red) into the existing AW manhole (brown) is shown in the figure below. The final layout would need to be agreed with the landowners.



Figure 8 - Possible Sewer Connection

The precise number of users are unknown, but it would be reasonable to assume that 500 visitors per day will be expected on peak days. The British flows and loads charts show that a user value of 40 litres per day would be appropriate for this building type. This gives an approximate distance of 800m, based on a discharge of 20m cube per day and in accordance with the EA's criteria. As a result, the EA will expect a connection to the sewer to be made. As a caveat here, the costings to undertake the sewer connection might be high and might have further limitations such as services located in the

road. It is also envisaged that although 512 persons is the maximum possible users available on the site, it is likely that the final number will be lower than this, therefore it is requested that a connection into the IDB watercourse is not explicitly ruled out, subject to further comments and investigations in regard to the sewer connection.

In summary, the applicant is looking to make a connection to the foul sewer in Monkshouse Court, but wishes to keep the use of a wastewater treatment plant and connection into the watercourse open for discussion in the event that the sewer connection cannot be made. It is understood that the connection into the ditch will require an EA permit as a result of how much treated water will be discharged.

Conclusion

- There is a natural site gradient to the north, where an existing ditch lies.
- 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 do not exceed greenfield runoff rates for the Q_{BAR} event, although negotiations to return this to the like-for-like event or even a peak discharge rate of 5l/s are to be undertaken to offer some resilience to the flow control size.
- Discharge from the site will be restricted to 0.5/s, subject to IDB byelaw consent. As above this is looking to be increased.
- No above ground flooding is present for the 1 in 30 occurs and limited flooding for the 1 in 100 year + 40% climate change event climate change event occurs, but this is held within the site, and would not pose a risk to life.
- 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.
- The risk of flooding as a result of the development is not increased 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: 0435-JCE-00-SI-DR-C-3000.
- Finished floor levels will be set at a minimum of 500mm above general existing ground levels. This has been taken as a minimum of 2.80m AOD – to be confirmed through detailed design.
- Foul water has two viable solutions; into the AW system – subject to capacity , or through a waste water treatment plant into the watercourse network – this is subject to EA and IDB approval.
- At present a foul pump is being proposed with 24 hour storage.

Appendices

Appendix A – 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	1.000	Include Intermediate Ground	✓
Time of Entry (mins)	5.00	Enforce best practice design rules	✓

Nodes

Name	T of E (mins)	Cover Level (m)	Diameter (mm)	Easting (m)	Northing (m)	Depth (m)	Invert Level (m)
S1	5.00	2.650	600	522378.001	323929.906	0.950	1.700
S2	5.00	2.650	600	522378.001	323960.556	0.970	1.680
S3		2.650	1350	522492.916	323929.923	1.331	1.319
S4		2.650	1350	522493.126	323960.548	1.399	1.251
HW1		2.650	1350	522493.126	324007.410	1.503	1.147
Outlet		2.500	1350	522491.730	324053.204	1.559	0.941
Hydrobrake	5.00	2.500	1200	522493.126	324044.452	1.500	1.000

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
1.000	S1	S3	114.915	0.600	1.700	1.394	0.306	375.0	375	7.06	1.0
1.001	S3	S4	30.626	0.600	1.319	1.251	0.068	450.0	450	7.60	1.0
1.002	S4	HW1	46.862	0.600	1.251	1.147	0.104	450.0	450	8.42	1.0
2.000	S2	S4	115.125	0.600	1.680	1.373	0.307	375.0	375	7.06	1.0
3.000	Hydrobrake	Outlet	8.863	0.600	1.000	0.941	0.059	150.0	150	5.18	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	0.929	102.7	0.8	0.575	0.881	0.208	0.0	23	0.275
1.001	0.952	151.4	0.8	0.881	0.949	0.208	0.0	22	0.246
1.002	0.952	151.4	1.3	0.949	1.053	0.373	0.0	29	0.296
2.000	0.929	102.7	0.6	0.595	0.902	0.165	0.0	20	0.251
3.000	0.818	14.5	0.0	1.350	1.409	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	114.915	375.0	375	Circular	2.650	1.700	0.575	2.650	1.394	0.881
1.001	30.626	450.0	450	Circular	2.650	1.319	0.881	2.650	1.251	0.949
1.002	46.862	450.0	450	Circular	2.650	1.251	0.949	2.650	1.147	1.053
2.000	115.125	375.0	375	Circular	2.650	1.680	0.595	2.650	1.373	0.902
3.000	8.863	150.0	150	Circular	2.500	1.000	1.350	2.500	0.941	1.409

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	S1	600	Manhole	1 Adoptable	S3	1350	Manhole	1 Adoptable
1.001	S3	1350	Manhole	1 Adoptable	S4	1350	Manhole	1 Adoptable
1.002	S4	1350	Manhole	1 Adoptable	HW1	1350	Manhole	1 Adoptable
2.000	S2	600	Manhole	1 Adoptable	S4	1350	Manhole	1 Adoptable
3.000	Hydrobrake	1200	Manhole	1 Adoptable	Outlet	1350	Manhole	1 Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
S1	522378.001	323929.906	2.650	0.950	600	 0	1.000	1.700	375
S2	522378.001	323960.556	2.650	0.970	600	 0	2.000	1.680	375
S3	522492.916	323929.923	2.650	1.331	1350	 1	1.000	1.394	375
						0	1.001	1.319	450
S4	522493.126	323960.548	2.650	1.399	1350	 1	1.001	1.251	450
						2	2.000	1.373	375
						0	1.002	1.251	450
HW1	522493.126	324007.410	2.650	1.503	1350	 1	1.002	1.147	450
Outlet	522491.730	324053.204	2.500	1.559	1350	 1	3.000	0.941	150
Hydrobrake	522493.126	324044.452	2.500	1.500	1200	 0	3.000	1.000	150

Simulation Settings

Rainfall Methodology	FEH-22	Analysis Speed	Detailed	Starting Level (m)	
Rainfall Events	Singular	Skip Steady State	x	Check Discharge Rate(s)	x
Summer CV	1.000	Drain Down Time (mins)	240	Check Discharge Volume	x
Winter CV	1.000	Additional Storage (m³/ha)	0.0		

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Storm Durations

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

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
1	0	0	0
2	0	0	0
10	0	0	0
30	0	0	0
100	0	0	0
100	40	0	0

Node Hydrobrake Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	x	Sump Available	✓
Invert Level (m)	1.000	Product Number	CTL-SHE-0034-5000-0800-5000
Design Depth (m)	0.800	Min Outlet Diameter (m)	0.075
Design Flow (l/s)	0.5	Min Node Diameter (mm)	1200

Node Hydrobrake Flow through Pond Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Main Channel Length (m)	32.000
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	1.000	Main Channel Slope (1:X)	200.0
Safety Factor	2.0	Time to half empty (mins)		Main Channel n	0.035

Inlets

HW1

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	529.0	529.0	1.000	851.0	864.0

Other (defaults)

Entry Loss (manhole)	0.250	Entry Loss (junction)	0.000	Apply Recommended Losses	x
Exit Loss (manhole)	0.250	Exit Loss (junction)	0.000	Flood Risk (m)	0.300

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Results for 1 year Critical Storm Duration. Lowest mass balance: 98.90%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	1	1.700	0.000	0.0	0.0000	0.0000	OK
15 minute summer	1.000:50%	11	1.661	0.114	22.1	0.0000	0.0000	OK
15 minute summer	S2	1	1.680	0.000	0.0	0.0000	0.0000	OK
15 minute summer	2.000:50%	11	1.625	0.099	17.5	0.2168	0.0000	OK
15 minute summer	S3	12	1.433	0.114	20.2	0.1628	0.0000	OK
15 minute summer	S4	12	1.410	0.159	35.5	0.2274	0.0000	OK
1440 minute winter	HW1	1380	1.207	0.060	2.7	0.0859	0.0000	OK
180 minute winter	Outlet	220	0.958	0.017	0.4	0.0000	0.0000	OK
1440 minute winter	Hydrobrake	1380	1.207	0.207	2.7	0.2342	0.0000	SURCHARGED

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 summer	S1	1.000	1.000:50%	0.0	0.000	0.000	0.8144	
15 minute summer	S1	1.000	S3	20.2	0.779	0.196	1.5020	
15 minute summer	S2	2.000	2.000:50%	0.0	0.000	0.000	0.6653	
15 minute summer	S2	2.000	S4	15.4	0.723	0.150	1.2274	
15 minute summer	S3	1.001	S4	20.1	0.501	0.133	1.2468	
15 minute summer	S4	1.002	HW1	35.2	1.468	0.233	1.2737	
1440 minute winter	HW1	Flow through Pond	Hydrobrake	2.7	0.007	0.000	70.9589	
1440 minute winter	Hydrobrake	3.000	Outlet	0.4	0.353	0.027	0.0098	28.7

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Results for 2 year Critical Storm Duration. Lowest mass balance: 98.91%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	1	1.700	0.000	0.0	0.0000	0.0000	OK
15 minute summer	1.000:50%	11	1.700	0.153	37.5	0.0000	0.0000	OK
15 minute summer	S2	1	1.680	0.000	0.0	0.0000	0.0000	OK
15 minute summer	2.000:50%	11	1.658	0.132	29.7	0.3860	0.0000	OK
15 minute summer	S3	12	1.484	0.165	35.2	0.2365	0.0000	OK
15 minute summer	S4	12	1.462	0.211	61.4	0.3019	0.0000	OK
1440 minute winter	HW1	1410	1.257	0.110	3.6	0.1569	0.0000	OK
8640 minute summer	Outlet	4260	0.958	0.017	0.4	0.0000	0.0000	OK
1440 minute winter	Hydrobrake	1410	1.257	0.257	3.3	0.2902	0.0000	SURCHARGED

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 summer	S1	1.000	1.000:50%	0.0	0.000	0.000	1.2121	
15 minute summer	S1	1.000	S3	35.2	0.908	0.342	2.2349	
15 minute summer	S2	2.000	2.000:50%	0.0	0.000	0.000	0.9932	
15 minute summer	S2	2.000	S4	26.7	0.844	0.260	1.8330	
15 minute summer	S3	1.001	S4	34.8	0.554	0.230	1.9248	
15 minute summer	S4	1.002	HW1	61.8	1.729	0.409	1.8614	
1440 minute winter	HW1	Flow through Pond	Hydrobrake	3.3	0.009	0.000	99.6138	
1440 minute winter	Hydrobrake	3.000	Outlet	0.4	0.353	0.027	0.0098	29.9

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Results for 10 year Critical Storm Duration. Lowest mass balance: 99.35%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	11	1.802	0.102	7.5	0.0289	0.0000	OK
15 minute summer	1.000:50%	11	1.785	0.238	83.0	0.0000	0.0000	OK
15 minute summer	S2	12	1.745	0.065	4.3	0.0183	0.0000	OK
15 minute summer	2.000:50%	11	1.730	0.204	65.9	0.9243	0.0000	OK
15 minute summer	S3	12	1.607	0.288	75.0	0.4127	0.0000	OK
15 minute summer	S4	12	1.578	0.327	132.5	0.4684	0.0000	OK
1440 minute winter	HW1	1440	1.389	0.242	6.1	0.3467	0.0000	OK
720 minute winter	Outlet	300	0.958	0.017	0.4	0.0000	0.0000	OK
1440 minute winter	Hydrobrake	1440	1.389	0.389	4.0	0.4403	0.0000	SURCHARGED

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 summer	S1	1.000	1.000:50%	-7.5	-0.192	-0.073	2.8210	
15 minute summer	S1	1.000	S3	75.0	1.111	0.731	3.9586	
15 minute summer	S2	2.000	2.000:50%	-4.3	-0.156	-0.042	2.0792	
15 minute summer	S2	2.000	S4	58.6	0.997	0.571	3.4935	
15 minute summer	S3	1.001	S4	74.6	0.646	0.493	3.5350	
15 minute summer	S4	1.002	HW1	128.8	2.081	0.851	3.1072	
1440 minute winter	HW1	Flow through Pond	Hydrobrake	4.0	0.010	0.000	180.2280	
1440 minute winter	Hydrobrake	3.000	Outlet	0.4	0.353	0.027	0.0098	30.7

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Results for 30 year Critical Storm Duration. Lowest mass balance: 99.47%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	11	1.862	0.162	12.2	0.0459	0.0000	OK
15 minute summer	1.000:50%	11	1.854	0.307	112.5	0.0000	0.0000	OK
15 minute summer	S2	11	1.793	0.113	9.0	0.0321	0.0000	OK
15 minute summer	2.000:50%	12	1.774	0.248	89.2	1.3673	0.0000	OK
15 minute summer	S3	12	1.694	0.375	102.8	0.5366	0.0000	OK
15 minute summer	S4	12	1.661	0.410	178.9	0.5869	0.0000	OK
2160 minute winter	HW1	2160	1.479	0.332	5.6	0.4749	0.0000	OK
2160 minute winter	Outlet	2160	0.958	0.017	0.4	0.0000	0.0000	OK
2160 minute winter	Hydrobrake	2160	1.479	0.479	2.9	0.5416	0.0000	SURCHARGED

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 summer	S1	1.000	1.000:50%	-12.2	-0.268	-0.118	4.0833	
15 minute summer	S1	1.000	S3	102.8	1.159	1.001	5.4040	
15 minute summer	S2	2.000	2.000:50%	-9.0	-0.205	-0.087	3.0193	
15 minute summer	S2	2.000	S4	79.6	1.014	0.775	4.8406	
15 minute summer	S3	1.001	S4	99.3	0.715	0.656	4.4829	
15 minute summer	S4	1.002	HW1	173.9	2.178	1.149	3.8044	
2160 minute winter	HW1	Flow through Pond	Hydrobrake	2.9	0.007	0.000	237.8995	
2160 minute winter	Hydrobrake	3.000	Outlet	0.4	0.355	0.027	0.0099	46.8

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Results for 100 year Critical Storm Duration. Lowest mass balance: 99.54%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	12	2.058	0.358	21.1	0.1012	0.0000	OK
15 minute summer	1.000:50%	12	2.074	0.527	146.7	0.0000	0.0000	SURCHARGED
15 minute summer	S2	13	1.898	0.218	12.2	0.0616	0.0000	OK
15 minute summer	2.000:50%	12	1.874	0.347	116.4	2.6796	0.0000	OK
15 minute summer	S3	12	1.863	0.544	124.6	0.7786	0.0000	SURCHARGED
15 minute summer	S4	12	1.800	0.549	207.8	0.7860	0.0000	SURCHARGED
2880 minute winter	HW1	2820	1.608	0.461	5.7	0.6597	0.0000	OK
2880 minute winter	Outlet	2820	0.959	0.018	0.4	0.0000	0.0000	OK
2880 minute winter	Hydrobrake	2820	1.608	0.608	3.1	0.6876	0.0000	SURCHARGED

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 summer	S1	1.000	1.000:50%	-21.1	-0.288	-0.205	6.2838	
15 minute summer	S1	1.000	S3	124.6	1.175	1.213	6.3374	
15 minute summer	S2	2.000	2.000:50%	17.8	0.304	0.174	4.7855	
15 minute summer	S2	2.000	S4	92.2	1.048	0.898	6.2415	
15 minute summer	S3	1.001	S4	123.9	0.782	0.818	4.8525	
15 minute summer	S4	1.002	HW1	205.3	2.276	1.357	3.9985	
2880 minute winter	HW1	Flow through Pond	Hydrobrake	3.1	0.008	0.000	325.5184	
2880 minute winter	Hydrobrake	3.000	Outlet	0.4	0.366	0.030	0.0107	67.0

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Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.67%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute summer	S1	10	2.650	0.950	37.3	0.2689	2.0574	FLOOD
15 minute summer	1.000:50%	11	2.655	1.108	205.3	1.0839	0.0000	PONDING
15 minute summer	S2	12	2.405	0.725	22.0	0.2052	0.0000	FLOOD RISK
15 minute summer	2.000:50%	13	2.275	0.749	163.0	12.4760	0.0000	SURCHARGED
15 minute summer	S3	12	2.221	0.902	187.7	1.2910	0.0000	SURCHARGED
15 minute summer	S4	12	2.112	0.861	265.4	1.2323	0.0000	SURCHARGED
2880 minute winter	HW1	2880	1.792	0.645	7.7	0.9227	0.0000	OK
2880 minute winter	Outlet	2880	0.960	0.019	0.5	0.0000	0.0000	OK
2880 minute winter	Hydrobrake	2880	1.792	0.792	4.1	0.8955	0.0000	SURCHARGED

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 summer	S1	1.000	1.000:50%	-37.3	-0.345	-0.364	6.3374	
15 minute summer	S1	1.000	S3	187.7	1.702	1.828	6.3374	
15 minute summer	S2	2.000	2.000:50%	23.1	0.351	0.225	6.3490	
15 minute summer	S2	2.000	S4	144.4	1.309	1.407	6.3490	
15 minute summer	S3	1.001	S4	184.7	1.166	1.220	4.8525	
15 minute summer	S4	1.002	HW1	262.3	2.524	1.733	4.6184	
2880 minute winter	HW1	Flow through Pond	Hydrobrake	4.1	0.007	0.000	459.4594	
2880 minute winter	Hydrobrake	3.000	Outlet	0.5	0.379	0.034	0.0116	73.6

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