VikingCCS

Viking CCS Pipeline

Environmental Statement Volume IV – Appendix 11-3: Drainage Strategy

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1 Introduction

1.1 Project Introduction

1.1.1 The Viking CCS Project intends to transport compressed and conditioned CO_2 from the Immingham Facility to store in depleted gas reservoirs in the Southern North Sea. The Oil and Gas Authority (OGA) awarded the Applicant a CO_2 appraisal and storage licence in 2021. The Viking CCS Project aims to transport and store up to 10 million tonnes of CO_2 annually by 2030, rising to 15 million tonnes by 2035.

1.1.2 Further information on the wider Viking CCS Project and the wider Humber region is contained within the report entitled *"Viking CCS – Transforming the Humber into a net zero Super Place"* (Ref 1).

1.1.3 This surface water drainage strategy report has been prepared on behalf of Chrysaor Production (U.K.) Limited, a Harbour Energy Company for the Viking CCS Pipeline. The report specifically covers six sites situated along the length of the proposed Viking CCS Pipeline (hereafter the Proposed Development). The sites considered include:

- Immingham Facility – X 516986 Y 416781;
- Washingdales Lane Block Valve Station – X 519460 Y 407048;
- Thoroughfare Block Valve Station – X 526248 Y 400236;
- Louth Road Block Valve Station – X 535809 Y 390543;
- Theddlethorpe Facility Option 1 – X 548623 Y 387508; and
- Theddlethorpe Facility Option 2 – X 548175 Y 387586.

1.1.4 This report is appended to the Viking CCS Environmental Statement (ES) and forms *ES Volume IV Appendix 11-3 Drainage Strategy (Application Document 6.4.11.3).*

1.1.5 The drainage design within this report has been developed against available site information and design details at the time of writing to provide a surface water drainage strategy. As part of further Front-End Engineering Design, this drainage design will be further developed and concluded in tandem with the overall scheme design development. Additional work will include a review of findings from investigations recommended in this report.

1.2 Legislation, Policy and Guidance

Introduction

1.2.1 The Legislation, Policy and Guidance section of this chapter provides an overview of the relevant legislation, planning policy and technical guidance relevant to the Surface Water Drainage Strategy.

Local and National Planning Policy

1.2.2 **National Planning Policy Framework** – National Planning Policy Framework (NPPF) (Ref 3) requires that new developments should not increase flood risk both on the Site and in the area surrounding it, meaning that surface water runoff should not exceed the peak volumes already generated on the Site and that betterment should be provided, where possible.

1.2.3 **North Lincolnshire Council SuDS and Flood Risk Guidance Document** (Ref 2) - The works that fall within the North Lincolnshire Council (NLS) district includes the Immingham Facility. The NLC are the Lead Local Flood Authority (LLFA).

1.2.4 NLC has a SuDS and Flood Risk Guidance Document to provide developers and designers with guidance on Sustainable urban Drainage Systems (SuDS) expected to be submitted with planning applications to NLC. The document also provides a checklist for Lead Local Flood Authority (LLFA) requirements to accompany a planning application.

1.2.5 The guidance provides criteria to be met by developers. Notable criteria that relate to the proposed Viking CCS developments are listed below:

- SuDS are required for all developments;
- No water should be stored above ground up to and including the 1 in 100 year event unless stored in a SuDS component;
- Surface water runoff should be limited for all new developments to greenfield runoff rate;
- Infiltration should only be viable for areas where the infiltration rate of the soils are above 1×10^{-6} m/s. Infiltration testing should be undertaken over a period of time, preferably over various seasons to obtain a range of infiltration rates; and
- The level of betterment will be considered on a site-by-site basis for all brownfield sites.

1.2.6 **North East Lincolnshire Council** – The works that fall within the North East Lincolnshire Council (NELC) jurisdiction includes the Washingdales Lane Block Valve Station and Thoroughfare Block Valve Station. The NELC are the LLFA. NELC were contacted and confirmed they do not have specific guidance with regards to SUDs and drainage design development.

1.2.7 **East Lindsey District Council** – The works that fall within the East Lindsey District Council (ELDC) jurisdiction includes Louth Road Block Valve Station and the Theddlethorpe Facility Option 1 and Option 2. Lincolnshire County Council (LCC) are the LLFA for East Lindsey. LCC have produced a Sustainable Drainage Design and Evaluation Guide (2018) (Ref 4) which includes criteria to be covered at the concept design stage. This includes the following points which are addressed in this report:

- Data gathering (geology topography, flood risk, utilities, landscape, community and wildlife);
- Existing site and modified site flow route analysis;
- SuDs Design Elements; and
- Quantity, Quality, Amenity and biodiversity.

1.2.8 **Non-Statutory Technical Standards for Sustainable Drainage Systems (2015) –** The Non-Statutory Technical Standards for Sustainable Drainage Systems produced by Department for Environment, Food & Rural Affairs (DEFRA) (Ref 5) represent the current guidance for the design, maintenance and operation of SuDS.

1.2.9 The standards set out that peak runoff rates from development sites should be as close as is reasonably practicable to the greenfield rate but should never exceed the pre-development runoff rate. The standards also set out that drainage systems should be designed so that flooding does not occur on any part of a site for a 1 in 30-year rainfall event, and that no flooding of a building (including basement) would occur during a 1 in 100-year rainfall event. It is also noted within the standards that pumping should only be used when it is not reasonably practicable to discharge by gravity.

1.2.10 The following guidance has been adopted for each of the sites considered within this drainage strategy:

- **Immingham Facility** – NLC SuDS and Flood Risk Guidance Document (Ref 2);
- **Washingdales Lane Block Valve Station** – Non-Statutory Technical Standards for Sustainable Drainage Systems (Ref 5);
- **Thoroughfare Block Valve Station** – Non-Statutory Technical Standards for Sustainable Drainage Systems (Ref 5);
- **Louth Road Block Valve Station** – LCC Sustainable Drainage Design and Evaluation Guide (Ref 4);
- **Theddlethorpe Facility Option 1** – LCC Sustainable Drainage Design and Evaluation Guide (Ref 4); and
- **Theddlethorpe Facility Option 2** – LCC Sustainable Drainage Design and Evaluation Guide (Ref 4).

Stakeholder Consultation

1.2.11 A summary of stakeholder engagement specific to the surface water drainage strategy has been provided in **Table 1**.

Table 1: Surface Water Drainage Strategy Stakeholder Consultation

Stakeholder	Date of communication	Summary of discussions
North East Lindsey IDB	5 June 2023	A request for information regarding the Immingham Facility
Lindsey Marsh IDB	5 June 2023	A request for information regarding the Theddlethorpe Facility

1.2.12 Communication with the Head of Technical & Engineering Services from the North East Lindsey Drainage Board (IDB) is included in **Annex D** and summarised below:

- The existing drainage channel between the railway line and Rosper Road is known as South Killingholme Drain Branch 1. The drainage channel running adjacent to Rosper Road is known as South Killingholme Drain. Both channels are maintained by North-East Lindsey IDB;
- South Killingholme Drain Branch 1 drainage is gravity and can be tide locked, Rosper Road Pits acts as attenuation in the system, but water levels may affect discharge. The site is at risk of flood, primarily from over topping or breach of the Humber flood banks;

- Any works within 9m of the top of the bank requires consent from the Board under the Byelaws;
- Consent to discharge into IDB maintained drainage is required; and
- Greenfield discharge rates are acceptable.

1.2.13 A meeting was undertaken with the IDB 19 June 2023. The communication with the Lindsey Marsh IDB is included in **Annex D** and summarised below:

- The Cut and other drainage channels in the Theddlethorpe area are IDB maintained as shown on plan in **Annex D**;
- There are no known concerning flood issues in the area. However, this query will be relayed to the local agent/surveyor for confirmation;
- The IDB confirmed a 9m easement is standard for maintenance access requirements;
- Consent is required for discharge into IDB controlled assets; and
- Greenfield discharge rates are preferred, however other rates and outlet sizes are considered on a mitigated, rational and evidential basis. 1.4l/s/ha is a rate considered by the IDB.

1.2.14 **Internal Drainage Board** – The six sites considered within this surface water drainage strategy are situated within IDB districts as listed below:

- **Immingham Facility** – North-East Lindsey IDB;
- **Washingdales Lane Block Valve Station** – Not situated within an IDB catchment;
- **Thoroughfare Block Valve Station** – Not situated within an IDB catchment;
- **Louth Road Block Valve Station** – Not situated within an IDB catchment;
- **Theddlethorpe Facility Option 1** – Lindsey Marsh IDB; and
- **Theddlethorpe Facility Option 2** – Lindsey Marsh IDB.

Existing Studies and Guidance

1.2.15 Land Drainage Consultancy Ltd undertook a desk study (Ref 6) considering soils and land drainage impacted by the proposed pipeline. Recommendations from the report with regards to drainage include:

- Landowners and/or tenants contacted and details of existing land drainage systems obtained;
- Site surveys completed to observe and record key drainage features and to undertake a detailed drainage topographical survey;
- Conceptual pre-construction drainage designs produced to ensure offsite land drainage systems continue to function during the construction phase of the Proposed Development; Conceptual designs also required to highlight key crossing drains where coincidence with the proposed pipeline is possible; and
- Conceptual post-construction drainage schemes are designed to replace drains damaged within the Proposed Development construction areas and to alleviate soil structural degradation.

1.2.16 The report undertook an assessment to understand if the pipeline route passes through arable land drained via land drainage. A review of the "Known Drainage" drawing suggests the following for the sites considered within this report:

- **Immingham Facility** – Non-Agricultural;

- **Washingdales Lane Block Valve Station** – Unknown Drainage;
- **Thoroughfare Block Valve Station** – Suspected Drainage;
- **Louth Road Block Valve Station** – Suspected Drainage;
- **Theddlethorpe Facility Option 1** – Non-Agricultural; and
- **Theddlethorpe Facility Option 2** – Unknown Drainage.

1.2.17 **CIRIA, SuDs Manual (C753), 2015** – Guidance has been taken from The SuDs Manual (Ref 7) for the development of SuDs infrastructure recommended as part this strategy. As per C753, the established discharge hierarchy for surface water is:

- infiltration to the ground;
- discharge to surface waters;
- discharge to a surface water, highway drain or another drainage system; and
- discharge to a combined sewer.

1.2.18 **Environment Agency, Rainfall Runoff Management for Developments (SC030219), 2013 (Ref 8)** – The report provides guidance on the management of stormwater drainage for developments for regulators, developers and local authorities.

2 Immingham Facility

2.1 Desktop Study – Immingham Facility

Introduction

2.1.1 The first component of the Proposed Development will consist of the Immingham Facility (X 516986 Y 416781) to be located in a currently unused section of brownfield land to the south of the VPI Immingham site. This facility would require an area of approximately 10,900m^2. The existing land is shown in **Figure 1** and comprises a grassed field to the west of Rosper Road, which was formerly used as a construction laydown for the Immingham power station.

2.1.2 An indicative layout of the Immingham Facility is shown on **Figure 7** in **Annex C**. In summary, the Immingham Facility would consist of the following key components:

- Inlet manifold with valve access platform;
- Permanent pig launcher and receiver to allow the onshore CO_2 pipeline to be cleaned and inspected during commissioning and operation and be suitable for intelligent pigging;
- Common pig handling area for the pig receiver and launcher, which includes a projectile blast wall;
- High-integrity pressure protection system (HIPPS);
- Emergency Shutdown Valve (ESDV) for each pipeline and Isolation valves;
- Venting system including vent pipework, valves and vent stack. Permanent vent stack to be a maximum of 24” diameter and 25 metres high;
- Various instruments installed on the pipework, including temperature and pressure measurement and ultrasonic flowmeter;
- Central control room (CCR);
- Local equipment room (LER);
- Analyser house; and
- Supporting utilities.

2.1.3 See **Annex A** for **Figure 1 – Site Overview and Topography**.

Site Topography

2.1.4 A detailed topographical survey has not yet been undertaken for the Immingham Facility. A review of available LiDAR information has been undertaken and indicates the site is relatively flat with land falling north/north east. Ground elevations range from approximately 2.8m Above Ordnance Datum (AOD) at the northern edge to around 4.5m AOD in the south. The site topography is shown on **Figure 1**, **Annex A**.

Local Hydrology

2.1.5 Two main river watercourse that are situated closest to the site location are Skitter Beck (approx.. 4.8km west) and North Beck Drain (6pprox.. 4.6km south).

2.1.6 A drainage channel (South Killingholme Drain Branch 1) to the north of the proposed site is assumed to collect and convey surface water to the east and connect into the South Killingholme Main Drain via a culvert beneath Rosper Road, ultimately discharging into the North Sea.

2.1.7 The Environment Agency flood maps (Ref 8) indicates the site has a very low risk of flooding (1 in 1000 year/0.1% AEP each year) from rivers or the sea. The site also has a very low risk of flooding from surface water.

Ground Conditions, Ground water and Infiltration

2.1.8 A review of the BGS Geology Viewer (Ref 10) indicates the bedrock geology is Burnham Chalk Formation with possibly two types of superficial deposits including Tidal Flat Deposits and Till, Devensian – Diamicton.

2.1.9 The site's underlaying strata is classified as Principal Bedrock Aquifer and a secondary (undifferentiated) Superficial Drift Aquifer. The site sits in a medium ground water vulnerability area.

2.1.10 The BGS borehole records (Ref 12) from holes drilled near the site indicate made ground or warp above various layers of differing strata including clays sands silts and gravels. A chalk bedrock is noted 18-20m below ground level.

2.1.11 A review of the Soil-Scapes layer on Magic Maps (Ref 11) indicates the site is situated on the border between two types of shallow strata. North is loamy and clayey soils of coastal flats with naturally high groundwater and a naturally wet drainage type. South is slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils with impeded drainage. The site is located in Zone III Source Protection Zone (regarding the safeguarding of drinking water quality).

2.1.12 Following the description of surface geology above it is not recommended to discharge surface water via infiltrating methods.

Existing Utilities

2.1.13 A desktop study was undertaken by GroundSure to gather available utility information from providers. This was submitted to AECOM as an AutoCAD DWG file covering the pipeline alignment and a buffer area either side. The DWG information indicates there is no known utilities within the site boundary.

2.2 Surface Water Drainage Strategy – Immingham Facility

Contributing Areas and Runoff Calculation

2.2.1 The contributing area has been measured from a scheme layout drawing produced by Kent Energies Ltd (drawing number: EN070008/APP/4.6). The proposed impermeable and permeable areas are summarised in **Table 2**.

2.2.2 The site will be predominantly permeable with unpaved areas to be graded to natural ground levels overlain with weed control membrane and 75 mm of 20mm single size gravel.

2.2.3 Impermeable areas will consist of a 6m wide access road spurring from Rosper Road and from an onsite Primary Access Road to the north. Within the fence line boundary, a splayed road is proposed to allow access to the Pig Launch area. Both the pig launch area and high-integrity pressure protection system area will sit upon concrete pads. Three kiosks with flat roofs will also be situated within the site boundary sat upon concrete bases.

2.2.4 The sites will be cleared, excavated and graded to achieve the approximate required finished levels. Surfaces will be constructed to falls so that rainwater can drain to the appropriate drainage system where required. Roads and hardstanding will have flush concrete kerbs to allow surface water run-off. The majority of the site will be permeable surface to minimise runoff. A cut-off drainage channel maybe required at the site entrance gate to control runoff offsite.

Table 2: Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment
Total Site Area	N/A	6530	Area within fence line
Stone area	Permeable	5835	The majority of ground surface within the fence line is to be stone aggregate
Roads Inside Immingham Facility Fence line	Impermeable	143	Access turning and parking is proposed to access the pig handling area and site
Roads Outside Immingham Facility Fence line	Impermeable	3636	Roads to enable access to the Immingham Facility
Roofs	Impermeable	363	3 buildings are proposed including Central Control Room, Local Equipment Room and Analyser House
Concrete Pad	Impermeable	189	High-integrity pressure protection system and pig handling area are assumed to be sited on concrete pads or similar impermeable ground
Totals		**m^2**	**ha**
Total Impermeable Area		4331	0.433
Total Permeable Area		5835	0.583
Total Contributing Area		4331	0.433

Greenfield Runoff

2.2.5 The greenfield runoff rates for the proposed Immingham Facility have been calculated based on the IH124 method using the HR Wallingford UK SuDS website (Ref 13). The greenfield runoff rates for a 50ha area were calculated using this method. A summary of the results can be seen in calculation report found in **Annex B**, with the peak greenfield runoff rates for the total contributing area interpolated from the results shown in **Table 3**.

Table 3: Peak Greenfield Runoff Rate

Rainfall Event Frequency	Runoff (l/s/ha)	Site Contributing Area (0.410ha discounting the access bridge area) GF Runoff l/s
1 in 1 Year (Approx. 99% AEP)	2.16	0.89
Qbar	2.49	1.02
1 in 30 Year (3.33% AEP)	6.08	2.49
1 in 100 Year (1% AEP)	8.86	3.64

Proposed Surface Water Runoff Rates

2.2.6 **Table 4** below shows the unrestricted surface water runoff rate post-development based on the Modified Rational Method. This method estimates runoff based on the nature of the ground surface (hardstanding, vegetation etc.) and rainfall depth, duration and frequency information for the immediate area, as follows:

- C (Coefficient of impermeability) = 1.0;
- A (area) = ha; 0.433; and
- I (Rainfall intensity based on FEH data (Ref 15)).

Table 4: Proposed Peak Runoff Rate

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	33.22	21.40	13.23	9.39	7.46	5.42	2.93	1.72	1.01
5 (20%)	55.47	35.70	22.02	14.31	10.95	7.67	3.98	2.31	1.34
10 (10%)	71.26	45.74	28.41	17.81	13.38	9.21	4.70	2.71	1.58
30 (3.3%)	95.91	62.31	38.66	23.30	17.22	11.67	5.86	3.37	1.97
50 (2%)	106.84	69.99	43.55	25.93	19.06	12.85	6.43	3.72	2.17
100 (1%)	122.69	80.60	50.41	59.25	21.66	14.53	7.28	4.24	2.17
100 +20% CC	147.22	96.72	60.50	35.55	25.99	17.44	8.73	5.09	2.61
100 +40% CC	171.76	112.84	70.58	41.47	30.32	20.34	10.19	5.94	3.04

Surface Water Drainage Concept

2.2.7 The existing ground conditions suggest infiltration of surface water is not recommended and, following the drainage hierarchy for discharge of surface water, the next favourable point of discharge is into a surface water body. An existing drainage channel situated to the north of the site known as South Killingholme Drain Branch 1 is likely to already receive a proportion of runoff from this area. A development extension of a facility north of the site

(VPI Carbon Capture Plant) is proposed as part of a separate scheme. As part of the works South Killingholme Drain Branch 1 is to be re-aligned further south and reconnected to the culvert beneath Rosper Road.

2.2.8 It is proposed to formally drain the hardstanding sections of the site including the access road and roof elements of kiosks via downpipes. There will be no change to the permanent land use or drained area within permeable gravel sections so the existing drainage principles will be maintained. Consequently, no formal drainage is proposed and gravel sections have not been considered as part of the contributing area.

2.2.9 Hardstanding areas are proposed to drain onto proposed filter drains. The filter drains are to be installed with an impermeable membrane to prevent the collection of ground water. A solid pipe branch will collect flows and convey them north for outfall into a swale.

2.2.10 Swale channels aligned adjacent to the proposed access roads will collect surface water runoff and convey flow for connection into the detention basin. The proposed road bridge crossing over the re-aligned drainage channel will be drained via swale channels with a restricted discharge into the realigned channel.

2.2.11 The detention basin will have a pipe outlet discharging to the realigned drainage channel through a flow control device. Any restricted flow will be attenuated within the detention basin and swales. An indicative drainage layout is shown on **Figure 7** in **Annex C**.

2.2.12 The components should be designed as shallow as possible to maintain an invert level above the local ground water level. The lifting of ground levels or implementing impermeable lining in some sections of drainage may be required to ensure this is possible. However, further investigation at a future design stage is recommended to monitor local ground water levels across the site to understand any impact on proposed SuDs components.

Climate Change

2.2.13 A climate change allowance for the 30 year and 100 year events have been applied based on the Environment Agency Flood Risk Assessments: climate change allowances (2022) (Ref 14). The Immingham Facility site falls within the Louth Grimsby and Ancholme Management Catchment. It is noted a 25 year design life is proposed for the overall project. However, for this preliminary assessment it is assumed the civil engineering elements of the site will remain in place beyond 25 years (estimated 2026 construction date) with onsite equipment being refurbished or replaced to continue operation. This would bring the expected lifetime of the development (not necessarily the operational life) beyond the year 2100 and consequently, a robust upper end climate change allowance has been adopted. This equates to a 35% uplift for a 30 year return period and 40% uplift for a 100 year return period as shown on **Table 5**.

Table 5: Louth Grimsby and Ancholme Management Catchment Peak Rainfall Allowances (values used highlighted green)

Epoch	Central Allowance	Upper End Allowance
3.3% Annual Exceedance Rainfall Event		
2050s	20%	35%
2070s	25%	**35%**
1% Annual Exceedance Rainfall Event		
2050s	20%	40%
2070s	25%	**40%**

*Use"2050" for development with a lifetime up 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125

Design Parameters

2.2.14 Swale channels are proposed to capture and convey runoff from the proposed site access roads. Swales have not been sized as part of this study. The channel side slope is to be 1:3 or 1:4 with a 0.5m base width and a minimum of 400mm deep. It may be possible to integrate mini swales with a reduced depth and base width considering the small area of hardstanding to be drained.

2.2.15 The surface water discharge rate is to ideally be controlled to Qbar for events between Qbar (approximately 1 in 2 year event) and 1 in 100 year event.

2.2.16 Discussions with local IDB have confirmed a greenfield discharge rate will be acceptable for the outfalling of runoff from the site into the re-aligned drainage channel. However, the greenfield discharge rates calculated for the site will likely result in outlet diameters smaller than 50-75mm. The blockage risk is discussed further in the Hydraulic Calculation section below. A check at a future design stage is required to confirm the outlet size required for the necessary flow control and the risk of blockage.

2.2.17 The proposed surface water attenuation is to be designed to accommodate a 1 in 100 year design storm event (1% AEP) plus a 40% climate change allowance with no surface water flooding on the site. Complying with NLC design requirement, no water will be stored above ground up to and including the 1 in 100 year event unless stored in a SuDs component.

2.2.18 Catchment descriptors and rainfall data has been downloaded from the Flood Estimation Handbook (FEH) web service (Ref 15) for use in calculations within this report.

Hydraulic Calculations

2.2.19 An InfoDrainage quick storage estimate calculation has been undertaken to understand attenuation requirements against a 1 in 100-year storm event. The calculation is based on a 1.02l/s Qbar discharge rate as calculated in **Table 3**, a 0.410ha drained area and includes a 40% climate change uplift. The default Summer/Winter Cv values in InfoDrainage have been used (0.750/0.840). The results predict a total attenuation storage volume of 322m^3 to 417m^3 is required (these results are estimates only and should not be used for design purposes). An average of the two values (370m^3) has been used for the purposes of this concept strategy but it should be noted that there is space on site to accommodate the larger volume.

2.2.20 A flow control outlet diameter to restrict flow to a 1l/s outflow will likely be under 50-75mm and could be at risk of blockage without protection. To prevent blockage a granular fill could be placed around the outlet to filter out sediment and debris and also prevent vegetation growth. Alternatively, a Hydrobrake arrangement could be used. However, preliminary calculations using Hydro-Internationals online Hydrobrake design tool suggests the outlet diameter will be approximately 50mm with a 0.5m head. The risk of blockage could be deemed reduced by using a Hydrobrake with the device being contained within a chamber with a sump in comparison with a standard orifice.

2.2.21 It is proposed to control the discharge rate as close to Qbar as reasonably practicable to prevent maintenance issues. This may require a discharge rate above the proposed greenfield rate but still controlled to a rate where detrimental flows are unlikely to be passed off site. Further investigation is recommended to understand an acceptable allowable discharge rate and flow control device.

2.2.22 The attenuation storage serving the Immingham Facility and access roads is proposed to be within a detention basin and swales as shown in **Table 6** before discharging from site at the greenfield rate. The connecting swales serving the access road from Rosper Road provide storage area with an assumed 250mm depth of water. The values shown are preliminary and should be updated at a future design stage.

Table 6: Proposed Surface Water Attenuation 1 in 100 Year Event + CC

Swale Length m	Swale Area (250mm depth of water)	Storage Volume within Swale m^3
240	0.38	90
Detention Basin m^2	**Depth m**	**Total Basin Storage Volume m^3**
560	0.5	280
Total Attenuation Storage m^3		370

2.2.23 The swales serving the roads leading up to a bridge crossing the re-aligned drainage channel are proposed to act as attenuation storage with a restricted flow discharge. The available storage volume will likely accommodate the predicted water volume for a 1 in 100 year storm event + climate change as shown in **Table 7**. The corresponding outlet size to control the flow at a greenfield rate would likely be below 50-75mm and be at risk of blockage. Alternatively, it is proposed to use permeable check dams along the swales length and end to slow the flow rate entering the watercourse. A Darcy's Law calculation was undertaken to understand horizontal flow through a granular check dam. A 0.2l/s flow is predicted for a 400 deep, 0.5m base width, 1:4 Side slope channel with a 1 in 250 fall. This method of flow control should be investigated in further detail at a future design stage. It is proposed to reduce the discharge rate as close to Qbar as reasonably practicable to prevent maintenance issues.

Table 7: Proposed Surface Water Attenuation Bridge Crossing Swales 1 in 100 Year Event + CC

Post construction Runoff. 15 min 100 yr +40%cc Rational Method (l/s)	Greenfield Runoff Bridge Area North /Qbar Restriction Rate	Contributing Area (ha)	Restriction Rate via Granular Check Dam (l/s)	Storage Vol Req (m^3)
6	0.03	0.014	0.2	9
Post construction Runoff. 15 min 100 yr +40%cc Rational Method (l/s)	**Greenfield Runoff Bridge Area South/Qbar Restriction Rate**	**Contributing Area (ha)**	**Restriction Rate via Granular Check Dam (l/s)**	**Storage Vol Req (m3)**
4	0.02	0.009	0.2	5

Sustainable Drainage Systems and Water Quality

2.2.24 CIRIA C753 The SuDS Manual (**Ref 7**) outlays a simple index method to account for water quality in the design of SuDS. It indicates the minimum treatment indices appropriate for contributing pollution hazards for different land use classifications. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each containment) that equals or exceeds the pollution hazard index.

2.2.25 The site is considered to have a low pollution hazard level as per Table 26.2 in The SuDS Manual (**Ref 7**).

2.2.26 The pollution hazard indices for a low pollution hazard level and the mitigating indices relating to the selected SuDs component are listed in **Table 8**. The results indicate the use of swales will provide adequate treatment of surface water runoff. As unlined swales are proposed some informal infiltration of runoff may occur. A check of mitigating indices based on the filtration capabilities of the chosen SuDs component and underlying soil properties indicate runoff should be adequately treated before entering ground water systems.

Table 8: Pollution Hazard and Mitigation Indices

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Immingham Facility	Low	0.5	0.4	0.4
SuDs Mitigation Indices for Discharge to Surface Water				
SuDs Component		**TSS**	**Metals**	**Hydrocarbons**
Swale		0.5	0.6	0.6
Attenuation storage		0.5	0.5	0.6
Total SuDs Mitigation–- Index Access Roads[1]		**0.75**	**0.85**	**0.9**
Filter Drain		0.4	0.4	0.4
Attenuation storage		0.5	0.5	0.6
Total SuDs Mitigation Index - Immingham Facility[1]		**0.65**	**0.65**	**0.7**
SuDs Mitigation Indices for Discharge to Ground Water				
Characteristics of material overlaying SuDs		**TSS**	**Metals**	**Hydrocarbons**
Layer of dense vegetation underlain by a soil with good contamination attenuation potential of at least 300mm in depth		**0.6**	**0.5**	**0.6**

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

Total SuDs Mitigation Index = mitigation index $_1$ + 0.5 (mitigation index $_2$)

Operation and Maintenance

2.2.27 An adopting party is to be agreed with the relevant the LLFA and any relevant stakeholders. It is likely the asset owner(s) will be responsible for the maintenance of drainage components.

2.2.28 A key objective of the adoption process is to ensure that any installed SuDS can be maintained easily over the development's lifetime and beyond. Therefore, the SuDS must be designed with maintenance in mind. Proposals for SuDS must include an operation and maintenance document, setting out details on the constructed SuDs and the inspection and maintenance required. This document should be developed at full detailed design but considered throughout the design process. The Operation and Maintenance details considered at this concept design stage are noted below.

2.2.29 Maintenance activities should be conducted in accordance with industry best practice e.g. CIRIA SuDS Manual. The drainage system proposed at Immingham Facility should be inspected at defined intervals and before and after major storm events. The proposed SuDs will require a maintenance regime including grass cutting, removal of sediment build up and clearance of the outfalls at defined intervals. The proposed SuDs features are to be shallow and allow easy access. The filter drains and permeable gravel sections of the site are deemed to have a low risk of sediment build up. The proposed system design life will likely meet the site design life with an adequate inspection and maintenance regime.

3 Washingdales Lane Block Valve Station

3.1 Desktop Study – Washingdales Lane Block Valve Station

Introduction

3.1.1 Three Block Valve Stations are required along the pipeline route to enable pipeline sections to be isolated for operational and maintenance reasons. This section considers Washingdales Lane Block Valve Station located X 519460 Y 407048.

3.1.2 The block valve would be buried with a valve actuator extended above ground (circa 1.5m), include a kiosk, between 2-3m in height and include a local vent to ensure that bypass pipework maintenance activities can be performed safely.

3.1.3 The Block Valve Stations would require security fencing, typically 3.2m high with double-leaf gates for vehicles with access from the adjacent road network, access tracks or similar. The ground surface within the fenced area will predominantly comprise stone with minimal tarmac/concrete internal access roads.

3.1.4 The Block Valve Stations would include associated landscaping such as planting or bunds to provide screening.

3.1.5 Washingdales Lane Block Valve Station is located in an agricultural field adjacent to the Washing Dales Farm access track signposted as Washingdales Lane. A covered reservoir is situated to the south west. See **Annex A** for **Figure 2 – Site Overview and Topography**.

Site Topography

3.1.6 A topographical survey has not yet been undertaken for the Washingdales Lane Block Valve Station. However, a review of available LiDAR information has been undertaken and indicates the site falls to the north east (towards the A18). Ground elevations range from approximately 35mAoD) at the western corner to around 32.5mAoD to the east. The site topography is shown on **Figure 2**, **Annex A**.

Local Hydrology

3.1.7 The main river watercourse situated closest to the site location is Laceby Beck (approx. 2.3km east).

3.1.8 A ditch drain is located 570m to the south of the site, which is assumed to take runoff from the field, discharging into Laceby Beck to the east and ultimately the North Sea via River Freshney.

3.1.9 The Environment Agency flood maps indicates the site has a very low risk of flooding (1 in 1000 year/0.1% AEP each year) from rivers or the sea. The site also has a very low risk of flooding from surface water.

Ground Conditions, Ground water and Infiltration

3.1.10 A review of the BGS Geology Viewer indicates the bedrock geology is Burnham Chalk Formation. with superficial deposits classed as Till, Devensian – Diamicton.

3.1.11 The site underlaying strata is classified as Principal Bedrock Aquifer and a secondary (undifferentiated) Superficial Drift Aquifer. The site sits in a medium-high ground water vulnerability area with a Soluble Rock Risk.

3.1.12 The BGS borehole records of holes drilled nearby indicate a silty topsoil overlaying silty clay with scattered chalk gravel. The two logs indicate a bedrock of chalk at shallow and deep levels.

3.1.13 A review of the Soil-Scapes layer on Magic maps indicates the site is situated in an area of freely draining lime-rich loamy soils on arable ground and grassland. The site is located in Zone II Source Protection Zone (outer protection zone regarding the safeguarding of drinking water quality).

3.1.14 Following the description of surface geology above, and the site location being situated on higher ground, it may be possible to discharge surface water via infiltrating methods.

Existing Utilities

3.1.15 A desktop study was undertaken by GroundSure to gather available utility information from providers. This was submitted to AECOM as an AutoCAD DWG file covering the pipeline alignment and a buffer area either side. The DWG information indicates there are High Voltage Northern Power Grid cables and an Openreach duct aligned with Washingdales Lane. No utilities were shown within the site boundary.

3.2 Surface Water Drainage Strategy – Washingdales Lane Block Valve Station

Contributing Areas and Runoff Calculation

3.2.1 The contributing area has been measured from a scheme layout drawing produced by Penspen (drawing number: EN070008/APP/4.14). The proposed impermeable and permeable areas are summarised in **Table 9**.

3.2.2 The site will be predominantly permeable with unpaved areas to be graded to natural ground levels overlain with weed control membrane and 75 mm of 20mm single size gravel.

3.2.3 Impermeable areas will consist of a 5m wide facility access road spurring from Washingdales Lane. The site will have two fenceline boundaries including a timber fenceline around a planting strip and a security fenceline between the planting strip and Washingdales Lane Block Valve Station. Within the security fence line boundary a 4m wide splayed road and turning head is proposed to allow access to the car park. 1No. 3 x 3.5m kiosk with a flat roof will also be situated within the site boundary sat upon a concrete base.

3.2.4 The sites will be cleared, excavated and graded to achieve the approximate required finished levels. Surfaces will be constructed to falls so that rainwater can drain to the proposed drainage system. Roads and hardstanding will have flush concrete kerbs to allow surface water run-off. Most of the site will be permeable surfacing to minimise runoff. A cut-off drainage channel maybe required at the site entrance gate to control runoff onto site.

Table 9: Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment
Total Site Area	N/A	1681	Access road and area within fence line
Stone area	Permeable	329	The majority of ground surface within the fence line is to be stone aggregate
Planting Strip	Permeable	1046	A planting strip is proposed around the Washingdales Lane Block Valve Station perimeter to hide proposed infrastructure

Ref	Surface Type	Area (m^2)	Comment
Roads Inside Washingdales Lane Block Valve Station Fence line	Impermeable	75	Access turning and parking is proposed to access site
Roads Outside Washingdales Lane Block Valve Station Fence line	Impermeable	221	Roads to enable access to the Washingdales Lane Block Valve Station
Roofs	Impermeable	11	1 site kiosk is proposed
Totals		**m^2**	**ha**
Total Impermeable Area		306	0.031
Total Permeable Area		1375	0.138
Total Contributing area (impermeable area)		306	0.031

Greenfield Runoff

3.2.5 The greenfield runoff rates for the proposed Washingdales Lane Block Valve Station have been calculated based on the IH124 method using the HR Wallingford UK SuDS website (Ref 13). The greenfield runoff rates for a 50ha area were calculated using this method. A summary of the results can be seen in the calculation report found in **Annex B**, with the peak greenfield runoff rates for the total contributing area interpolated from the results shown in **Table 10**.

Table 10: Peak Greenfield Runoff Rate

Rainfall Event Frequency	Runoff (l/s/ha)	Site Contributing Area (0.031ha) GF Runoff l/s
1 in 1 Year (Approx. 99% AEP)	3.82	0.12
Qbar	4.39	0.13
1 in 30 Year (3.33% AEP)	10.75	0.33
1 in 100 Year (1% AEP)	15.62	0.48

Proposed Surface Water Runoff Rates

3.2.6 **Table 11** below shows the unrestricted surface water runoff rate post-development based on the Modified Rational Method. This method estimates runoff based on the nature of the ground surface (hardstanding, vegetation etc.) and rainfall depth, duration and frequency information for the immediate area, as follows:

- C (Coefficient of impermeability) = 1.0;
- A (area) = ha; 0.031; and
- i (Rainfall intensity based on FEH data (Ref 15)).

Table 11: Proposed Peak Runoff Rate

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	2.48	1.60	0.99	0.69	0.54	0.39	0.21	0.12	0.07
5 (20%)	4.14	2.66	1.64	1.06	0.80	0.56	0.29	0.16	0.10
10 (10%)	5.28	3.41	2.11	1.31	0.98	0.67	0.34	0.19	0.11
30 (3.3%)	7.12	4.63	2.87	1.72	1.26	0.85	0.42	0.24	0.14
50 (2%)	7.95	5.20	3.24	1.91	1.40	0.94	0.47	0.27	0.16
100 (1%)	9.10	6.00	3.75	4.37	1.59	1.06	0.53	0.31	0.16
100 +20% CC	10.92	7.20	4.50	2.62	1.91	1.27	0.63	0.37	0.19
100 +40% CC	12.74	8.40	5.25	3.06	2.22	1.48	0.74	0.43	0.22

Surface Water Drainage Concept

3.2.7 Based on an evaluation of local ground conditions it is recommended to infiltrate runoff using an infiltration trench or similar if site conditions deem this possible through site testing (trial holes, infiltration tests to BRE365 and ground water level monitoring). Site investigation is recommended to understand the site infiltration rate, assess ground conditions/ inspect for contamination and check any potential adverse ground water that would impact upon infiltration SuDs components.

3.2.8 It is proposed to formally drain the hardstanding sections of the site including the access road and roof elements of kiosks via downpipes. Consideration of runoff from the permeable section of the site has been included as a percentage of the total area. The remaining area is to be constructed from permeable material and consequently these areas can continue to drain informally as per existing conditions with limited risk of increasing runoff or flood risk. An infiltration trench is proposed to be aligned adjacent to the access road and splayed Washingdales Lane Block Valve Station access to capture runoff from the carriageways. The roof downpipes will connect into a piped drainage branch connecting into the infiltration trench. An indicative drainage layout is shown on **Figure 8** in **Annex C**.

3.2.9 A site survey will be undertaken to understand if any land drainage systems exist beneath the site or within the vicinity before any onsite activities commence. Consideration of land drainage is required to ensure it is not disrupted by the construction of the facility. This will allow the facility and surrounding land to continue to drain as per the existing drainage regime with the incorporation of infiltration trenches. Further investigation is recommended to understand local ground water levels across the site to understand any impact on proposed infiltration SuDs components.

Climate Change

3.2.10 A climate change allowance for the 30 year and 100 year events have been applied based on the Environment Agency Flood Risk Assessments: climate change allowances (2022) (**Ref 14**). The Washingdales Lane Block Valve Station site falls within the Louth Grimsby and Ancholme Management Catchment. It is noted a 25 year design life is proposed for the overall scheme. However, for this preliminary assessment it is assumed the civil engineering elements of the site will remain in place beyond 25 years (estimated 2026 construction date) with onsite equipment being refurbished or replaced to continue operation. This would bring the expected lifetime of the development (not necessarily the operational life) beyond the year 2100 and consequently, a robust upper end climate change allowance has been adopted. This equates to a 35% uplift for a 30 year return period and 40% uplift for a 100 year return period as shown on **Table 12**.

Table 12: Louth Grimsby and Ancholme Management Catchment Peak Rainfall Allowances (values used highlighted green)

Epoch	Central Allowance	Upper End Allowance
3.3% Annual Exceedance Rainfall Event		
2050s	20%	35%
2070s	25%	**35%**
1% Annual Exceedance Rainfall Event		
2050s	20%	40%
2070s	25%	**40%**

*Use '2050s' for development with a lifetime up 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125

Design Parameters

3.2.11 Infiltration trenches are proposed to capture and attenuate runoff from the proposed site. Trenches are to be approximately 1m wide with a 0.3 void ratio.

3.2.12 The proposed surface water infiltration trench is to be designed to accommodate a 1 in 100 year design storm event (1% AEP) plus a 40% climate change allowance with no surface water flooding on the site. The trench is to have a half drain down time within 24 hours.

3.2.13 Catchment descriptors and rainfall data has been downloaded from the Flood Estimation Handbook (FEH) web service (Ref 15) for use in calculations within this report.

Hydraulic Calculations

3.2.14 A MicroDrainage Quick Design Infiltration Systems calculation was undertaken to understand component sizes. A nominal infiltration rate of 0.036m/hr (Table 25.1 The SuDs Manual (**Ref 7**)) has been selected to represent infiltration at the site based on loamy soils and the potential of land drainage in the vicinity. The infiltration rate will require confirmation through site testing to ensure it is feasible to discharge runoff to ground. Simulations were run for the drained area shown in **Table 9** and 40% climate change against the 1 in 100 year storm event. Results predict runoff could be infiltrated in a 1m wide x 40m long infiltration trench with a half drain down time under 24 hours. The predicted length of trench could fit adjacent to the proposed access roads and car park.

3.2.15 A MicroDrainage Quick Design Infiltration Systems calculation against a silty clay loam has also been undertaken using a 0.0036m/hr infiltration rate. When testing against a factor of safety equal to 2, the results suggested a 0.4 wide 136m long infiltration trench which would not fit within the proposed site.

3.2.16 There is little risk to infrastructure or the surrounding area if the infiltration performance is uncertain. Consequently, the factor of safety can be reduced from 2 to 1.5 in the calculation complying with The SuDs Manual Table 25.2 (**Ref 7**). Suggested Factors of Safety for Infiltration Systems. Results predict runoff could be infiltrated in a 0.6m wide x 80.5m long infiltration trench with a half drain down time under 24 hours. The predicted length of trench can fit adjacent to the proposed access roads and car park.

Sustainable Drainage Systems and Water Quality

3.2.17 CIRIA C753 The SuDS Manual (**Ref 7**) outlays a simple index method to account for water quality in the design of SuDS. It indicates the minimum treatment indices appropriate for contributing pollution hazards for different land use classifications. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each containment) that equals or exceeds the pollution hazard index.

3.2.18 The Washingdales Lane Block Valve Station will be unmanned and will therefore have infrequent vehicle movements and no polluting activities are expected. Consequently, the site is considered to have a low pollution hazard level as per Table 26.2 in The SuDS Manual (**Ref 7**). The pollution hazard indices for a low pollution hazard level and the mitigating indices relating to the selected SuDs component are listed in **Table 13**. The results indicate the use of infiltration will provide adequate treatment of surface water runoff for Metals and Hydrocarbons but not Total Suspended Solids. The site is unlikely to produce harmful levels of suspended solids from the drained surfaces. Also, the movement of runoff through the remaining ground towards land drainage or in natural ground water movements would likely allow suspended solids to drop out. This treatment approach is therefore considered to be appropriate for the site.

Table 13: Pollution Hazard and Mitigation Indices

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Block Valve 1	Low	0.5	0.4	0.4
SuDs Mitigation Indices for Discharge to Surface Water				
SuDs Component		**TSS**	**Metals**	**Hydrocarbons**
N/A				
SuDs Mitigation Indices for Discharge to Ground Water				
Characteristics of material overlaying SuDs		**TSS**	**Metals**	**Hydrocarbons**
Infiltration trench underlain by a soil with good contamination attenuation potential of at least 300mm in depth		0.4	0.4	0.4

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

Total SuDs Mitigation Index = mitigation index $_1$ + 0.5 (mitigation index $_2$)

Operation and Maintenance

3.2.19 An adopting party is to be agreed with the relevant the LLFA and any relevant stakeholders. It is likely the asset owner will be responsible for the maintenance of drainage components.

3.2.20 A key objective of the adoption process is to ensure that any installed SuDS can be maintained easily over the development's lifetime and beyond. Therefore, the SuDS must be designed with maintenance in mind. Proposals for SuDS must include an operation and maintenance document, setting out details on the constructed SuDs and the inspection and maintenance required. This document should be developed at full detailed design but considered throughout the design process. The Operation and Maintenance details considered at this concept design stage are noted below.

3.2.21 Maintenance activities should be conducted in accordance with industry best practice e.g. CIRIA SuDS Manual. The drainage system proposed at Block Valve 1 should be inspected at defined intervals and before and after major storm events. The proposed SuDs will require a maintenance regime including vegetation control around infiltration components and aggregate removal and cleaning/sediment removal at defined intervals. The proposed SuDs features are proposed to be shallow and allow easy access. The proposed system design life will likely meet the site design life with an adequate inspection and maintenance regime.

4 Thoroughfare Block Valve

4.1 Desktop Study – Thoroughfare Block Valve

Introduction

4.1.1 Three Block Valve Stations are required along the pipeline route to enable pipeline sections to be isolated for operational and maintenance reasons. This section considers Thoroughfare Block Valve located at X 526248 Y 400236.

4.1.2 The Block Valve Station would be buried with a valve actuator extended above ground (circa 1.5m), housed in a kiosk, between 2-3m in height and include a local vent to ensure that bypass pipework maintenance activities can be performed safely.

4.1.3 The Block Valve Stations would require security fencing, typically 2.4m high with double-leaf gates for vehicles with access from the adjacent road network, access tracks or similar. The ground surface within the fenced area will predominantly comprise stone with minimal tarmac/concrete internal access roads.

4.1.4 The Block Valve Stations would include associated landscaping such as planting or bunds to provide screening.

4.1.5 Thoroughfare Block Valve Station is located in an agricultural field adjacent to a single track lane known as Thoroughfare which is lined with trees in the vicinity of the site. A farm track cuts through the fields to the east of the site. See **Annex A** for **Figure 3 – Site Overview and Topography**.

Site Topography

4.1.6 A topographical survey has not yet been undertaken for Thoroughfare Block Valve Station. However, a review of available LiDAR information has been undertaken and indicates the site relatively flat with an easterly fall (towards the access track). Ground elevations range from approximately 22.5mAoD at the western corner to around 22.3mAoD in the east. The site topography is shown on **Figure 3**, **Annex A**.

Local Hydrology

4.1.7 The Waithe Beck main river watercourse is situated 1.14km to the north east of the site.

4.1.8 OS mapping suggests a short length of drain exists in the north east corner of the site. A drain is also noted to exist 140m to the south.

4.1.9 The Environment Agency flood maps indicate the site has a very low risk of flooding (1 in 1000 year/0.1% AEP each year) from rivers.

4.1.10 The surface water flood maps indicate the site has low risk of flooding (1 in 100 year/1% AEP to 1 in 1000 year/0.1% AEP each year). The mapping suggests runoff collects at a low lying section of the field at the farm track/Thoroughfare road junction where low-lying ground levels allows surface water to flow north east towards Waithe Beck (see **Figure 3**, **Annex A**).

Ground Conditions, Ground water and Infiltration

4.1.11 A review of the BGS Geology Viewer indicates the bedrock geology is Welton Chalk Formation with superficial deposits classed as Till, Devensian – Diamicton.

4.1.12 The site underlaying strata is classified as Principal Bedrock Aquifer and a secondary (undifferentiated) Superficial Drift Aquifer. The site sits in a medium ground water vulnerability area.

4.1.13 The BGS borehole records of holes drilled in the site vicinity indicate clay with layers of sands and gravels. The bedrock found is noted to be Chalk at 108ft below ground level (33m).

4.1.14 A review of the Soil-Scapes layer on Magic maps indicates the site is situated in an area of slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The drainage type is described as impeded drainage. The site is located in Zone III Source Protection Zone (regarding the safeguarding of drinking water quality).

4.1.15 Following the description of surface geology above it is not recommended to discharge surface water via infiltrating methods.

Existing Utilities

4.1.16 A desktop study was undertaken by GroundSure to gather available utility information from providers. This was submitted to AECOM as an AutoCAD DWG file covering the pipeline alignment and a buffer area either side. The DWG information indicates there is a 33kv Northern Power Grid cable north of the site. No utilities were shown within the site boundary.

4.2 Surface Water Drainage Strategy – Thoroughfare Block Valve Station

Contributing Areas and Runoff Calculation

4.2.1 The contributing area has been measured from a scheme layout drawing produced by Penspen (drawing number: EN070008/APP/4.15). The proposed impermeable and permeable areas are summarised in **Table 14**.

4.2.2 The site will be predominantly permeable with unpaved areas to be graded to natural ground levels overlain with weed control membrane and 75 mm of 20mm single size gravel.

4.2.3 Impermeable areas will consist of a 5m wide facility access road spurring from the Thoroughfare single track road. The site will have two fenceline boundaries including a timber fenceline around a planting strip and a security fenceline between the planting strip and Thoroughfare Block Valve Station. Within the security fence line boundary a 4m wide splayed road access and turning head is proposed to allow access to the car park. 1 No. 3 x 3.5m kiosk with a flat roof will also be situated within the site boundary sat upon a concrete base.

4.2.4 The sites will be cleared, excavated and graded to achieve the approximate required finished levels. Surfaces will be constructed to falls so that rainwater can drain to any proposed drainage system. Roads and hardstanding will have flush concrete kerbs to allow surface water run-off. Most of the site will be permeable surfacing to minimise runoff. A cut-off drainage channel maybe required at the site entrance gate to control runoff onto site.

Table 14: Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment
Total Site Area	N/A	1693	Access road and area within fence line
Stone Area	Permeable	329	The majority of ground surface within the fence line is to be stone aggregate
Planting Strip	Permeable	1046	A planting strip is proposed around the Thoroughfare Block Valve Station perimeter to hide proposed infrastructure
Roads Inside Thoroughfare Block Valve Station Fence line	Impermeable	75	Access turning and parking is proposed to access the site
Roads Outside Thoroughfare Block Valve Station Fence line	Impermeable	233	Roads to enable access to the Thoroughfare Block Valve Station
Roofs	Impermeable	11	1 site kiosk is proposed
Totals		**m^2**	**ha**
Total Impermeable Area		318	0.032
Total Permeable Area		1375	0.137
Total Contributing area (impermeable area)		318	0.032

Greenfield Runoff

4.2.5 The greenfield runoff rates for the proposed Thoroughfare Block Valve Station have been calculated based on the IH124 method using the HR Wallingford UK SuDS website. The greenfield runoff rates for a 50ha area were calculated using this method. A summary of the results can be seen in the calculation report found in **Annex B**, with the peak greenfield runoff rates for the total contributing area interpolated from the results shown in **Table 15**.

Table 15: Peak Greenfield Runoff Rate

Rainfall Event Frequency	Runoff (l/s/ha)	Site Contributing Area (0.032ha) GF Runoff l/s
1 in 1 Year (Approx. 99% AEP)	3.82	0.12
Qbar	4.40	0.14
1 in 30 Year (3.33% AEP)	10.77	0.34
1 in 100 Year (1% AEP)	15.65	0.50

Proposed Surface Water Runoff Rates

4.2.6 **Table 16** below shows the unrestricted surface water runoff rate post-development based on the Modified Rational Method. This method estimates runoff based on the nature of the ground surface (hardstanding, vegetation etc.) and rainfall depth, duration and frequency information for the immediate area, as follows:

- C (Coefficient of impermeability) = 1.0;
- A (area) = ha; 0.032; and
- I (Rainfall intensity based on FEH data (Ref 15)).

Table 16: Proposed Peak Runoff Rate

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	2.70	1.74	1.07	0.75	0.59	0.42	0.22	0.13	0.08
5 (20%)	4.55	2.92	1.81	1.16	0.88	0.60	0.30	0.17	0.10
10 (10%)	5.86	3.77	2.34	1.45	1.08	0.73	0.36	0.20	0.12
30 (3.3%)	7.90	5.13	3.19	1.90	1.39	0.93	0.46	0.26	0.15
50 (2%)	8.82	5.77	3.59	2.12	1.54	1.03	0.50	0.29	0.16
100 (1%)	10.10	6.65	4.16	4.84	1.75	1.16	0.57	0.33	0.16
100 +20% CC	12.12	7.98	4.99	2.90	2.10	1.40	0.69	0.40	0.20
100 +40% CC	14.14	9.31	5.82	3.39	2.46	1.63	0.80	0.46	0.23

Surface Water Drainage Concept

4.2.7 Existing ground conditions suggests infiltration of surface water is not recommended and, following the drainage hierarchy for discharge of surface water, the next favourable point of discharge is into a surface water body. An existing drainage ditch situated to the north of the site will receive runoff from the existing road and some field area. The ditch falls northeast towards a low point in the field which is suspected to be culverted beneath Thorough Road or the field access track for connection with low lying surface water ditches.

4.2.8 It is proposed to formally drain the hardstanding sections of the site including the access road and roof elements of kiosks via downpipes. Swale channels aligned adjacent to the proposed access road will collect surface water runoff and convey flow for connection into the field edge drainage ditch. The connection into the existing ditch will include a control to restrict flow to a set discharge rate. Any restricted flow will be attenuated within a detention basin in the planting strip. The outfall location has been positioned in the north eastern corner to allow connection into suspected deeper sections of the existing field edge drainage ditch. The remaining area is to be constructed from permeable material and consequently these areas can continue to drain informally as per existing conditions with limited risk of increasing runoff or flood risk to the detriment of the site and its surroundings. An indicative drainage layout is shown on **Figure 9** in **Annex C**.

4.2.9 A site survey will be undertaken to understand if any land drainage systems exist beneath the site or within the vicinity before any on-site activities commence. Consideration of land drainage is required to ensure it is not disrupted by the construction of the facility. This will allow the facility and surrounding land to continue to drain as per the existing drainage regime with the incorporation of sustainable drainage. Further investigation is recommended to understand local ground water levels across the site to understand any impact on proposed SuDs components.

4.2.10 The components should be designed as shallow as possible to maintain an invert level above the local ground water level. The lifting of ground levels or implementing impermeable lining in some sections of drainage may be required to ensure this is possible. However, if existing land drainage exists the ground water level will be artificially lowered. Further investigation is recommended to understand local ground water levels across the site to understand any impact on proposed SuDs components.

Climate Change

4.2.11 A climate change allowance for the 30 year and 100 year events have been applied based on the Environment Agency Flood Risk Assessments: climate change allowances (2022) (Ref 14). The Block Valve 2 Facility site falls within the Louth Grimsby and Ancholme Management Catchment. It is noted a 25 year design life is proposed for the overall scheme. However, for this preliminary assessment it is assumed the civil engineering elements of the site will remain in place beyond 25 years potentially up to 75-100 years (estimated 2026 construction date) with onsite equipment being refurbished or replaced to continue operation. This would bring the expected lifetime of the development (not necessarily the operational life) beyond the year 2100 and consequently, a robust upper end climate change allowance has been adopted. This equates to a 35% uplift for a 30 year return period and 40% uplift for a 100 year return period as shown on **Table 17**.

Table 17: Louth Grimsby and Ancholme Management Catchment Peak Rainfall Allowances (values used highlighted green)

Epoch	Central Allowance	Upper End Allowance
3.3% Annual Exceedance Rainfall Event		
2050s	20%	35%
2070s	25%	**35%**
1% Annual Exceedance Rainfall Event		
2050s	20%	40%
2070s	25%	**40%**

*Use"2050" for development with a lifetime up 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125

Design Parameters

4.2.12 Swale channels are proposed to capture and convey runoff from the proposed site. Swales have not been sized as part of this study. The channel side slope is to be 1:3 or 1:4 with a 0.5m base width and a minimum of 400mm deep. It may be possible to integrate mini swales with a reduced depth and base width considering the small area of hardstanding to be drained.

4.2.13 The surface water discharge rate is to ideally be controlled to Qbar for events between Qbar (approximately 1 in 2 year event) and 1 in 100 year event.

4.2.14 Discussions with local IDB have confirmed greenfield discharge rates are preferred, however other rates and outlet sizes are considered on a mitigated, rational and evidential basis. The greenfield discharge rates calculated for the site will likely result in outlet diameters smaller than 50-75mm. The blockage risk is discussed further in the Hydraulic Calculation section below. A check at a future design stage is required to confirm the outlet size required for the necessary flow control and the risk of blockage.

4.2.15 The proposed surface water attenuation is to be designed to accommodate a 1 in 100 year design storm event (1% AEP) plus a 40% climate change allowance with no surface water flooding on the site. No water will be stored above ground up to and including the 1 in 100 year event unless stored in a SuDs component.

4.2.16 Catchment descriptors and rainfall data has been downloaded from the Flood Estimation Handbook (FEH) web service (Ref 15) for use in calculations within this report.

Hydraulic Calculations

4.2.17 An InfoDrainage quick storage estimate calculation has been undertaken to understand attenuation requirements against a 1 in 100-year storm event. The calculation is based on a 0.14l/s Qbar discharge rate as calculated in **Table 15**, a 0.032ha drained area and includes a 40% climate change uplift. The default Summer Winter Cv values in InfoDrainage have been used (0.750/0.840). The results predict an attenuation storage volume of 24m^3 to 31m^3 is required (these results are estimates only and should not be used for design purposes). An average of the two values (28m^3) has been used for the purposes of this concept strategy.

4.2.18 To meet greenfield rates the flow discharge control device would likely have a small opening and be at risk of blockage. It is proposed to control the discharge rate as close to Qbar as reasonably practicable to prevent maintenance issues. This may require a discharge rate above the proposed greenfield rate but still controlled to a rate where detrimental flows are unlikely to be passed off site. Further investigation is recommended to understand an acceptable allowable discharge rate and flow control device.

Sustainable Drainage Systems and Water Quality

4.2.19 CIRIA C753 The SuDS Manual (**Ref 7**) outlays a simple index method to account for water quality in the design of SuDS. It indicates the minimum treatment indices appropriate for contributing pollution hazards for different land use classifications. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each containment) that equals or exceeds the pollution hazard index.

4.2.20 The Block Valve Station will be unmanned and will therefore have infrequent vehicle movements and no polluting activities are expected. Consequently, the site is considered to have a low pollution hazard level as per Table 26.2 in The SuDS Manual (**Ref 7**).

4.2.21 The pollution hazard indices for a low pollution hazard level and the mitigating indices relating to the selected SuDs component are listed in **Table 18**. The results indicate the use of swales will provide adequate treatment of surface water runoff. As unlined swales are proposed some informal infiltration of runoff may occur. A check of mitigating indices based on the filtration capabilities of the chosen SuDs component and underlying soil properties indicate runoff should be adequately treated before entering ground water systems.

Table 18: Pollution Hazard and Mitigation Indices

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Block Valve 2	Low	0.5	0.4	0.4
SuDs Mitigation Indices for Discharge to Surface Water				
SuDs Component		**TSS**	**Metals**	**Hydrocarbons**
Swale		0.5	0.6	0.6
Attenuation storage		0.5	0.5	0.6
Total SuDs Mitigation Index[1]		0.75	0.85	0.9
SuDs Mitigation Indices for Discharge to Ground Water				

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Characteristics of material overlaying SuDs		**TSS**	**Metals**	**Hydrocarbons**
Layer of dense vegetation underlain by a soil with good contamination attenuation potential of at least 300mm in depth		0.6	0.5	0.6

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

Total SuDs Mitigation Index = mitigation index $_1$ + 0.5 (mitigation index $_2$)

Operation and Maintenance

4.2.22 An adopting party is to be agreed with the LLFA and any relevant stakeholders. It is likely the asset owner will be responsible for the maintenance of drainage components.

4.2.23 A key objective of the adoption process is to ensure that any installed SuDS can be maintained easily over the Proposed Development's lifetime and beyond. Therefore, the SuDS must be designed with maintenance in mind. Proposals for SuDS must include an operation and maintenance document, setting out details on the constructed SuDs and the inspection and maintenance required. This document should be developed at full detailed design but considered throughout the design process. The Operation and Maintenance details considered at this concept design stage are noted below.

4.2.24 Maintenance activities should be conducted in accordance with industry best practice e.g. CIRIA SuDS Manual. The drainage system proposed at Block Valve 2 should be inspected at defined intervals and before and after major storm events. The proposed SuDs will require a maintenance regime including grass cutting, removal of sediment build up and clearance of the outfalls at defined intervals. The proposed SuDs features are to be shallow and allow easy access. The proposed system design life will likely meet the site design life with an adequate inspection and maintenance regime.

5 Louth Road Block Valve Station

5.1 Desktop Study – Louth Road Block Valve Station

Introduction

5.1.1 Three Block Valve Stations are required along the pipeline route to enable pipeline sections to be isolated for operational and maintenance reasons. This section considers Louth Road Block Valve Station located at X 535809 Y 390543.

5.1.2 The Block Valve Station would be buried with a valve actuator extended above ground (circa 1.5m), housed in a kiosk, between 2-3m in height and include a local vent to ensure that bypass pipework maintenance activities can be performed safely.

5.1.3 The Block Valve Stations would require security fencing, typically 2.4m high with double-leaf gates for vehicles with access from the adjacent road network, access tracks or similar. The ground surface within the fenced area will predominantly comprise stone with minimal tarmac/concrete internal access roads.

5.1.4 The Block Valve Stations would include associated landscaping such as planting or bunds to provide screening.

5.1.5 Louth Road Block Valve Station is located in an agricultural field adjacent to Alvingham Road which is lined with bushes along the field boundaries. An existing twin farm gate acts as access to the field in the vicinity of the proposed Louth Road Block Valve Station. See **Annex A** for **Figure 4 – Site Overview and Topography**.

Site Topography

5.1.6 A topographical survey has not yet been undertaken for the Louth Road Block Valve Station. However, a review of available LiDAR information has been undertaken and indicates the site falls to the south (towards Alvingham Road). Ground elevations range from approximately 14mAOD at the northern edge to around 13mAOD in the south. The road adjacent crests with the carriageway falling in both directions from this point. The site topography is shown on **Figure 4, Annex A**.

Local Hydrology

5.1.7 The River Lud main river watercourse is situated 420m to the south east of the site. Also to the southeast is the Louth Canal which runs between the site and River Lud at a distance of approximately 250m. It is understood the Louth Canal is a canalisation of the River Lud and is maintained for the continuation of land drainage and water supply.

5.1.8 A visible ditch drain runs adjacent to the southern edge of Alvingham Road east of the site. The drain continues for approximately 180m before changing direction to the southeast and connecting into the Louth Canal. Runoff from the current site area is assumed to be collected by land drainage following the fall of the land south and to the southwest. Field/road edge ditches are assumed to exist but are not visible on available online street maps, hidden by field boundary hedges. Another ditch drain falling towards the Louth Canal following a field boundary is situated approximately 200m south west of the site.

Ground Conditions, Ground water and Infiltration

5.1.9 A review of the BGS Geology Viewer indicates the bedrock geology is Welton Chalk Formation with superficial deposits classed as Till, Devensian – Diamicton.

5.1.10 The site underlaying strata is classified as Principal Bedrock Aquifer and a secondary (undifferentiated) Superficial Drift Aquifer. The site sits in a medium ground water vulnerability area.

5.1.11 The BGS borehole records of holes drilled in the site vicinity indicate clay above a layer of sand. The bedrock found is noted to be Chalk at 171ft below ground level (52m).

5.1.12 A review of the Soil-Scapes layer on Magic maps indicates the site is situated in an area of slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The drainage type is described as impeded drainage. The site is not located in a Source Protection Zone but is noted to be in a Drinking Water Protected Area under Anglian Water.

5.1.13 Following the description of surface geology above it is not recommended to discharge surface water via infiltrating methods.

Existing Utilities

5.1.14 A desktop study was undertaken by GroundSure to gather available utility information from providers. This was submitted to AECOM as an AutoCAD DWG file covering the pipeline alignment and a buffer area either side. The DWG information indicates there is an Anglian Water foul water pipeline aligned with Alvingham Road south of the site. No utilities were shown within the site boundary.

5.2 Surface Water Drainage Strategy – Louth Road Block Valve Station

Contributing Areas and Runoff Calculation

5.2.1 The contributing area has been measured from a scheme layout drawing produced by Penspen (drawing number: EN070008/APP/4.16). The proposed impermeable and permeable areas are summarised in **Table 19**.

5.2.2 The site will be predominantly permeable with unpaved areas to be graded to natural ground levels overlain with weed control membrane and 75 mm of 20mm single size gravel.

5.2.3 Impermeable areas will consist of a 5m wide facility access road spurring from the Alvingham Road. The site will have two fenceline boundaries including a timber fenceline around a planting strip and a security fenceline between the planting strip and Louth Road Block Valve Station. Within the security fence line boundary a 4m wide splayed road access and turning head is proposed to allow access to the car park. 1No. 3 x 3.5m kiosk with a flat roof will also be situated within the site boundary sat upon a concrete base.

5.2.4 The sites will be cleared, excavated and graded to achieve the approximate required finished levels. Surfaces will be constructed to falls so that rainwater can drain to any proposed drainage system. Roads and hardstanding will have flush concrete kerbs to allow surface water run-off. Most of the site will be permeable surfacing to minimise runoff. A cut-off drainage channel maybe required at the site entrance gate to control runoff offsite.

Table 19: Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment
Total Site Area	N/A	1681	Access road and area within fence line
Stone area	Permeable	329	The majority of ground surface within the fence line is to be stone aggregate

Ref	Surface Type	Area (m^2)	Comment
Planting Strip	Permeable	1046	A planting strip is proposed around the Louth Road Block Valve Station perimeter to hide proposed infrastructure
Roads Inside Louth Road Block Valve Station Fence line	Impermeable	75	Access turning and parking is proposed to access site
Roads Outside Louth Road Block Valve Station Fence line	Impermeable	221	Roads to enable access to the Louth Road Block Valve Station
Roofs	Impermeable	11	1 site kiosk is proposed
Totals		**m^2**	**ha**
Total Impermeable Area		306	0.031
Total Permeable Area		1375	0.137
Total Contributing area (impermeable area)		306	0.031

Greenfield Runoff

5.2.5 The greenfield runoff rates for the proposed Louth Road Block Valve Station have been calculated based on the IH124 method using the HR Wallingford UK SuDS website. The greenfield runoff rates for a 50ha area were calculated using this method. A summary of the results can be seen in the calculation report found in **Annex B**, with the peak greenfield runoff rates for the total contributing area interpolated from the results shown in **Table 20**.

Table 20: Peak Greenfield Runoff Rate

Rainfall Event Frequency	Runoff (l/s/ha)	Site Contributing Area (0.031ha) GF Runoff l/s
1 in 1 Year (Approx. 99% AEP)	3.78	0.12
Qbar	4.35	0.13
1 in 30 Year (3.33% AEP)	10.65	0.33
1 in 100 Year (1% AEP)	15.48	0.47

Proposed Surface Water Runoff Rates

5.2.6

5.2.7 **Table** 21 below shows the unrestricted surface water runoff rate post-development based on the Modified Rational Method. This method estimates runoff based on the nature of the ground surface (hardstanding, vegetation etc.) and rainfall depth, duration and frequency information for the immediate area, as follows:

- C (Coefficient of impermeability) = 1.0;
- A (area) = ha; 0.031; and
- i (Rainfall intensity based on FEH data (Ref 15)).

Table 21: Proposed Peak Runoff Rate

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	2.67	1.72	1.06	0.74	0.58	0.41	0.21	0.12	0.07
5 (20%)	4.55	2.92	1.81	1.16	0.87	0.60	0.30	0.17	0.10
10 (10%)	5.87	3.78	2.35	1.45	1.08	0.73	0.36	0.20	0.12
30 (3.3%)	7.94	5.16	3.21	1.91	1.40	0.93	0.46	0.26	0.15
50 (2%)	8.89	5.81	3.62	2.13	1.56	1.04	0.51	0.29	0.17
100 (1%)	10.19	6.71	4.19	4.89	1.77	1.18	0.59	0.34	0.17
100 +20% CC	12.23	8.05	5.03	2.94	2.13	1.42	0.70	0.41	0.20
100 +40% CC	14.27	9.40	5.87	3.42	2.48	1.65	0.82	0.48	0.23

Surface Water Drainage Concept

5.2.8 Existing ground conditions suggest infiltration of surface water is not recommended and, following the drainage hierarchy for discharge of surface water, the next favourable point of discharge is into a surface water body. An existing drainage ditch is situated to the south of the site, likely receiving runoff from the existing road and field. The ditch falls south west towards a perpendicular drainage channel connecting into the canal. Further investigation will be required to confirm the field drainage connectivity and to understand the impact the proposed site will have upon any existing land drainage.

5.2.9 It is proposed to formally drain the hardstanding sections of the site, including; the access road, and roof elements of kiosks via downpipes. Swale channels aligned adjacent to the proposed access road will collect surface water runoff and convey flow for connection into the field edge drainage ditch. The connection into the existing ditch will include a control to restrict flow to a set discharge rate. Any restricted flow will be attenuated within a detention basin in the planting strip. The connection into the existing ditch will include a control to restrict flow to a set discharge rate. The remaining area is to be constructed from permeable material and consequently these areas can continue to drain informally as per existing conditions with limited risk of increasing runoff or flood risk to the detriment of the site and its surroundings. An indicative drainage layout is shown on **Figure 10** in **Annex C**.

5.2.10 A site survey will be undertaken to understand if any land drainage systems exist beneath the site or within the vicinity before any on-site activities commence. Consideration of land drainage is required to ensure it is not disrupted by the construction of the facility. This will allow the facility and surrounding land to continue to drain as per the existing drainage regime with the incorporation of sustainable drainage.

5.2.11 The components should be designed as shallow as possible to maintain an invert level above the local ground water level. The lifting of ground levels or implementing impermeable lining in some sections of drainage may be required to ensure this is possible. However, if existing land drainage exists the ground water level will be artificially lowered.

Further investigation is recommended to understand local ground water levels across the site to understand any impact on proposed SuDs components.

Climate Change

5.2.12 A climate change allowance for the 30 year and 100 year events have been applied based on the Environment Agency Flood Risk Assessments: climate change allowances (2022) (Ref 14). The Louth Road Block Valve Station site falls within the Louth Grimsby and Ancholme Management Catchment. It is noted a 25 year design life is proposed for the overall scheme. However, for this preliminary assessment it is assumed the civil engineering elements of the site will remain in place beyond 25 years potentially up to 75-100 years (estimated 2026 construction date) with onsite equipment being refurbished or replaced to continue operation. This would bring the expected lifetime of the development (not necessarily the operational life) beyond the year 2100 and consequently, a robust upper end climate change allowance has been adopted. This equates to a 35% uplift for a 30 year return period and 40% uplift for a 100 year return period as shown on **Table 22**.

Table 22: Louth Grimsby and Ancholme Management Catchment Peak Rainfall Allowances (values used highlighted green)

Epoch	Central Allowance	Upper End Allowance
3.3% Annual Exceedance Rainfall Event		
2050s	20%	35%
2070s	25%	**35%**
1% Annual Exceedance Rainfall Event		
2050s	20%	40%
2070s	25%	**40%**

*Use '2050s' for development with a lifetime up 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125

Design Parameters

5.2.13 Swale channels are proposed to capture and convey runoff from the proposed site. Swales have not been sized as part of this study. The channel side slope is to be 1:3 or 1:4 with a 0.5m base width and a minimum of 400mm deep. It may be possible to integrate mini swales with a reduced depth and base width considering the small area of hardstanding to be drained.

5.2.14 Discussions with local IDB have confirmed greenfield discharge rates are preferred, however other rates and outlet sizes are considered on a mitigated, rational and evidential basis. The greenfield discharge rates calculated for the site will likely result in outlet diameters smaller than 50-75mm. The blockage risk is discussed further in the Hydraulic Calculation section below. A check at a future design stage is required to confirm the outlet size required for the necessary flow control and the risk of blockage.

5.2.15 The proposed surface water attenuation is to be designed to accommodate a 1 in 100 year design storm event (1% AEP) plus a 40% climate change allowance with no surface water flooding on the site. No water will be stored above ground up to and including the 1 in 100 year event unless stored in a SuDs component.

5.2.16 Catchment descriptors and rainfall data has been downloaded from the Flood Estimation Handbook (FEH) web service (Ref 15) for use in calculations within this report.

Hydraulic Calculations

5.2.17 An InfoDrainage quick storage estimate calculation has been undertaken to understand attenuation requirements against a 1 in 100-year storm event. The calculation is based on a 0.13l/s Qbar discharge rate as calculated in **Table 20**, a 0.031ha drained area and includes a 40% climate change uplift. The default Summer Winter Cv values in InfoDrainage have been used (0.750/0.840). The results predict an attenuation storage volume of 26m^3 to 35m^3 is required (these results are estimates only and should not be used for design purposes). An average of the two values (31m^3) has been used for the purposes of this concept strategy.

5.2.18 To meet greenfield rates the flow discharge control device would likely have a small opening and be at risk of blockage. It is proposed to control the discharge rate as close to Qbar as reasonably practicable to prevent maintenance issues. This may require a discharge rate above the proposed greenfield rate but still controlled to a rate where detrimental flows are unlikely to be passed off site. Further investigation is recommended to understand an acceptable allowable discharge rate and flow control device.

Sustainable Drainage Systems and Water Quality

5.2.19 CIRIA C753 The SuDS Manual (**Ref 7**) outlays a simple index method to account for water quality in the design of SuDS. It indicates the minimum treatment indices appropriate for contributing pollution hazards for different land use classifications. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each containment) that equals or exceeds the pollution hazard index.

5.2.20 The Block Valve Station will be unmanned and will therefore have infrequent vehicle movements and no polluting activities are expected. Consequently, the site is considered to have a low pollution hazard level as per Table 26.2 in The SuDS Manual (**Ref 7**).

5.2.21 The pollution hazard indices for a low pollution hazard level and the mitigating indices relating to the selected SuDs component are listed in **Table 23**. The results indicate the use of swales will provide adequate treatment of surface water runoff. As unlined swales are proposed some informal infiltration of runoff may occur. A check of mitigating indices based on the filtration capabilities of the chosen SuDs component and underlying soil properties indicate runoff should be adequately treated before entering ground water systems.

Table 23: Pollution Hazard and Mitigation Indices

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Block Valve 3	Low	0.5	0.4	0.4
SuDs Mitigation Indices for Discharge to Surface Water				
SuDs Component		**TSS**	**Metals**	**Hydrocarbons**
Swale		0.5	0.6	0.6
Attenuation storage		0.5	0.5	0.6
Total SuDs Mitigation Index[1]		0.75	0.85	0.9
SuDs Mitigation Indices for Discharge to Ground Water				
Characteristics of material overlaying SuDs		**TSS**	**Metals**	**Hydrocarbons**
Layer of dense vegetation underlain by a soil with good contamination attenuation potential of at least 300mm in depth		0.6	0.5	0.6

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

Total SuDs Mitigation Index = mitigation index $_1$ + 0.5 (mitigation index $_2$)

Operation and Maintenance

5.2.22 An adopting party is to be agreed with the relevant the LLFA and any relevant stakeholders. It is likely the asset owner will be responsible for the maintenance of drainage components.

5.2.23 A key objective of the adoption process is to ensure that any installed SuDS can be maintained easily over the development's lifetime and beyond. Therefore, the SuDS must be designed with maintenance in mind. Proposals for SuDS must include an operation and maintenance document, setting out details on the constructed SuDs and the inspection and maintenance required. This document should be developed at full detailed design but considered throughout the design process. The Operation and Maintenance details considered at this concept design stage are noted below.

5.2.24 Maintenance activities should be conducted in accordance with industry best practice e.g. CIRIA SuDS Manual. The drainage system proposed at Block Valve 3 should be inspected at defined intervals and before and after major storm events. The proposed SuDs will require a maintenance regime including grass cutting, removal of sediment build up and clearance of the outfalls at defined intervals. The proposed SuDs features are to be shallow and allow easy access. The proposed system design life will likely meet the site design life with an adequate inspection and maintenance regime.

6 Theddlethorpe Facility Option 1

6.1 Desktop Study – Theddlethorpe Facility Option 1

Introduction

6.1.1 There are currently two options considered for the location of the Theddlethorpe Facility. This section considers Option 1:

- Theddlethorpe Facility Option 1: new facility at the former Theddlethorpe Gas Terminal (TGT) site. Demolition of the former TGT was completed in 2021 but as the site was previously an operational facility, existing security fencing and road infrastructure remain in place. The site is currently clear with a mixture of hard standing, stoned areas and pipeline stubs. Access to the site would be via an existing gate at the south west corner of the site (X 548623 Y 387508).

6.1.2 For Option 1, the onshore pipeline would enter the repurposed TGT site from the west and terminate at new facilities built next to the existing LOGGS Pipeline, which enters the site from the east. The CO_2 would enter the site via the 24'' onshore pipeline and would be routed into the 36'' LOGGS pipeline. An additional connection would be provided to allow for future carbon capture projects to connect to the Theddlethorpe Facility.

6.1.3 The Theddlethorpe Facility is required to enable the CO_2 to flow from the new 24" pipeline into the existing LOGGS (36") pipeline.

6.1.4 The Theddlethorpe Facility would comprise the following key components:

- LOGGS pipeline tie-in;
- Emergency Shutdown Valves;
- Pig receiver and launcher;
- High-integrity Pressure Protection System;
- Venting system including vent pipework, valves, and vent stack; and
- Local equipment room (LER); and
- Supporting Infrastructure.

6.1.5 The Theddlethorpe Facility would be secured by a single palisade security fence 3.2 m high.

6.1.6 The ground surface within the boundary of the Theddlethorpe Facility will be predominantly stone with a minimal number of internal tarmac/concrete access roads.

6.1.7 The Theddlethorpe Facility Option 1 is located within the existing TGT north of Mablethorpe. The site is accessed from the A1031 and is situated 670m from the coastline and the Saltfleetby/Theddlethorpe Dunes. A grazing marsh is sited between the terminal and sand dunes to the east. A Gas Transmission Terminal exists to the south. See **Annex A** for **Figure 5 – Site Overview and Topography**.

Site Topography

6.1.8 A topographical survey has not yet been undertaken for the Theddlethorpe Option 1. However, a review of available LiDAR information has been undertaken and indicates the site is relatively flat. Levels on site range between approximately 2.1 and 2.3mAoD. The site topography is shown on **Figure 5**, **Annex A**.

Local Hydrology

6.1.9 The main river watercourse that is situated closest to the site location is the Great Eau (approx. 4.8km west).

6.1.10 Ditches spurring from the existing complex boundary connect into a drain known as The Cut running west and south of the site. To the east a drain known as Crook Bank runs between Sand Hills Farm towards Bleak house also connecting with The Cut drain. The Cut ultimately discharges to the North Sea via an outfall adjacent Quebec Road Car Park. The proposed site area currently drains formally via a land drainage system serving permeable areas discharging from the site to the south and east. A closed drainage system serving hardstanding discharging to the North Sea by pumping. A plan of IDB maintained assets in the site vicinity has been provided in **Annex D**.

6.1.11 The Environment Agency flood maps indicates the site has a medium risk of flooding (between 1 in 100 year/0.1% AEP and 1 in 30 year/3.3% AEP each year) from rivers or the sea. The site has a low risk of flooding from surface water (between 1 in 1000 year /0.1% and 1 in 100 year/1% AEP each year).

Ground Conditions, Ground water and Infiltration

6.1.12 A review of the BGS Geology Viewer indicates the bedrock geology is Burnham Chalk Formation with superficial deposits classed as Tidal Flat Deposits of clay and silt.

6.1.13 The site underlying strata is classified as Principal Bedrock Aquifer and an unproductive Superficial Drift Aquifer. The site sits in a low ground water vulnerability area.

6.1.14 The BGS borehole records of holes drilled within the site boundary or immediately adjacent indicate the geological sequence from ground level is a thin layer of made ground, above silty clay before peat traces encountered at approximately 1.5m depth. Beyond this are varying layers of sandy silty clay with a bedrock of Chalk at approximately 25m depth.

6.1.15 A review of the Soil-Scapes layer on Magic maps indicates the site is situated in an area of loamy and clayey soils of coastal flats with naturally high groundwater. The drainage type is described as naturally wet. The site is not located in a Source Protection Zone.

6.1.16 Following the description of surface geology above it is not recommended to discharge surface water via infiltrating methods.

Existing Utilities

6.1.17 A desktop study was undertaken by GroundSure to gather available utility information from providers. This was submitted to AECOM as an AutoCAD DWG file covering the pipeline alignment and a buffer area either side. The DWG information indicates no utilities within the site boundary. However, the site may have on site services serving past infrastructure within the terminal.

6.1.18 Two existing surface water systems exist as listed below. The plans showing the surface water systems are included in **Annex E**.

- A closed surface water system serves hardstanding sections of the site. Water is conveyed to a central east section of site before a pumping station discharges surface water into the North Sea; and
- A surface water land drainage system serves the permeable gravel sections of the site and runoff from existing access roads. Outfalls are located on the southern and eastern site boundaries discharging into The Cut and Crook Bank.

6.2 Surface Water Drainage Strategy – Theddlethorpe Facility Option 1

Contributing Areas and Runoff Calculation

6.2.1 The existing contributing area has been measured from the scheme layout referenced above and available OS background mapping. The catchment areas will require re-calculation at a future design stage against detailed topographic surveys. The existing impermeable and impermeable areas are summarised in **Table 24**.

Table 24: Existing Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment	Existing Outfall
Total Area Site Boundary	N/A	16500	Access road and area within fenceline	N/A
Stone area	Permeable	8599	Existing Permeable Area	Land drainage/The Cut
Roads	Impermeable	2138	Existing Access Roads	Land drainage/The Cut
Concrete Pad	Impermeable	5763	Existing Drained Infrastructure Hardstanding	Sea pump
Totals		m2	ha	
Total Impermeable Area		7901	0.790	
Total Permeable Area		8599	0.860	
Area Draining to Land Drainage/ The Cut		10737	1.074	

6.2.2 The existing drainage catchment is shown in **1.**

6.2.3 The proposed contributing area has been measured from a scheme layout drawing produced by Kent Energies Ltd (drawing number: EN070008/APP/4.7). The proposed impermeable and permeable areas are summarised in **Table 25**. The proposed drainage catchments are shown in **igure 2.**

Figure 1: Existing Drainage Catchment

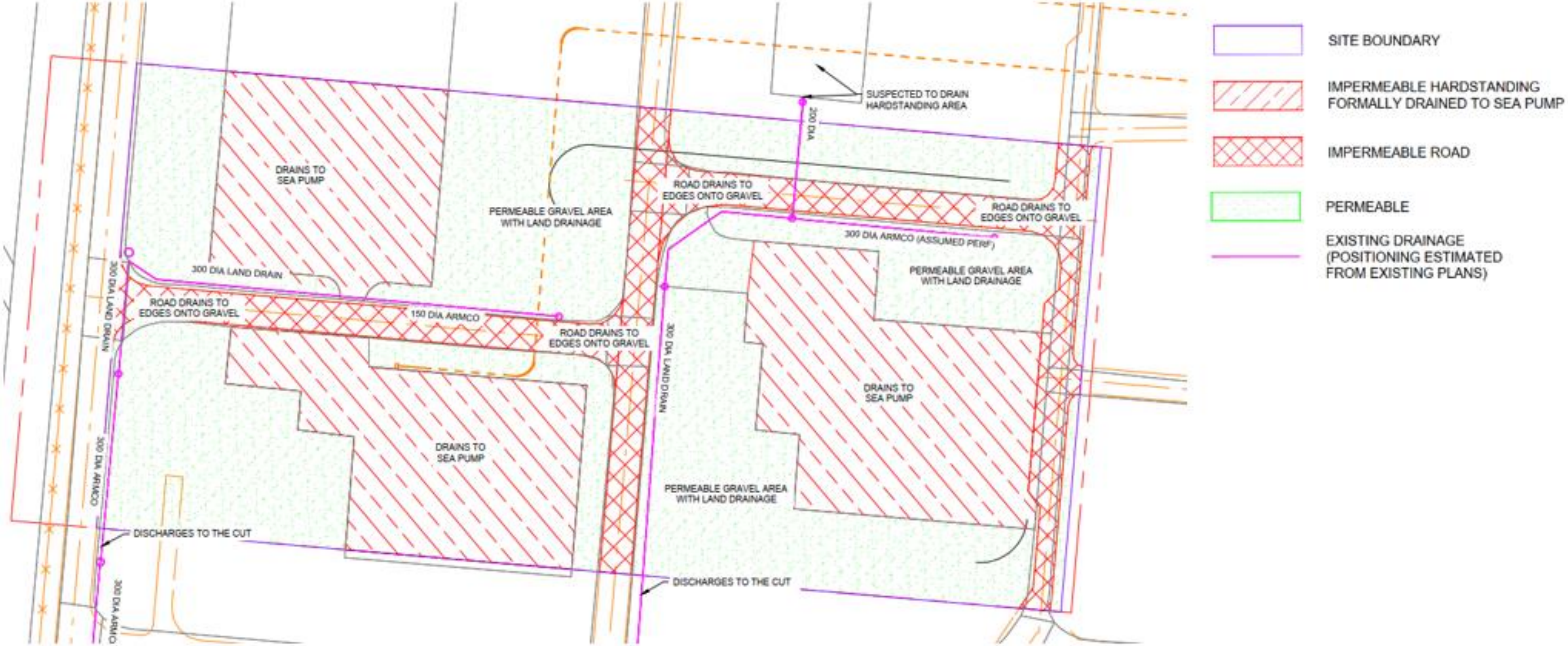
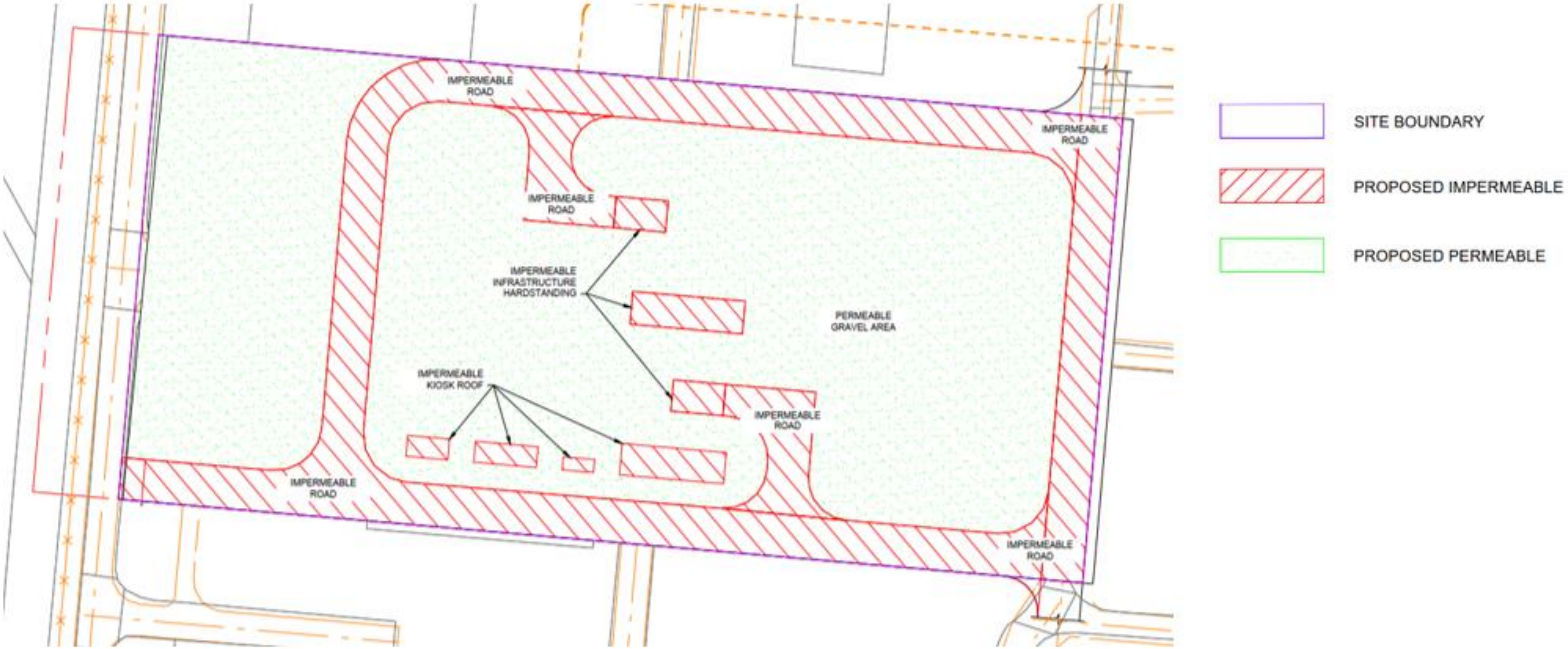


Figure 2: Proposed Drainage Catchment



6.2.4 The site will be predominantly permeable with unpaved areas to be graded to natural ground levels overlain with weed control membrane and 75mm of 20mm single size gravel.

6.2.5 Access roads are proposed to surround the site area spurring from the existing concrete access road in the terminal. Within the site boundary two splayed roads are proposed to allow access to the Pig Launch areas. Both the pig launch areas and high-integrity pressure protection system area will sit upon concrete pads. Four kiosks with flat roofs will also be situated within the site boundary sat upon concrete bases.

6.2.6 The sites will be cleared, excavated and graded to achieve the approximate required finished levels. Surfaces will be constructed to falls so that rainwater can drain to any proposed drainage system. Roads and hardstanding will have flush concrete kerbs to allow surface water run-off. Most of the site will be permeable surfacing to minimise runoff.

Table 25: Proposed Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment	Proposed Outfall
Total Area Site Boundary	N/A	16500	Access road and area within fenceline	N/A
Stone area	Permeable	11606	The majority of ground surface is to be stone aggregate	Land drainage/The Cut
Roads	Impermeable	4428	Access Roads, Access turning and parking is proposed to access the pig handling area and site	Land drainage/The Cut via attenuation and discharge control
Roofs	Impermeable	215	4 buildings are proposed including Central Control Room, Local Equipment Room, Analyser House and Metering Package Hold	Land drainage/The Cut via attenuation and discharge control
Concrete Pad	Impermeable	255	High-integrity pressure protection system and pig handling area are assumed to be sited on concrete pads or similar impermeable ground	Land drainage/The Cut via attenuation and discharge control
Totals		**m^2**	**ha**	
Total Impermeable Area		**4898**	0.490	
Total Permeable Area		**11606**	1.161	

Greenfield Runoff

6.2.7 The greenfield runoff rates for the proposed Louth Road Block Valve Station have been calculated based on the IH124 method using the HR Wallingford UK SuDS website. The greenfield runoff rates for a 50ha area were calculated using this method. A summary of the results can be seen in the calculation report found in **Annex B**, with the peak greenfield runoff rates for the total contributing area interpolated from the results shown in **Table 26**.

Table 26: Peak Greenfield Runoff Rate

Rainfall Event Frequency	Runoff (l/s/ha)	Site Contributing Area (0.640ha) GF Runoff l/s
1 in 1 Year (Approx. 99% AEP)	2.15	1.38
Qbar	2.47	1.58
1 in 30 Year (3.33% AEP)	6.05	3.87
1 in 100 Year (1% AEP)	8.79	5.63

Existing and Proposed Surface Water Runoff Rates

6.2.8 **Table 27** and **Table 28** below shows the unrestricted surface water runoff rate pre-development and post development based on the Modified Rational Method. This method estimates runoff based on the nature of the ground surface (hardstanding, vegetation etc.) and rainfall depth, duration and frequency information for the immediate area, as follows:

- C (Coefficient of impermeability) = 1.0;
- A (area) = ha; 0.644 (existing including 100% impermeable road area and 50% permeable area) and 0.490 (proposed); and
- i (Rainfall intensity based on FEH data (Ref 15)).

Table 27: Existing Peak Runoff Rates

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	68.25	38.67	23.49	15.98	12.34	8.69	4.47	2.56	1.48
5 (20%)	118.02	66.92	39.48	24.73	18.51	12.62	6.26	3.52	2.00
10 (10%)	151.89	86.47	50.88	30.84	22.77	15.31	7.51	4.20	2.36
30 (3.3%)	207.61	118.55	68.95	40.43	29.46	19.60	9.54	5.36	3.01
50 (2%)	233.10	133.52	77.52	44.98	32.65	21.68	10.59	5.99	3.38
100 (1%)	269.05	153.97	89.59	102.80	37.21	24.70	12.21	7.05	3.38
100 +20% CC	322.86	184.76	107.51	61.68	44.65	29.63	14.66	8.46	4.06
100 +40% CC	376.67	215.55	125.42	71.96	52.09	34.57	17.10	9.87	4.73

Table 28: Proposed Peak Runoff Rates

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	51.93	29.42	17.87	12.16	9.39	6.61	3.40	1.95	1.13
5 (20%)	89.80	50.92	30.04	18.82	14.08	9.60	4.76	2.68	1.52
10 (10%)	115.57	65.79	38.71	23.46	17.32	11.65	5.71	3.19	1.80
30 (3.3%)	157.96	90.20	52.46	30.77	22.41	14.92	7.26	4.08	2.29
50 (2%)	177.36	101.59	58.98	34.23	24.84	16.49	8.06	4.56	2.57
100 (1%)	204.71	117.15	68.16	78.22	28.31	18.79	9.29	5.36	2.57
100 +20% CC	245.65	140.58	81.80	46.93	33.97	22.55	11.15	6.43	3.09
100 +40% CC	286.60	164.01	95.43	54.75	39.64	26.31	13.01	7.51	3.60

Surface Water Drainage Concept

6.2.9 As existing ground conditions suggests infiltration of surface water is not recommended, following the drainage hierarchy for discharge of surface water, the next favourable point of discharge is into a surface water body.

6.2.10 It is proposed to formally drain the hardstanding sections of the site including the access roads and roof elements of kiosks via downpipes.

6.2.11 Two options could be considered for the drainage of the site. This is dependent on the proposed redevelopment of the remaining TGT and the retention of the existing drainage systems. The options include:

- **Drainage Option A SW Sea Pump in Service** – drain surface water from all proposed roads and hardstanding (located within the proposed site boundary) to the pumping station utilising the existing closed system. There is an increase in permeable area drained (located within the proposed site boundary) to the existing land drainage and consequently The Cut. Runoff from the additional permeable area may require controlling to ensure the existing system capacity is not overwhelmed and check the existing discharge rate at outfall is not exceeded; and
- **Drainage Option B SW Sea Pump Out of Service** - The proposed impermeable area (located within the proposed site boundary) runoff is to be attenuated and discharged into the existing land drainage system. The catchment area also accounts for an increase of permeable area runoff (located within the proposed site boundary) that would have previously discharged to sea via pump from former hardstanding areas. The existing and proposed catchment areas are shown in **Table 24** and **Table 25**.

6.2.12 This report considers Option B which is deemed the worst case, described in further detail below. The report only considers draining the proposed site area under development and does not consider the remaining area of the former TGT site. It should be noted the remaining part of the former TGT may be re-developed but details were not available at the time of writing.

6.2.13 Hardstanding areas are proposed to drain into a filter drainage conveyance system around the site perimeter. The system will collect runoff and convey flows west to outfall in an attenuation basin. The basin will have a piped outlet connecting into the existing drainage system with a flow control device controlling the discharge rate. An indicative drainage layout is shown on **Figure 11** in **Annex C**. Further information on the existing system is required to understand its depth to enable a connection with the proposed drainage and its drainage capacity.

6.2.14 The majority of runoff from permeable sections of the site is to continue to be drained as per the existing drainage regime. Review of existing site plans indicates the site is served by a land drainage system for permeable sections of the site which discharges collected flows to The Cut along the southern perimeter or Crook Bank channel to the east. The proposed site location is likely to discharge into the system outfalling into The Cut. Further investigation is required to understand existing land drainage catchments and if adjustment of the system is required to account for the proposed site.

6.2.15 **Table 29** outlines the contributing drainage catchment area. The removal of large concrete infrastructure pads that previously drained to the Sea Pump increases the permeable area discharging to The Cut. The impermeable area discharging to the cut is also shown to increase compared to the existing site. It is proposed to formally drain and control the discharge rate from the total proposed impermeable area and the additional permeable area. The former permeable area is to continue to drain as per the existing drainage regime. The large spacings between existing land drainage serving the former permeable area slows runoff and allows some storage within the gravel voids.

6.2.16 The additional permeable area and total proposed impermeable area will be captured and discharged into an attenuation basin as described above. This method will allow a controlled discharge of runoff into the existing drainage system at greenfield discharge rates.

Table 29: Option B – Catchment Makeup and Areas Contributing to the Proposed Drainage System

<table>
<tr><th>Scenario</th><th colspan="5">Option B</th></tr>
<tr><td rowspan="4">Existing</td><td>Catchment Makeup</td><td>Surface Type</td><td>Area (m²)</td><td>Assumed Positively Drained Area (m²)</td><td>ha</td></tr>
<tr><td rowspan="2">Total Area discharging via The Cut</td><td>Permeable Gravel</td><td>8599</td><td>4300</td><td>0.430</td></tr>
<tr><td>Impermeable Roads</td><td>2138</td><td>2138</td><td>0.214</td></tr>
<tr><td colspan="3">Total Area to the Cut</td><td>6438</td><td>0.644</td></tr>
<tr><td rowspan="4">Proposed</td><td>Catchment Makeup</td><td>Surface Type</td><td>Area (m²)</td><td>Assumed Positively Drained Area (m²)</td><td>ha</td></tr>
<tr><td rowspan="2">Total Area discharging via The Cut</td><td>Permeable Gravel</td><td>11606</td><td>5803</td><td>0.580</td></tr>
<tr><td>Impermeable Roads/Roofs/ Concrete</td><td>4898</td><td>4898</td><td>0.490</td></tr>
<tr><td colspan="3">Total Area to the Cut</td><td>10701</td><td>1.070</td></tr>
<tr><td colspan="3">Catchment Areas to Drainage System</td><td>Area (m²)</td><td>Assumed Positively Drained Area (m²)</td><td>ha</td></tr>
</table>

Scenario	Option B		
Permeable area contributing to proposed drainage system	3007	1504	0.15
Impermeable area contributing to proposed drainage system	4898	4898	0.49
Total Area Contributing to the Proposed Drainage System	7905	6402	0.64

6.2.17 The components should be designed as shallow as possible to maintain an invert level above the local ground water level. The lifting of ground levels or implementing impermeable lining in some sections of drainage may be required to ensure this is possible. However, if existing land drainage exists the ground water level may be artificially lowered. Further investigation is recommended to understand local ground water levels across the site to understand any impact on proposed SuDs components.

Climate Change

6.2.18 A climate change allowance for the 30 year and 100 year events have been applied based on the Environment Agency Flood Risk Assessments: climate change allowances (2022) (Ref 14). The Theddlethorpe Facility Option 1 site falls within the Witham Management Catchment. It is noted a 25 year design life is proposed for the overall scheme. However, for this preliminary assessment it is assumed the civil engineering elements of the site will remain in place beyond 25 years potentially up to 75-100 years (estimated 2026 construction date) with onsite equipment being refurbished or replaced to continue operation. This would bring the expected lifetime of the development (not necessarily the operational life) beyond the year 2100 and consequently, a robust upper end climate change allowance has been adopted. This equates to a 35% uplift for a 30 year return period and 40% uplift for a 100 year return period as shown on **Table 30**.

Table 30: Witham Management Catchment Peak Rainfall Allowances (values used highlighted green)

Epoch	Central Allowance	Upper End Allowance
3.3% Annual Exceedance Rainfall Event		
2050s	20%	35%
2070s	25%	**35%**
1% Annual Exceedance Rainfall Event		
2050s	20%	40%
2070s	25%	**40%**

*Use '2050s' for development with a lifetime up 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125

Design Parameters

6.2.19 The surface water discharge rate is to ideally be controlled to Qbar for events between Qbar (approximately 1 in 2 year event) and 1 in 100 year event.

6.2.20 Discussions with local IDB have confirmed greenfield discharge rates are preferred, however other rates and outlet sizes are considered on a mitigated, rational and evidential basis. The greenfield discharge rates calculated for the site will likely result in outlet diameters smaller than 50-75mm. The blockage risk is discussed further in the Hydraulic Calculation section below. A check at a future design stage is required to confirm the outlet size required for the necessary flow control and the risk of blockage.

6.2.21 The proposed surface water attenuation is to be designed to accommodate a 1 in 100 year design storm event (1% AEP) plus a 40% climate change allowance with no surface water flooding on the site. No water will be stored above ground up to and including the 1 in 100 year event unless stored in a SuDs component.

6.2.22 Catchment descriptors and rainfall data has been downloaded from the Flood Estimation Handbook (FEH) web service (Ref 15) for use in calculations within this report.

Hydraulic Calculations

6.2.23 An InfoDrainage quick storage estimate calculation has been undertaken to understand attenuation requirements against a 1 in 100-year storm event. The calculation is based on a 1.58l/s Qbar discharge rate as calculated in **Table 25**, a 0.640ha drained area and includes a 40% climate change uplift. The default Summer Winter Cv values in InfoDrainage have been used (0.750/0.840). The results predict an attenuation storage volume of 611m^3 to 749m^3 is required (these results are estimates only and should not be used for design purposes). An average of the two values (680m^3) has been used for the purposes of this concept strategy.

6.2.24 To meet greenfield rates the flow discharge control device will likely have a small opening and be at risk of blockage. To minimise the risk of blockage it is proposed to use a Hydrobrake device within a chamber for flow control. Preliminary calculations using the Hydro-International online design tool predicts the device will have a 65mm diameter outlet with 0.6m head and 1.58l/s discharge rate. Whilst this diameter does not meet a nationally recognised 75mm minimum outlet size, the risk of blockage for the proposed outlet arrangement is deemed low based on existing site conditions.

Sustainable Drainage Systems and Water Quality

6.2.25 CIRIA C753 The SuDS Manual (**Ref 7**) outlays a simple index method to account for water quality in the design of SuDS. It indicates the minimum treatment indices appropriate for contributing pollution hazards for different land use classifications. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each containment) that equals or exceeds the pollution hazard index.

6.2.26 The proposed development will be unmanned and will therefore have infrequent vehicle movements and no polluting activities are expected. Consequently, the site is considered to have a low pollution hazard level as per Table 26.2 in The SuDS Manual (**Ref 7**).

6.2.27 The pollution hazard indices for a low pollution hazard level and the mitigating indices relating to the selected SuDs component are listed in **Table 31**. The results indicate the use of filter drainage combined with a detention basin will provide adequate treatment of surface water runoff. As lined filter drains are proposed infiltration of runoff will not occur, consequently the hazard to groundwater has not been assessed for this component. The detention basin could be unlined, and some informal infiltration of runoff may occur. A check of mitigating indices based on the filtration capabilities of a detention basin SuDs component and underlying soil properties indicate runoff should be adequately treated before entering ground water systems.

Table 31: Pollution Hazard and Mitigation Indices

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Theddlethorpe Facility	Low	0.5	0.4	0.4
SuDs Mitigation Indices for Discharge to Surface Water				
SuDs Component		**TSS**	**Metals**	**Hydrocarbons**
Filter Drain		0.4	0.4	0.4
Detention Basin		0.5	0.5	0.6
Total SuDs Mitigation Index[1]		0.65	0.65	0.7
SuDs Mitigation Indices for Discharge to Ground Water				
Characteristics of material overlaying SuDs		**TSS**	**Metals**	**Hydrocarbons**
Layer of dense vegetation underlain by a soil with good contamination attenuation potential of at least 300mm in depth		0.6	0.4	0.6

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

Total SuDs Mitigation Index = mitigation index $_1$ + 0.5 (mitigation index $_2$)

Operation and Maintenance

6.2.28 An adopting party is to be agreed with the relevant the LLFA and any relevant stakeholders. It is likely the asset owner will be responsible for the maintenance of drainage components.

6.2.29 A key objective of the adoption process is to ensure that any installed SuDS can be maintained easily over the development's lifetime and beyond. Therefore, the SuDS must be designed with maintenance in mind. Proposals for SuDS must include an operation and maintenance document, setting out details on the constructed SuDs and the inspection and maintenance required. This document should be developed at full detailed design but considered throughout the design process. The Operation and Maintenance details considered at this concept design stage are noted below.

6.2.30 Maintenance activities should be conducted in accordance with industry best practice e.g. CIRIA SuDS Manual. The drainage system proposed at Theddlethorpe Facility should be inspected at defined intervals and before and after major storm events. The proposed SuDs will require a maintenance regime including grass cutting, removal of sediment build up at and clearance of the outfalls at defined intervals. The proposed SuDs features are proposed to be shallow and allow easy access. The filter drains and permeable gravel sections of the site are deemed to have a low risk of sediment build up. The proposed system design life will likely meet the site design life with an adequate inspection and maintenance regime.

7 Theddlethorpe Facility Option 2

7.1 Desktop Study – Theddlethorpe Facility Option 2

Introduction

7.1.1 There are currently two options considered for the location of the Theddlethorpe Facility. This section considers Option 2:

- **Theddlethorpe Facility Option 2**: new facility to the west of the former TGT site, located on arable land directly west of The Cut (an ordinary watercourse). This facility would be accessed from the north off the A1031 Mablethorpe Road. (X 548175 Y 387586).

7.1.2 For Option 2 the existing LOGGS pipeline would be extended to the new site to the west using sections of 36” pipeline.

7.1.3 The Theddlethorpe Facility is required to enable the CO_2 to flow from the new 24” pipeline into the existing LOGGS (36”) pipeline.

7.1.4 An indicative layout of the Theddlethorpe Facility would comprise the following key components:

- LOGGS pipeline tie-in;
- Emergency Shutdown Valves;
- Pig receiver and launcher;
- High-integrity Pressure Protection System;
- Venting system including vent pipework, valves, and vent stack; and
- Local equipment room (LER); and
- Supporting Infrastructure.

7.1.5 The Theddlethorpe Facility would be secured by a single palisade security fence 3.2 m high.

7.1.6 The ground surface within the boundary of the Theddlethorpe Facility will be predominantly stone with a minimal number of internal tarmac/concrete access roads.

7.1.7 The Theddlethorpe Facility Option 2 is located within an agricultural field west of the Viking Gas Terminal. The field is bounded by the A1031, The Cut drainage channel and unnamed drainage channels to the south. See **Annex A** for **Figure 6 – Site Overview and Topography**.

Site Topography

7.1.8 A topographical survey has not yet been undertaken for the Theddlethorpe Option 2. However, a review of available LiDAR information has been undertaken and indicates the site is relatively flat. Levels on site sit at approximately 1.5 mAoD. The site topography is shown on **Figure 6, Annex A**.

Local Hydrology

7.1.9 The main river watercourse that is situated closest to the site location is the Great Eau (approx. 2.5km west).

7.1.10 Drainage ditches follow most of the field boundary including The Cut running to the north and east of the site. The current site is likely to drain surface water informally or via land drainage into the cut or other field boundary drainage ditches. A plan of IDB maintained assets in the site vicinity has been provided in **Annex D**.

7.1.11 The Environment Agency flood maps indicates the site has a medium risk of flooding (between 1 in 100 year/0.1% AEP and 1 in 30 year/3.3% AEP each year) from rivers or the sea. The site has a very low risk of flooding from surface water (1 in 1000 year /0.1% each year).

Ground Conditions, Ground water and Infiltration

7.1.12 A review of the BGS Geology Viewer indicates the bedrock geology is Burnham Chalk Formation with superficial deposits classed as Tidal Flat Deposits of clay and silt.

7.1.13 The site underlaying strata is classified as Principal Bedrock Aquifer and an unproductive Superficial Drift Aquifer. The site sits in a low ground water vulnerability area.

7.1.14 The BGS borehole records of holes drilled within the field boundary indicates the geological sequence from ground level is varying descriptions of silty clay. A bedrock of Chalk was hit at approximately 129 feet (39m) below ground level.

7.1.15 A review of the Soil-Scapes layer on Magic maps indicates the site is situated in an area of loamy and clayey soils of coastal flats with naturally high groundwater. The drainage type is described as naturally wet. The site is not located in a Source Protection Zone.

7.1.16 Following the description of surface geology above it is not recommended to discharge surface water via infiltrating methods.

Existing Utilities

7.1.17 A desktop study was undertaken by GroundSure to gather available utility information from providers. This was submitted to AECOM as an AutoCAD DWG file covering the pipeline alignment and a buffer area either side. The DWG information indicates there is no known utilities within the site boundary.

7.2 Surface Water Drainage Strategy – Theddlethorpe Facility Option 2

Contributing Areas and Runoff Calculation

7.2.1 The contributing area has been measured from a scheme layout drawing produced by Kent Energies Ltd (drawing number: EN070008/APP/4.8). The proposed impermeable and permeable areas are summarised in

7.2.2 **Table** 32.

7.2.3 The site will be predominantly permeable with unpaved areas to be graded to natural ground levels overlain with weed control membrane and 75 mm of 20mm single size gravel.

7.2.4 A facility access road spurring from the Mablethorpe Road will be constructed to enable construction access to the site. The access track is to be retained post construction and will be constructed with an impermeable surface. For this study it is assumed the track will be 8m wide, to be confirmed at a future design stage.

7.2.5 Access roads are proposed to surround the site area. Within the fence line boundary two splayed roads are proposed to allow access to the Pig Launch areas. Both the pig launch areas and high-integrity pressure protection system area will sit upon concrete pads. Four kiosks with flat roofs will also be situated within the site boundary sat upon concrete bases. A 10m planting strip is proposed around the Theddlethorpe Facility Option 2 perimeter which will continue to drain naturally and is not included in the drainage catchment area.

7.2.6 The sites will be cleared, excavated and graded to achieve the approximate required finished levels. Surfaces will be constructed to falls so that rainwater can drain to any proposed drainage system. Roads and hardstanding will have flush concrete kerbs to allow surface water run-off. Most of the site will be permeable surfacing to minimise runoff. A cut-off drainage channel maybe required at the site entrance gate to control runoff onto site.

Table 32: Drainage Catchment Area Take-Off

Ref	Surface Type	Area (m^2)	Comment
Total Site Area	N/A	20345	Access road and area within fenceline
Stone area	Permeable	12385	The majority of ground surface within the fenceline is to be stone aggregate within the access roads
Roads Inside Theddlethorpe Facility Option 2 Fenceline	Impermeable	3310	Access turning and parking is proposed to access the pig handling area and site
Road Access from Mablethorpe Road	Impermeable	4180	8m Wide access between the Theddlethorpe Facility Option 2 and Mablethorpe Road
Roofs	Impermeable	215	4 buildings are proposed including Central Control Room, Local Equipment Room, Analyser House and Metering Package Hold
Concrete Pad	Impermeable	255	High-integrity pressure protection system and pig handling area are assumed to be sited on concrete pads or similar impermeable ground
Totals		**m^2**	**ha**
Total Impermeable Area		7960	0.796
Total Permeable Area		12385	1.238
Total Contributing area (impermeable area)		7960	0.796

Greenfield Runoff

7.2.7 The greenfield runoff rates for the proposed Theddlethorpe Facility have been calculated based on the IH124 method using the HR Wallingford UK SuDS website. The greenfield runoff rates for a 50ha area were calculated using this method. A summary of the results can be seen in the calculation report found in **Annex B**, with the peak greenfield runoff rates for the total contributing area interpolated from the results shown in **Table 33**.

Table 33: Peak Greenfield Runoff Rate

Rainfall Event Frequency	Runoff (l/s/ha)	Site Contributing Area (0.378ha, discounting the access road) GF Runoff l/s	Site Contributing Area (0.796ha, including the access road) GF Runoff l/s

1 in 1 Year (Approx. 99% AEP)	2.15	0.81	1.71
Qbar	2.47	0.93	1.97
1 in 30 Year (3.33% AEP)	6.05	2.29	4.82
1 in 100 Year (1% AEP)	8.79	3.32	7

Proposed Surface Water Runoff Rates

7.2.8 **Table 34** below shows the unrestricted surface water runoff rate post-development based on the Modified Rational Method. This method estimates runoff based on the nature of the ground surface (hardstanding, vegetation etc.) and rainfall depth, duration and frequency information for the immediate area, as follows:

- C (Coefficient of impermeability) = 1.0;
- A (area) = ha; 0.796; and
- i (Rainfall intensity based on FEH data (Ref 15)).

Table 34: Proposed Peak Runoff Rate

Rainfall Event Frequency	Duration								
	15 min	30 min	1 h	2 h	3 h	5 h	12 h	24 h	48 h
2 (50%)	95.60	52.22	32.00	20.64	15.66	10.90	5.54	3.17	1.83
5 (20%)	165.52	89.84	52.42	31.64	23.36	15.77	7.76	4.36	2.47
10 (10%)	213.68	116.57	66.90	39.29	28.68	19.11	9.31	5.19	2.92
30 (3.3%)	293.16	159.73	89.73	51.31	37.03	24.46	11.84	6.63	3.72
50 (2%)	330.16	180.08	100.53	56.99	41.02	27.04	13.14	7.42	4.18
100 (1%)	380.62	208.81	115.78	130.14	46.73	30.82	15.16	8.73	4.18
100 +20% CC	456.74	250.57	138.93	78.08	56.07	36.98	18.19	10.47	5.02
100 +40% CC	532.86	292.33	162.09	91.10	65.42	43.14	21.22	12.22	5.86

Surface Water Drainage Concept

7.2.9 As existing ground conditions suggests infiltration of surface water is not recommended, following the drainage hierarchy for discharge of surface water, the next favourable point of discharge is into a surface water body. An existing drainage channel (The Cut) runs to the east of the site and likely receives runoff from the existing field.

7.2.10 It is proposed to formally drain the hardstanding sections of the site including the access road and roof elements of kiosks via downpipes. There will be no change to the permanent land use or drained area within permeable gravel sections so the existing drainage principles will be maintained. Consequently, no formal drainage is proposed and gravel sections have not been considered as part of the contributing area.

7.2.11 Hardstanding areas (infrastructure foundations) are proposed to drain into proposed filter drains. The filter drains are to be installed with impermeable membrane to prevent the collection of ground water. A solid pipe branch will collect flows and convey them north and south for connection into a swale.

7.2.12 Swale channels aligned adjacent to the proposed access roads will collect surface water runoff and convey flow for connection into the detention basin. The basin will have a piped outlet connecting into The Cut with a flow control device controlling the discharge rate. Any restricted volume will be attenuated within the detention basin. An indicative drainage layout is shown on **Figure 12** in **Annex C**.

7.2.13 A site survey will be undertaken to understand if any land drainage systems exist beneath the site or within the vicinity before any on-site activities commence. Consideration of land drainage is required to ensure it is not disrupted by the construction of the facility. This will allow the facility and surrounding land to continue to drain as per the existing drainage regime with the incorporation of sustainable drainage.

7.2.14 A review of available water level records for The Cut should be undertaken to understand any impact on the proposed surface water system and local hydrology.

7.2.15 The components should be designed as shallow as possible to maintain an invert level above the local ground water level. The lifting of ground levels or implementing impermeable lining in some sections of drainage may be required to ensure this is possible. However, if existing land drainage exists the ground water level will be artificially lowered. Further investigation is recommended to understand local ground water levels across the site to understand any impact on proposed SuDs components.

Climate Change

7.2.16 A climate change allowance for the 30 year and 100 year events have been applied based on the Environment Agency Flood Risk Assessments: climate change allowances (2022) (Ref 14). The Theddlethorpe Facility Option 2 site falls within the Witham Management Catchment. It is noted a 25 year design life is proposed for the overall scheme. However, for this preliminary assessment it is assumed the civil engineering elements of the site will remain in place beyond 25 years potentially up to 75-100 years (estimated 2026 construction date) with onsite equipment being refurbished or replaced to continue operation. This would bring the expected lifetime of the development (not necessarily the operational life) beyond the year 2100 and consequently, a robust upper end climate change allowance has been adopted. This equates to a 35% uplift for a 30 year return period and 40% uplift for a 100 year return period as shown on Table 35.

Table 35: Witham Management Catchment Peak Rainfall Allowances (values used highlighted green)

Epoch	Central Allowance	Upper End Allowance
3.3% Annual Exceedance Rainfall Event		
2050s	20%	35%
2070s	25%	**35%**
1% Annual Exceedance Rainfall Event		
2050s	20%	40%
2070s	25%	**40%**

*Use '2050s' for development with a lifetime up 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125

Design Parameters

7.2.17 The surface water discharge rate is to ideally be controlled to Qbar for events between Qbar (approximately 1 in 2 year event) and 1 in 100 year event.

7.2.18 Discussions with local IDB have confirmed greenfield discharge rates are preferred, however other rates and outlet sizes are considered on a mitigated, rational and evidential basis. The greenfield discharge rates calculated for the site will likely result in outlet diameters smaller than 50-75mm. The blockage risk is discussed further in the Hydraulic Calculation section below. A check at a future design stage is required to confirm the outlet size required for the necessary flow control and the risk of blockage.

7.2.19 The proposed surface water attenuation is to be designed to accommodate a 1 in 100 year design storm event (1% AEP) plus a 40% climate change allowance with no surface water flooding on the site. No water will be stored above ground up to and including the 1 in 100 year event unless stored in a SuDs component.

7.2.20 Catchment descriptors and rainfall data has been downloaded from the Flood Estimation Handbook (FEH) web service (Ref 15) for use in calculations within this report.

Hydraulic Calculations

7.2.21 An InfoDrainage quick storage estimate calculation has been undertaken to understand attenuation requirements against a 1 in 100-year storm event. The calculation is based on a 0.93l/s Qbar discharge rate, a 0.378ha drained area (excluding the access road served by swale conveyance and attenuation) and includes a 40% climate change uplift. The default Summer Winter Cv values in InfoDrainage have been used (0.750/0.840). The results predict an attenuation storage volume of 361m^3 to 442m^3 is required (these results are estimates only and should not be used for design purposes). An average of the two values (401m^3) has been used for the purposes of this concept strategy.

7.2.22 An orifice flow control outlet diameter to restrict flow to a 0.93l/s outflow will likely be under 50-75mm and could be at risk of blockage without protection. To prevent blockage a granular fill could be placed around the outlet to filter out sediment and debris and also prevent vegetation growth. Alternatively, a Hydrobrake arrangement could be used. However, preliminary calculations using Hydro-International's online Hydrobrake design tool suggests the outlet diameter will be approximately 52mm with a 0.5m head and a Qbar discharge rate. The risk of blockage could be deemed reduced by using a Hydrobrake with the device being contained within a chamber with a sump in comparison with a standard orifice.

7.2.23 It is proposed to control the discharge rate as close to Qbar as reasonably practicable to prevent maintenance issues. This may require a discharge rate above the proposed greenfield rate but still controlled to a rate where detrimental flows are unlikely to be passed off site. Further investigation at a future design stage is recommended to understand an acceptable allowable discharge rate and flow control device.

7.2.24 The swales serving the access track from Mablethorpe Road (A1031) are proposed to act as attenuation storage with a restricted flow discharge. The available storage volume will likely accommodate the predicted water volume for a 1 in 100 year storm event + climate change as shown in **Table 36**, but may require some minor shallow storage/ground lowering in land adjacent (to be confirmed at a future design stage). The corresponding outlet size to control the flow at a greenfield rate would likely be below 50-75mm and be at risk of blockage. Alternatively, it is proposed to use permeable check dams along the swales length and end to slow the flow rate entering the watercourse. A Darcy's Law calculation was undertaken to understand horizontal flow through a granular check dam. A 0.2l/s flow is predicted for a 400 deep, 0.5m base width, 1:4 Side slope channel with a 1 in 250 fall. This method of flow control should be investigated in further detail at a future design stage. It is proposed to reduce the discharge rate as close to Qbar as reasonably practicable to prevent maintenance issues. As the local ground levels are flat one long continuous channel would result in a deep and wide channel to maintain a gradient capable of conveyance. Alternatively, the channel should be split into segments to keep the channel depth shallow and discharge at multiple small channel outlets. If it is possible to remove the need for an impermeable road access surface, it is recommended to use a permeable surface and remove the need for proposed formal drainage.

Table 36: Proposed Surface Water Access Road Swales 1 in 100 Year Event + CC

Post construction Runoff. 15 min 100 yr +40%cc Rational Method (l/s)	Greenfield Runoff Bridge Area North /Qbar Restriction Rate	Contributing Area (ha)	Restriction Rate via Granular Check Dam (l/s)	Storage Vol Req (m3)
92	0.34	0.137	0.20	159

Sustainable Drainage Systems and Water Quality

7.2.25 CIRIA C753 The SuDS Manual (**Ref 7**) outlays a simple index method to account for water quality in the design of SuDS. It indicates the minimum treatment indices appropriate for contributing pollution hazards for different land use classifications. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each containment) that equals or exceeds the pollution hazard index.

7.2.26 The proposed development will be unmanned and will therefore have infrequent vehicle movements and no polluting activities are expected. Consequently, the site is considered to have a low pollution hazard level as per **Table 37** in The SuDS Manual (**Ref 7**).

7.2.27 The pollution hazard indices for a low pollution hazard level and the mitigating indices relating to the selected SuDs component are listed in **Table 37**. The results indicate the use of swales will provide adequate treatment of surface water runoff. As unlined swales are proposed some informal infiltration of runoff may occur. A check of mitigating indices based on the filtration capabilities of the chosen SuDs component and underlying soil properties indicate runoff should be adequately treated before entering ground water systems.

Table 37: Pollution Hazard and Mitigation Indices

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Theddlethorpe Option 2	Low	0.5	0.4	0.4
SuDs Mitigation Indices for Discharge to Surface Water				
SuDs Component		**TSS**	**Metals**	**Hydrocarbons**
Swale		0.5	0.6	0.6
Attenuation storage		0.5	0.5	0.6
Total SuDs Mitigation - Index Access Roads[1]		**0.75**	**0.85**	**0.9**
Filter Drain		0.4	0.4	0.4
Attenuation storage		0.5	0.5	0.6

Pollution Hazard Indices				
Location	**Pollution Hazard Level**	**Total Suspended Solids (TSS)**	**Metals**	**Hydrocarbons**
Total SuDs Mitigation Index - Theddlethorpe Option 2[1]		**0.65**	**0.65**	**0.7**
SuDs Mitigation Indices for Discharge to Ground Water				
Characteristics of material overlaying SuDs		**TSS**	**Metals**	**Hydrocarbons**
Layer of dense vegetation underlain by a soil with good contamination attenuation potential of at least 300mm in depth		**0.6**	**0.5**	**0.6**

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

Total SuDs Mitigation Index = mitigation index $_1$ + 0.5 (mitigation index $_2$)

Operation and Maintenance

7.2.28 An adopting party is to be agreed with the relevant the LLFA and any relevant stakeholders. It is likely the asset owner will be responsible for the maintenance of drainage components.

7.2.29 A key objective of the adoption process is to ensure that any installed SuDS can be maintained easily over the development's lifetime and beyond. Therefore, the SuDS must be designed with maintenance in mind. Proposals for SuDS must include an operation and maintenance document, setting out details on the constructed SuDs and the inspection and maintenance required. This document should be developed at full detailed design but considered throughout the design process. The Operation and Maintenance details considered at this concept design stage are noted below.

7.2.30 Maintenance activities should be conducted in accordance with industry best practice e.g. CIRIA SuDS Manual. The drainage system proposed at Theddlethorpe Facility should be inspected at defined intervals and before and after major storm events. The proposed SuDs will require a maintenance regime including grass cutting, removal of sediment build up at and clearance of the outfalls at defined intervals. The proposed SuDs features are proposed to be shallow and allow easy access. The filter drains and permeable gravel sections of the site are deemed to have a low risk of sediment build up. The proposed system design life will likely meet the site design life with an adequate inspection and maintenance regime.

8 Drainage Strategy Conclusion

Immingham Facility

8.1.1 Infiltration of surface water runoff is not deemed achievable based on a review of available geological information. Consequently, runoff from the site will be discharged to the re-aligned existing drainage channel via swales, filter drains and a detention basin. Discharge rates from attenuation areas should be restricted to greenfield runoff rates where possible.

8.1.2 Further investigation is recommended to understand the impact of ground water on the site and proposed drainage systems to ensure SuDs components are not detrimentally impacted. Testing could include ground water level monitoring.

8.1.3 A review of any available water level records for existing IDB drainage channels should be undertaken to understand any impact on the proposed surface water system and local hydrology.

Washingdales Lane Block Valve Station

8.1.4 Infiltration of surface water runoff could be possible based on a review of available geological information. Consequently, runoff from the site is proposed to be discharged via an infiltration trench.

8.1.5 The suitability of infiltration should be confirmed through site testing (trial holes, infiltration tests to BRE365 and ground water level monitoring). Site investigation is recommended to understand the site infiltration rate, assess ground conditions/ inspect for contamination and check any potential adverse ground water that would impact upon infiltration SuDs components.

8.1.6 Further investigation is recommended to understand the impact of ground water on the site and proposed drainage systems to ensure SuDs components are not detrimentally impacted. Testing could include ground water level monitoring.

8.1.7 A site survey would be undertaken prior to the commencement of any on-site works to understand if any land drainage systems exist beneath the site or within the vicinity. Consideration of land drainage is required to ensure it is not disrupted by the construction of the Washingdales Lane Block Valve Station.

Thoroughfare Block Valve Station

8.1.8 Infiltration of surface water runoff is not deemed achievable based on a review of available geological information. Consequently, runoff from the site will be discharged to field edge drainage channels. Discharge rates from detention basin areas should be restricted to greenfield runoff rates where possible.

8.1.9 Further investigation is recommended to understand the impact of ground water on the site and proposed drainage systems to ensure SuDs components are not detrimentally impacted. Testing could include ground water level monitoring.

8.1.10 A site survey would be undertaken prior to the commencement of any on-site works to understand if any land drainage systems exist beneath the site or within the vicinity. Consideration of land drainage is required to ensure it is not disrupted by the construction of Thoroughfare Block Valve Station.

Louth Road Block Valve Station

8.1.11 Infiltration of surface water runoff is not deemed achievable based on a review of available geological information. Consequently, runoff from the site will be discharged to field edge drainage channels. Discharge rates from detention basin areas should be restricted to greenfield runoff rates where possible.

8.1.12 Further investigation is recommended to understand the impact of ground water on the site and proposed drainage systems to ensure SuDs components are not detrimentally impacted. Testing could include ground water level monitoring.

8.1.13 A site survey would be undertaken prior to the commencement of any on-site works to understand if any land drainage systems exist beneath the site or within the vicinity. Consideration of land drainage is required to ensure it is not disrupted by the construction of the Louth Road Block Valve Station.

Theddlethorpe Facility Option 1

8.1.14 Infiltration of surface water runoff is deemed to be unachievable based on a review of available geological information. Two options have been considered for the control and discharge of surface water from site including:

- **Drainage Option A SW Sea Pump in Service** – drain surface water from all proposed roads and hardstanding to the pumping station utilising the existing closed system. There is an increase in permeable area drained to the existing land drainage system and consequently The Cut. The additional permeable area may require controlling to ensure the existing system capacity is not overwhelmed and check the existing discharge rate at outfall is not exceeded.
- **Drainage Option B SW Sea Pump Out of Service** - The proposed impermeable area runoff is to be attenuated to a greenfield runoff rate and discharged into the existing land drainage system. The catchment area also accounts for an increase of permeable area runoff that would have discharged to sea via pump from previously hardstanding areas.

8.1.15 A site survey would be undertaken prior to the commencement of any on-site works to understand the existing drainage systems on site to allow either of the options above to be taken forward. This information is also required to understand its depth to enable a connection with the proposed drainage and its drainage capacity.

8.1.16 Future site development proposals for the wider TGT should be confirmed. This will allow a combined drainage design for the whole site to be developed including the Theddlethorpe Facility Option 1 considered in this report.

8.1.17 Further investigation is recommended to understand the impact of ground water on the site and proposed drainage systems to ensure SuDs components are not detrimentally impacted. Testing could include ground water level monitoring.

Theddlethorpe Facility Option 2

8.1.18 Infiltration of surface water runoff is deemed to be unachievable based on a review of available geological information. Consequently, runoff from the site will be discharged to an existing drainage channel known as The Cut via swales, filter drains and a detention basin. Discharge rates from detention basin areas should be restricted to greenfield runoff rates where possible.

8.1.19 Further investigation is recommended to understand the impact of ground water on the site and proposed drainage systems to ensure SuDs components are not detrimentally impacted. Testing could include ground water level monitoring.

8.1.20 A site survey would be undertaken prior to the commencement of any on-site works to understand if any land drainage systems exist beneath the site or within the vicinity. Consideration of land drainage is required to ensure it is not disrupted by the construction of the facility.

8.1.21 A review of available water level records for The Cut should be undertaken to understand any impact on the proposed surface water system and local hydrology.

Flow Discharge Rates

8.1.22 Discharge rates from the sites should be restricted to greenfield runoff rates. Where greenfield discharge rates are low and create a small flow control outlet diameter under 50-75mm there could be an unacceptable risk of blockage. Consequently, further investigation and liaison with the LLFA and IDB is required at a future design stage to determine acceptable flow controls and either protect small outlets or potentially increase discharge rates higher than greenfield but still controlled to a rate where detrimental flows are unlikely to be passed off site.

Assumptions and Limitations

The drainage design within this report has been developed against available site information and design details at the time of writing to provide a surface water drainage strategy. As part of further Front-End Engineering Design, this drainage design will be further developed and concluded in tandem with the overall scheme design development. Additional work will include a review of findings from investigations recommended in this report.

9 References

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