ANNEX A3

Process Basis of Design Report

This document has been written in line with the requirements of the RAPID gate two guidance and to comply with the regulatory process pursuant to Severn Trent Water's statutory duties. The information presented relates to material or data which is still in the course of completion. Should the solution presented in this document be taken forward, Severn Trent Water will be subject to the statutory duties pursuant to the necessary consenting process, including environmental assessment and consultation as required. This document should be read with those duties in mind.

STSources SRO
Severn Trent Water

Severn Trent Water

SEVERN TRENT SOURCES STRATEGIC RESOURCE OPTIONS

Netheridge Process Basis of Design Report

70088464-WSP-NETHSRO-RP-CY-3000
OCTOBER 2022

CONFIDENTIAL

Severn Trent Water

SEVERN TRENT SOURCES STRATEGIC RESOURCE OPTIONS

Netheridge Process Basis of Design Report

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ABBREVIATION AND ACRONYM LIST

Table 1 – Abbreviation and Acronym List

Abbreviation or Acronym	Meaning
°	Degrees
°C	Degrees Celsius
µg/l	Microgram per litre
µm	Micrometre
AFFF	Aqueous film forming foam
AMP8	Asset management plan eight
AOP	Advanced oxidation process
AOR	Actual oxygen requirement
AOX	Halogenated organic compounds
ASP	Activated sludge process
BAF	Biologically active filtration
BOD	Biological oxygen demand
BV/hr	Bed volumes per hour
CAPEX	Capital expenditure
C-Cl	Carbon-chlorine
CDR	Concept design report
C-F	Carbon-fluorine
C-H	Carbon-hydrogen
Cl	Chlorine
COD	Chemical oxygen demand
CRT	Canals and River Trust
CS_2	Carbon disulphide
DBP	Disinfection by-products
DEC	Design envelope confirmation

Abbreviation or Acronym	Meaning
DFMA	Design for manufacture and assembly
DOC	Dissolved organic carbon
DOL	Direct online
DSEAR	Dangerous substances and explosive atmospheres regulations
DWF	Dry weather flow
DWPA	Drinking water protected area
EA	Environment Agency
EDC	Endocrine disrupting chemical
EQS	Environmental quality standards
F	Fluorine
FBDA	Fine bubble diffused aeration
FFT	Full flow to treatment
FIT	Flow indicating transmitter
FOG	Fats, oils and grease
FST	Final settlement tank
G&S	Gloucester and Sharpness
g/hr	Grams per hour
g/l	Grams per litre
g/Nm^3	Grams per normal metre cubed
GAC	Granular activated carbon
GBT	Gravity belt thickener
gm^{-2}	Grams per square metre
H_2O_2	Hydrogen peroxide
HBCDD	Hexabromocyclododecane
HDPE	High density polyethylene
HGV	Heavy goods vehicle
HMI	Human machine interface

Abbreviation or Acronym	Meaning
HOF	Hands off flow
hPa	Hectopascal
IBC	Intermediate bulk container
IO	Input/output
kg/day	Kilograms per day
kg/ha/yr	Kilograms per hectare per year
kg/hr	Kilograms per hour
kg/l	Kilograms per litre
kg/m^2/hr	Kilograms per square metre per hour
kgO_2/day	Kilograms of oxygen per day
km	Kilometre
kV	Kilovolt
kW	Kilowatt
kWh/day	Kilowatt hours per day
kWh/kg	Kilowatt hours per kilogram
kWh/m^3	Kilowatt hours per cubic metre
l	Litre
l/day	Litres per day
l/hr	Litres per hour
l/min	Litres per minute
l/s	Litres per second
LoD	Limit of detection
LTP	Liquor treatment plant
m	Metres
m/hr	Metres per hour
m/s	Metres per second
m^2	Square metres

Abbreviation or Acronym	Meaning
m^3/hd/day	Cubic metres per person per day
m^3/m^2/hr	Cubic metres per square metre per hour
MBBR	Moving Bed Biofilm Reactor
MCC	Motor control centre
MCERT	Monitoring Certification Scheme
m^3	Cubic metres
m^3/day	Cubic metres per day
m^3/hr	Cubic metres per hour
mg/l	Milligram per litre
mJ/cm^2	Millijoules per square centimetre
ML	Mega litres
MLD	Megalitres per day
MLSS	Mixed liquor suspended solids
N/A	Not applicable
NDMA	N-Nitrosdimethylamine
nm	Nanometre
Nm^3/hr	Normal cubic metres per hour
no.	Number
NPV	Net present value
OPEX	Operational expenditure
PAC	Powdered activated carbon
PE	Population equivalent
PFAS	Perfluoroalkyl and polyfluoroalkyl substances
PFHxS	Perfluorohexane sulfonic acid
PFOA	Perfluorooctanoic acid
PFOS	Perfluorooctane sulfonic acid
PIT	Pressure indicating transmitter

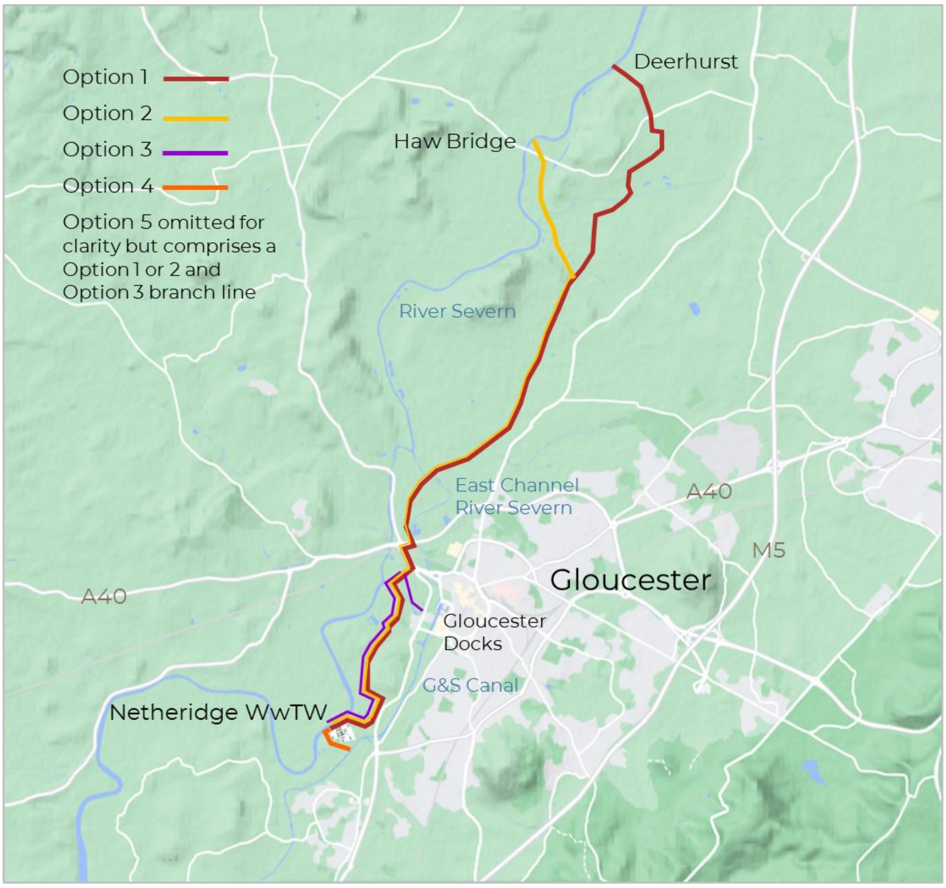
Abbreviation or Acronym	Meaning
PLC	Programmable logic controller
PPE	Personal protective equipment
ppm	Parts per million
ppmv	parts per million volume
PS	Pumping station
PST	Primary settlement tank
RAPID	Regulator's Alliance for Progressing Infrastructure Development
RAS	Return Activated Sludge
SAFF	Surface active foam fractionation
SAS	Surplus activated sludge
SBR	Sequential batch reactor
SCADA	Supervisory control and data acquisition system
SO_4	Sulphate
SRO	Strategic resource option
SSSI	Site of special scientific interest
SSVI	Stirred specific volume index
STS	Severn Trent sources
STW	Severn Trent Water
T	Tonnes
TDS	Tonnes dry solids
THP	Thermal hydrolysis process
TOC	Total organic carbon
TPT	Tryphenyltin
TSS	Total Suspended Solids
TWL	Top water level
UK	United Kingdom
USA	United States of America

Abbreviation or Acronym	Meaning
UV	Ultraviolet
VAT	Value added tax
VSS	Volatile suspended solids
WFD	Water framework directive
wt%	Weight percent
WTW	Water treatment works
WwTW	Wastewater Treatment Works

EXECUTIVE SUMMARY

35 megalitres per day (MLD) of treated sewage from Netheridge wastewater treatment works (WwTW) will be subjected to additional treatment and diverted to one of four proposed discharge locations to supplement water that has been abstracted as part of the wider Severn to Thames Transfer project. The treatment requirements vary dependent upon the discharge location. Final effluent flows in excess of 35 MLD at Netheridge WwTW will continue to be discharged to the existing final effluent outfall into the tidal zone of the River Severn.

Figure 1 – Severn Trent Sources (STS) Strategic Resource Option (SRO) Netheridge Transfer pipework overview



The design of each treatment train at the feasibility stage of the project will be used to develop capital expenditure (CAPEX), operational expenditure (OPEX) and carbon calculations to support solution decisions at the end of Gate 2. CAPEX, OPEX and carbon emissions details can be found in supplementary reports to this process basis of design report.

For each treatment option, permit requirements have been assumed based on preliminary screening performed by others for Option 1, and an assumption to prevent a deterioration in river quality for an array of micropollutants sampled during gate 1.

Given the uncertainty of permit requirements and treatment capability, a robust treatment train has been provided which can be rationalised at Gate 3 subject to bench scale and pilot plant studies.

OPTION 1 – RIVER SEVERN - DEERHURST

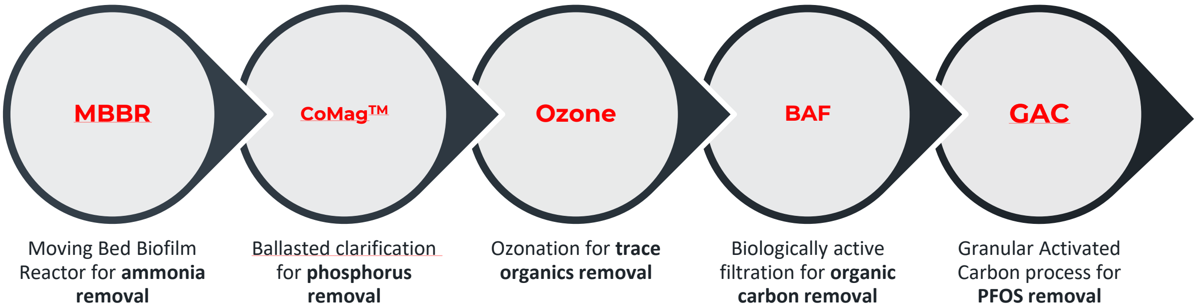
Option 1 provides treatment to permit discharge to the fresh water stretch of the River Severn at Deerhurst, approximately 18 kilometres (km) further North along the River Severn from Netheridge WwTW). The precise discharge location will be downstream of the proposed abstraction point at Deerhurst which will supply a new Water Treatment Works prior to transfer to the River Thames.

For option 1, there is an assumed requirement to remove:

- Ammonia
- Total phosphorus
- Biological Oxygen Demand
- Total suspended solids
- 2,4, dichlorophenol
- Chlorothalonil
- Nonylphenols (4-nonylphenol technical mix)
- Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol)
- Perfluorooctane sulfonic acid and its derivatives

This will be achieved using the treatment train shown in Figure 2:

Figure 2 –Option 1 Treatment Summary



OPTION 2 – RIVER SEVERN – HAW GAUGING STATION

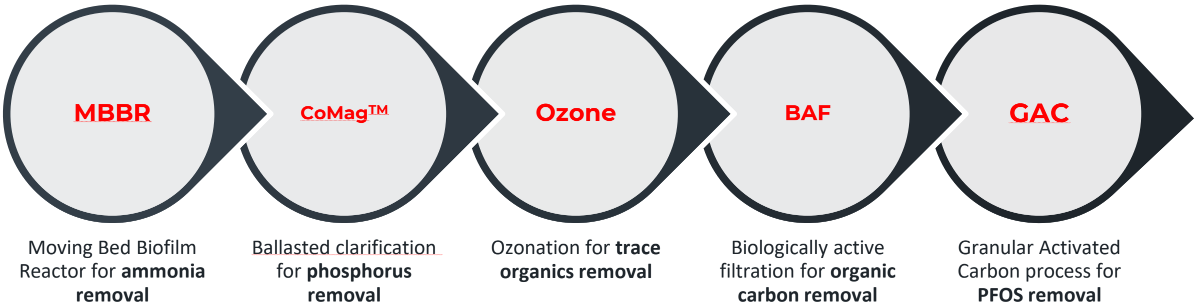
Option 2 provides treatment to permit discharge to the fresh water stretch of the River Severn upstream of the Environment Agency's gauging station at Haw via new 15.5 km long pipework north of Netheridge WwTW. This will supplement water that has been abstracted further upstream for treatment at Deerhurst water treatment works prior to transfer to the River Thames.

The water quality requirements to permit discharge to Haw gauging station are assumed to be identical to option 1, a requirement to remove:

- Ammonia
- Total phosphorus
- Biological Oxygen Demand
- Total suspended solids
- 2,4, dichlorophenol
- Chlorothalonil
- Nonylphenols (4-nonylphenol technical mix)
- Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol)
- Perfluorooctane sulfonic acid and its derivatives

This will be achieved using the treatment train identical to Option 1, as depicted by Figure 3.

Figure 3 - Option 2 Treatment Summary



OPTION 3 – RIVER SEVERN – EAST CHANNEL

Option 3 provides treatment to permit discharge to the east channel of the River Severn as far downstream of the existing Canal and Rivers Trust pumping station to Gloucester Docks as practicable via new pipework approximately 5 km in length from Netheridge WwTW, prior to the East Channel becoming tidal to prevent the risk of undiluted effluent drawn into the pumping station if the pumps cause backflow when operating.

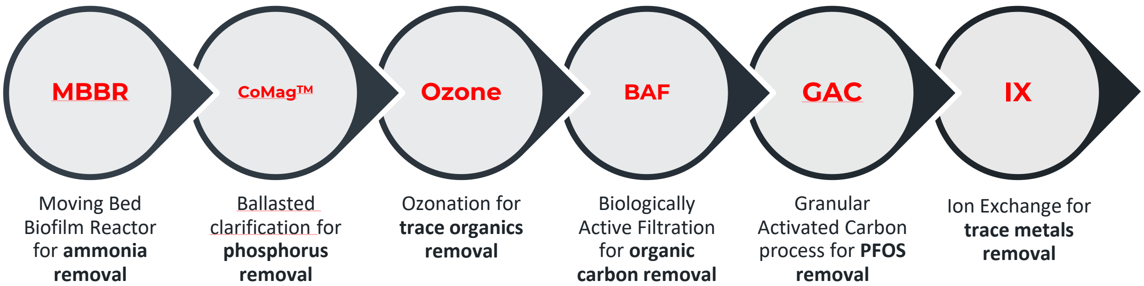
The water quality requirements to permit discharge to the East Channel have been assumed in the absence of screening exercises or dispersion and dilution modelling. The chemicals included in the list below in addition to the sanitary permit requirements of option 1 and 2 will not meet likely discharge permit conditions as a result of a review of environmental quality standards for this section of the River Severn and sampling data undertaken at gate 1, and therefore require removal:

- Ammonia
- Total phosphorus
- Biological Oxygen Demand
- Total suspended solids
- 2,4, dichlorophenol
- Chlorothalonil
- Nonylphenols (4-nonylphenol technical mix)
- Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol)
- Perfluorooctane sulfonic acid and its derivatives
- Chromium (III) dissolved
- Glyphosate
- Mercury dissolved
- Nickel dissolved
- Pentachlorophenol
- Terbutryn
- Tributyltin compounds (as tributyltin cation)
- Boron total
- Chloride
- Dibutyl phthalate
- Diethyl phthalate
- Diflubenzuron
- EDTA

- Mecoprop
- Permethrin
- Triclosan
- Cypermethrin
- Dichloromethane
- Hexabromocyclododecane (HBCDD)
- Lead dissolved
- Fluoride
- Mancozeb
- Maneb
- Sulphate
- Tributyl phosphate
- Triphenyltin (TPT) compounds

This will be achieved using the treatment train identical to Option 1 and Option 2 with additional ion exchange for metals removal downstream of the granular activated carbon stage, as depicted by Figure 4.

Figure 4 - Option 3 Treatment Summary



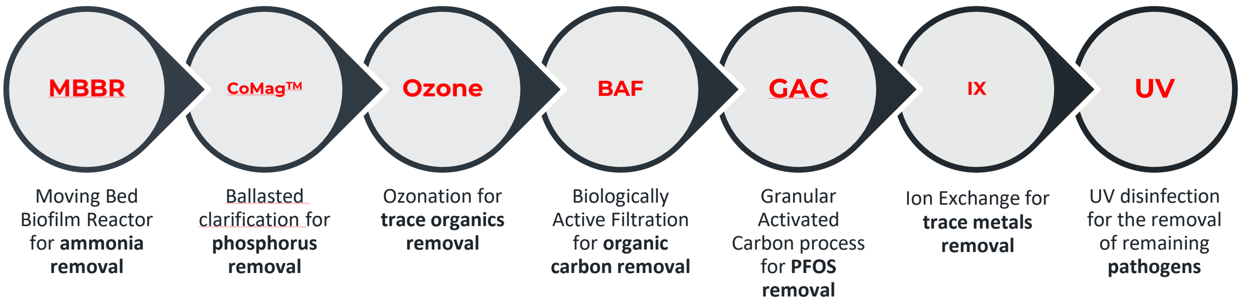
OPTION 4 – GLOUCESTER AND SHARPNESS CANAL

Option 4 provides treatment to permit discharge to the Gloucester and Sharpness Canal adjacent to Netheridge WwTW via new pipework approximately 400 metres (m) in length. This is to address the needs for additional water resources in the Wessex Water and Bristol Water regions.

The Gloucester and Sharpness Canal is a drinking water protected area. Water is abstracted from the canal by Bristol Water at Purton Water Treatment Works approximately 9.5 km downstream of the proposed discharge location. Engagement between Severn Trent Water (STW) and the drinking water inspectorate (DWI) / Environment Agency (EA) has begun, but there has been no confirmed water quality requirement. Therefore, it is assumed that the same ammonia and total phosphorus permit requirements will apply, as will a robust treatment process to remove micropollutants, similar to the East Channel treatment proposal.

The proposed treatment train is identical to Option 3, with an additional ultraviolet (UV) unit to provide disinfection to comply with drinking water protected area discharge requirements:

Figure 5 - Option 4 Treatment Summary



OPTION 5 – SOUTHWEST REGION BRANCH PIPELINE

Option 5 comprises additional pipeline for diversion of flows from the Netheridge to Deerhurst (or Haw Bridge) pipeline for discharge to the East Channel of the River Severn downstream of the intake for Gloucester Docks. This branch will follow the same route as Option 3.

The treatment process will be identical to option 3.

Figure 6 - Option 5 Treatment Summary

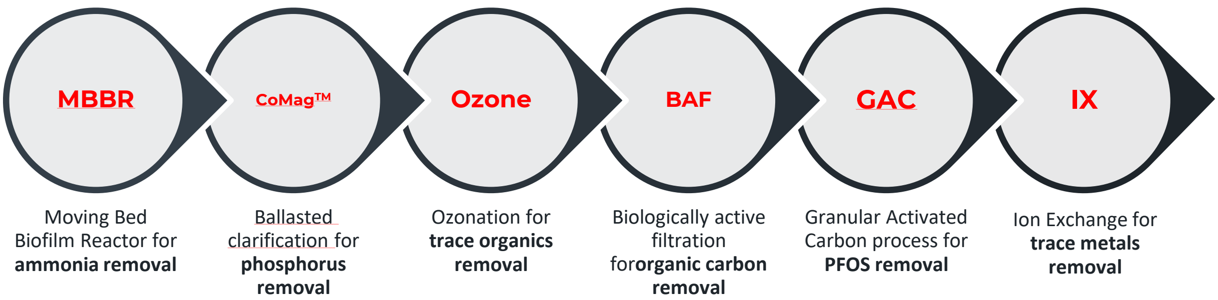


Figure 7 - Option 5 Discharge Location

KEY RISKS

The following key risks have been identified at Gate 2:

Table 2 – Key Risks

Key Risk	Description
Unconfirmed permit requirements	The permit requirements for each discharge location have been assumed, and therefore there is a risk proposed treatment scope is not suitable. A robust treatment train has been proposed to mitigate risk at the next gate.
Unknown treatment performance	The treatment technologies proposed will remove the chemicals identified, however how they will operate as one system is unknown (for example loading onto the Biologically Active Filtration (BAF) process as a result of oxidation of calcitrant organics by ozone). The results of pilot plant trials could affect unit sizing and scope requirements.
17 day notice period	A 17-day notice period has been given by Severn to Thames Transfer (STT) for the requirement of flow to be transferred from Netheridge SRO. There is a risk that loads will not be available during this period to test the system at full loading, there will be a period of less than 14 days for process validation, or of equipment failure during performance validation, risking noncompliance with water requirements.
Flow availability	There is a risk that without buffering capacity, 35 MLD cannot be provided by the SRO scheme on some days per year.
Proposed schemes at Netheridge	Possible improvement schemes at Netheridge WwTW have not been included in the proposed design for the SRO treatment plant. There is a risk that the overall solution provided is not a holistic one, risk unnecessary CAPEX and carbon emissions.
Ammonia feed to the Moving Bed Biolfilm Reactors (MBBR)	Given the performance of the existing activated sludge process (ASP), which achieves low ammonia concentrations, there is a risk the ammonia loading onto the MBBR process is too low to develop a healthy biomass.

Key Risk	Description
Perfluoroalkyl and polyfluoroalkyl substances	There is a risk that ozone will break down longer Perfluoroalkyl and polyfluoroalkyl substances (PFAS) chains into shorter chains, which are more difficult to remove and could pass straight through the granular activated carbon (GAC) adsorbers. This is particularly important with regards to the requirement to remove perfluorooctane sulfonic acid (PFOS). The fate of PFAS chains would need to be determined during a pilot trial.
Disinfection by-products	Ozone addition to water forms disinfection by products (DBPs) and could have a significant impact from a Water Safety Planning perspective. A BAF process has been included downstream to mitigate the emission of DBPs, and the GAC process provides resilience.
PFAS compounds permitting	Only PFOS has been listed as a PFAS compound requiring removal. There is likely to be more PFOA than PFOS in the effluent given the source of the wastewater and transitional processes in the wastewater treatment. According to industry experts, PFOA is currently the only PFAS compound the EA have listed with regards to legislation, but that is under review. The Drinking Water Inspectorate has produced specific guidance with regards to PFOA and PFOS in drinking water. Experts have also commented that perfluorohexane sulfonic acid (PFHxS) is about to come under the spotlight and has been used in far greater quantities over the last 20 years than PFOS, mainly in aqueous film forming foam (AFFF) where it forms a multitude of precursors. This will impact the capacity of the adsorbent process and reduce the throughput.
Supply of carbon	There are differing views, amongst industry experts, on the availability of carbon and whether the existing infrastructure in the United Kingdom (UK) is suitable to accommodate a treatment works in this scale. There is a risk, particularly if GAC is installed at other SRO schemes, that sufficient regeneration capacity is not available. There is no UK market for regeneration of GAC media used in a wastewater environment currently.
PFAS contaminated waste	The PFOS contaminated carbon will require vast amounts of energy to destroy the PFAS at 1200 degrees Celsius (°C) adding to the overall carbon footprint of the treatment, and the overall carbon impact of the scheme may not outweigh the benefit of reducing PFOS concentrations by 0.0063 micrograms per litre (µg/l) of an effluent that contributes 1 percent (%) to overall river flow. If

Key Risk	Description
	temperatures are not hot enough, hydrochloric acid is formed and there is no guarantee all PFOS will be removed. Industry experts have commented on the limited availability of PFAS contaminated waste disposal facilities in the UK and Europe. Waste disposal facilities in the United States of America (USA) (where PFAS removal is becoming more prominent) are beginning to reject waste contaminated with PFAS due to its legacy if not completely destroyed. Some companies have up to 100 tonnes of waste on site that they cannot currently dispose of.

OPPORTUNITIES

The following next steps have been identified at gate 2 and should be investigated further during gate 3:

Table 3 - Opportunities

Opportunities	Description
Permitting	The proposed processes are operationally intensive and chemically and electrically demanding. The overall environmental impact of the scheme may offset the benefit of removing trace pollutants by less than 1 µg/l. Permit requirements should be discussed with regulatory bodies to achieve the most environmentally beneficial solution.
Source control/ import reduction	Netheridge WwTW receives trade imports for sludge treatment and some trade waste. An investigation into these imports may highlight an opportunity for source control or import redirection to reduce the scope of the treatment options proposed.
Alternative primary phosphorus removal chemicals	Dosing ferric sulphate into the crude sewage can reduce biological oxygen demand load onto the ASPs, increasing the capacity for ammonia removal to achieve the assumed 1 milligram per litre (mg/l) consent in the existing ASP, reducing the requirement for tertiary ammonia removal.
Biological phosphorus removal	Biological treatment for phosphorus removal could reduce chemical consumption compared to chemical removal and offer resource recovery. Some biological phosphorus removal processes can remove ammonia, that would eliminate the need for MBBR processes.
Optimisation of the existing ASP process	Actual performance of the ASP exceeds design expectations, and the construction of the proposed tertiary MBBR process could be deferred until the requirement materialises (driven by population

Opportunities	Description
	growth). Optimisation of the existing process could provide robustness in the interim.
Wetlands technology for phosphorus removal	If after discussion with regulatory bodies the assumed phosphorus permit is relaxed, Wetlands may present themselves as a low carbon solution to phosphorus removal. The associated land take will be significant but there are environmental benefits. Water companies are currently being asked by Ofwat to include nature-based solutions in their business plans for asset management period eight (AMP8).
Filtration technologies for tertiary phosphorus removal	Filtration tertiary solids removal processes for low phosphorus permits may be more suitable for Netheridge because they can be turned off when not required, require fewer chemicals than CoMag^{TM} and may be better suited to the large variation in flow. The impact and control of backwash returns to the head of the works must be reviewed to confirm suitability.
Removal performance confirmed by pilot plant	Pilot plants can be used to assess the removal performance of advanced treatment processes with regards to micropollutants. The results of the pilot plants may highlight opportunities for scope reduction.
Reduction in sludge volume	If the MBBR performs better than expected with regards to solids carry over or is deemed surplus to requirement this will significantly reduce the sludge production - potentially by up to 10 times. This reduction in sludge volume could lead to the utilisation of the existing sludge handling facilities rather than constructing new.

NEXT STEPS

The following next steps have been recommended for inclusion during gate 3:

Table 4 – Next steps

Next steps	Description
Permit requirements	The permit requirements for the proposed discharge location should be discussed and confirmed with the EA.
Data capture	Further data on sewage throughout the treatment works should be collected to better identify opportunities for existing asset optimisation and potentially reduce SRO treatment scope.

Next steps	Description
	Existing final effluent quality and river data will continue to be collected to improve dilution modelling to help further clarify treatment targets for Gate 3 scheme design.
Netheridge WwTW upgrades	Seek further confirmation within STW of proposed upgrades to the existing works to comply with potential DWF, phosphorus removal or THP projects. This may lead to opportunities to deliver holistic solutions and reduce the proposed scope of the SRO treatment process, or in the case of a DWF project, highlight an issue with the hydraulic capacity of the existing works. .
17 day start up period	Discuss the 17 day start up period with Severn Trent's process engineering team and relevant water safety planning teams to develop a robust strategy to increasing flow and validating performance prior to transfer to the new discharge location.
Flow	The approach to turndown of the treatment processes should be further refined to optimise plant stability and buffering storage volumes required. The requirement for pumping between the SRO treatment plant discharge and existing Netheridge final effluent outfall should be confirmed. Flow under gravity may be possible if plant elevation allows.
Flow availability	Review the availability of final effluent flow to confirm the viability of the Netheridge STS SRO to provide 35 MLD, and the requirement for storage capacity based on the validity of MCERTS data.
Primary phosphorus removal	Model the existing ASP and final settlement tank (FST) processes to confirm in more detail the impact of ferrous sulphate dosing into the recycled activated sludge (RAS) stream. Review existing surplus activated sludge (SAS) thickening capacity and confirm available headroom. Produce an alkalinity consumption model encompassing future growth. An assessment of the existing primary sludge handling capacity. Review the benefits of dosing ferric sulphate upstream of the primary settlement tanks.

Next steps	Description
Ammonia removal	Confirm with regulatory bodies the requirement for ammonia removal at the proposed discharge locations. Confirm the existing ASP capacity and when it will be met in relation to expected growth and potential thermal hydrolysis process (THP) scheme.
Tertiary phosphorus removal	Confirm the total phosphorus permit requirements for each discharge location. Review the impact of backwash returns from filtration processes on hydraulics at the head of the works.
Advanced treatment processes	Confirm the expected advanced treatment removal performance with pilot trials to fully understand water chemistry and confirm suitability and design parameters.
Returns	Review the hydraulic capacity at the head of the works and confirm the capacity to receive returns from the tertiary treatment process.
Geotechnical investigation	Undertake geotechnical ground investigations in the proposed treatment location to confirm ground and groundwater conditions.
Topographic survey of proposed area	Undertake detailed topographic survey of proposed construction & tie-in locations. This will facilitate cut/fill calculations to be undertaken as well as provide reliable elevations for further design work. This in turn will inform the system hydraulics and opportunities to reduce pumping requirements.
Utilities	Undertake a full and detailed utilities survey of the proposed areas of construction, roads that could be used for pipeline corridors and any tie-in locations. Statutory providers should be engaged early to commence discussions around the proposed utilities diversions and decommissioning of the gas main
Potable water supply	An assessment of the current potable supply to site needs to be undertaken to determine if the existing supply can be improved or if a new supply to site needs to be provided.
Water safety planning risks	Option 4 discharges treated effluent directly into the Gloucester and Sharpness Canal, a drinking water protected area. This option to supply Purton WTW (Bristol Water) is identified as an opportunity and if

Next steps	Description
	selected at Gate 3, the impact of the discharged effluent, which should be confirmed by pilot plant studies, on existing water safety planning risks should be further assessed using the All Company Working Group template.
Environmental surveys	An appropriate environmental assessment should be undertaken, this will most likely include a Phase 1 habitat assessment as well as reptile surveys, badger surveys and bat surveys. The EA flood risk maps show that the proposed location for the new treatment is in an area at very low risk of flooding from rivers or surface water. It should be considered whether further flood risk assessment is required.
Control system	At this point, a single motor control centre (MCC) with single programmable logic controller (PLC) has been assumed for the whole treatment plant. Review the benefit of multiple MCCs and alternative PLC arrangements which may be preferred. Further definition of the operational philosophy will help confirm the human machine interface (HMI) requirements, i.e., an extension to the existing site supervisory control and data acquisition (SCADA) or a standalone.
Dangerous substances and explosive atmospheres (DSEAR)	A full DSEAR assessment should be carried out once the level of design detail has sufficiently increased.

1 INTRODUCTION

1.1 CONTEXT

This Netheridge Process Basis of Design Report details the treatment and technology requirements for the Severn Trent Sources Strategic Resource Option (STS SRO) Netheridge Concept Design, building on the Gate 1 report (by others).

It should be read in conjunction with the Severn Trent Sources Strategic Resource Option Netheridge Concept Design Report (CDR) and is part of the suite of reports completed in support of Severn Trent Water's (STW) Regulator's Alliance for Progressing Infrastructure Development (RAPID) Gate 2 Submission.

Other reports completed as part of the Gate 2 concept design development include:

- Severn Trent Source SRO - Netheridge Concept Design Report (Annex A1)
- Severn Trent Source SRO - Netheridge Pipeline Route Appraisal Report (Annex A2)
- Severn Trent Source SRO - Netheridge Carbon Report (Annex A4)
- Severn Trent Source SRO - Netheridge Carbon Report (Annex A5)

The STS SRO comprises of two elements: the diversion of treated effluent from Netheridge wastewater treatment works (WwTW) and the reallocation of the Mythe water treatment works (WTW) River Severn abstraction licence. This report covers the Netheridge element of the STS SRO and will be referred to as the Netheridge SRO.

1.2 SEVERN TO THAMES TRANSFER STRATEGIC RESOURCE OPTION (STT SRO)

The intent of the STT SRO scheme is to transfer up to 500 megalitres per day (MLD) from the River Severn to the Thames region to mitigate forecast supply risks. The aim of the Netheridge SRO is to divert part of the Netheridge WwTW final effluent flow to augment flow transfers from the River Severn to the Thames Water regions.

At Gate 1 there were two options being considered:

1. Abstraction from the River Severn with treatment at Deerhurst before transfer via pipeline to Culham for onward distribution.
2. Abstraction via the Gloucester Docks, transfer along the Cotswolds canals, treatment and then pumped from Lechlade to Culham for onwards distribution.

The option choice for the STT SRO will dictate the discharge location for the Netheridge SRO, with flows being transferred to Deerhurst for option 1 or to the canal for option 2.

The Netheridge SRO requirements are primarily determined by the STT SRO operational regime. The scheme will only transfer flows when called for by the STT SRO. The key operational parameters of the STT SRO are as follows:

- The STT SRO will provide at least 17 days' advance notice of the intent to begin transfer of flow to the Thames region to allow for time to increase flow through the treatment process from sweetening flows (20 MLD) to full flow (35 MLD), testing, priming of the pipeline and testing of the pump system.

- The STT SRO will operate for a minimum of 20 days once fully operational to prevent frequent recommissioning and decommissioning of the transfer pipe and pumps and the SRO treatment process at full flow.
- The Netheridge SRO will provide 35 MLD for River Severn flow augmentation when the STT SRO scheme is operational.
- The Netheridge SRO will provide the sweetening flow to the STT SRO (20MLD) when levels in the River Severn are below 'hands off' flow'(HOF) AND the STT SRO cannot abstract "sweetening" flow from the River Severn without augmentation from other source.

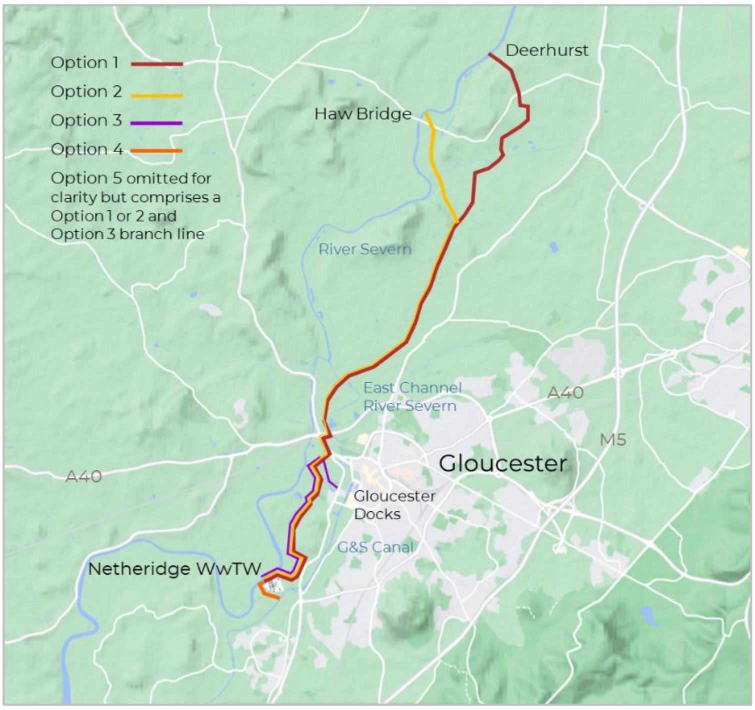
1.3 SCHEME OVERVIEW AND LOCATION

The intent of the Netheridge part of the STS SRO scheme is to divert up to 35 MLD of treated effluent from the Netheridge WwTW to augment the STT SRO at its point of abstraction from the Severn. This report details the development of the treatment options in support of STW's Gate 2 STS SRO submission.

Netheridge WwTW is located south of Gloucester and is bounded by the River Severn to the west and the Gloucester and Sharpness Canal (G&S Canal) to the east, with flows arriving from Gloucester and the surrounding catchments. Permitted dry weather flow (DWF) is 495 litres per second (l/s) (42.8 MLD) and full flow to treatment is 1215 l/s (105 MLD). It is important to note for later discussions that whilst this is the permitted DWF, the actual DWF recorded for the WwTW is lower.

The existing treatment process comprises screening, grit removal, 4 number (No.) primary settlement tanks (PST), biological treatment via 6 No. Activated Sludge Process (ASP) lanes and 6 No. final settlement tanks (FST). The WwTW currently discharges treated effluent to the tidal zone of the River Severn adjacent to site. Figure 1-1 shows the location of Netheridge WwTW and the new discharge location options.

Figure 1-1 - STS SRO Netheridge Transfer Pipeline Overview



The Netheridge scheme will comprise two main elements:

- Additional treatment of the diverted flow to meet the assumed higher water quality standards at the relevant discharge location.
- Conveyance of flows to the discharge point via pumped pipeline.
- The discharge location of Option 4 is into the Gloucester and Sharpness Canal. This option will impact the raw water used for abstraction at Purton Water Treatment Works further downstream and constitute a form of indirect water reuse.

The scheme will also include the required monitoring and control to allow linkages to the STT SRO.

The additional treatment required for the Netheridge WwTW final effluent is understood to be dependent on the discharge location. The current discharge location for Netheridge effluent is in the tidal zone of the River Severn. This allows for an effluent quality standard that is less stringent than that required for discharge to a freshwater river environment. Therefore, any discharge further north on the River Severn will need additional treatment to meet the required quality standards.

1.4 GATE 2 OPTIONS

A summary of the Gate 2 options is provided below.

OPTION 1 – RIVER SEVERN – DEERHURST

Option 1 comprises:

- A pipeline (approximately 18 kilometres (km) in length) from Netheridge WwTW to the River Severn immediately downstream of the new STT SRO Deerhurst Water Treatment Works.
- Treatment to comprise:

- Primary total phosphorus removal (Iron based coagulant dosing into the existing ASP)
- Nitrification (Moving Bed Biofilm Reactor (MBBR))
- Tertiary total phosphorus removal (CoMag^{TM})
- Pesticide, herbicide and perfluorooctanesulfonic acid (PFOS) removal (ozonation, biological aerated flooded filter (BAF) unit and Granular Activated Carbon (GAC).

Figure 1-2 - Option 1 Treatment Summary

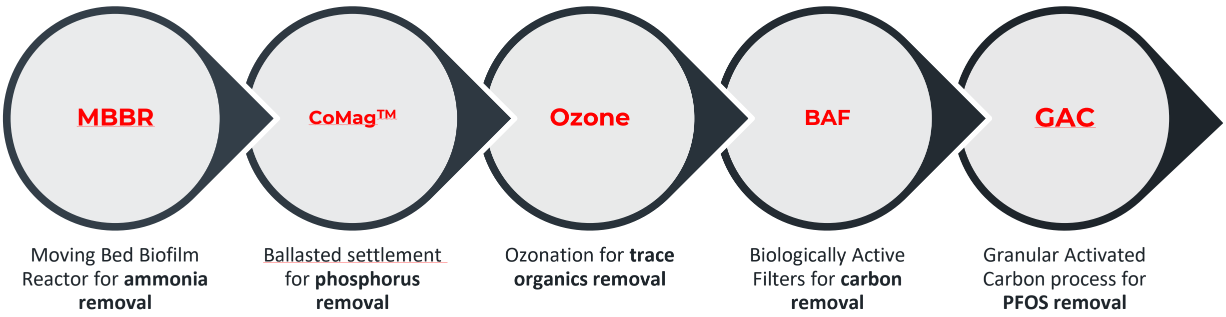


Figure 1-3 - Option 1 and 2 Discharge locations

OPTION 2 – RIVER SEVERN - HAW GAUGING STATION

Option 2 comprises:

- A pipeline (approximately 15.5km in length) from Netheridge WwTW to the River Severn immediately upstream of the Environment Agency's (EA) Gauging Station at Haw.
- Treatment to comprise:

- Primary total phosphorus removal (Iron based coagulant dosing into the existing ASP)
- Nitrification (MBBR)
- Tertiary total phosphorus removal (CoMag™)
- Pesticide, herbicide and PFOS removal (ozonation, BAF units and GAC)

Figure 1-4 - Option 2 Treatment Summary

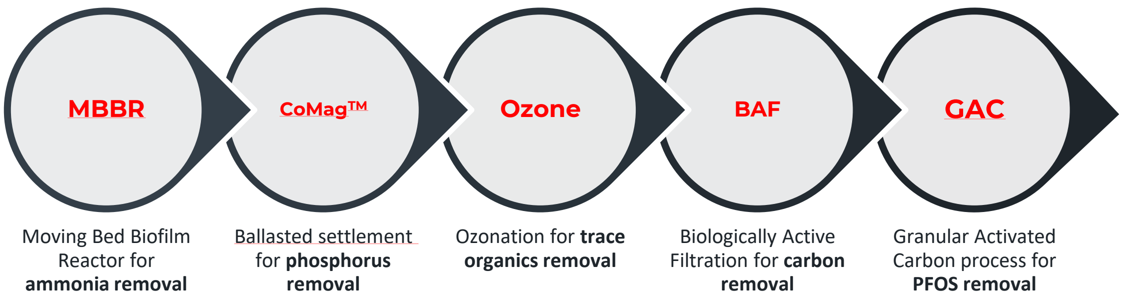


Figure 1-5 - Option 1 and 2 discharge location

OPTION 3 – RIVER SEVERN - EAST CHANNEL

Option 3 comprises:

- A pipeline (approximately 5km in length) from Netheridge WwTW to the east channel of the River Severn downstream of the existing Canal and Rivers Trust (CRT) pumping station to Gloucester Docks. The River Severn splits into east and west channels at Upper Parting near Maisemore to the North of Gloucester, before the channels join again at Lower Parting West of Gloucester.
- Treatment to comprise:

- Primary total phosphorus removal (Iron based coagulant dosing into the existing ASP)
- Nitrification (MBBR)
- Tertiary total phosphorus removal (CoMag™)
- Micropollutants removal (ozonation, BAF units, GAC and ion exchange)

Figure 1-6 - Option 3 Treatment Summary

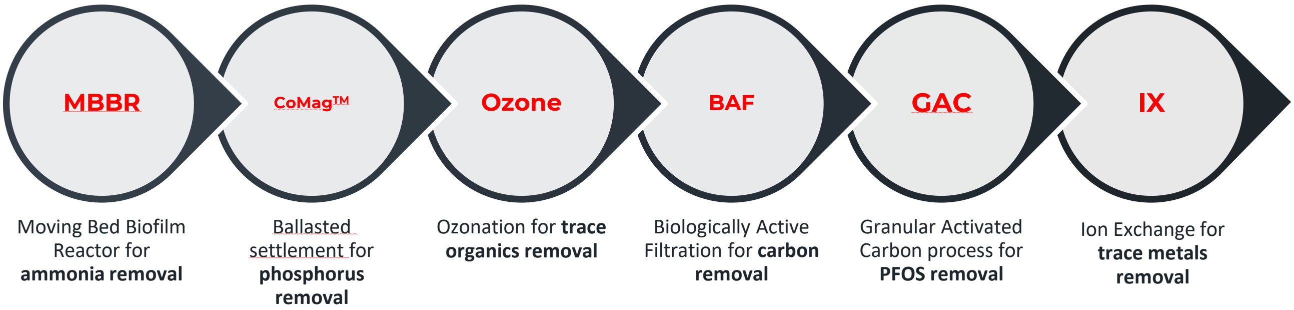
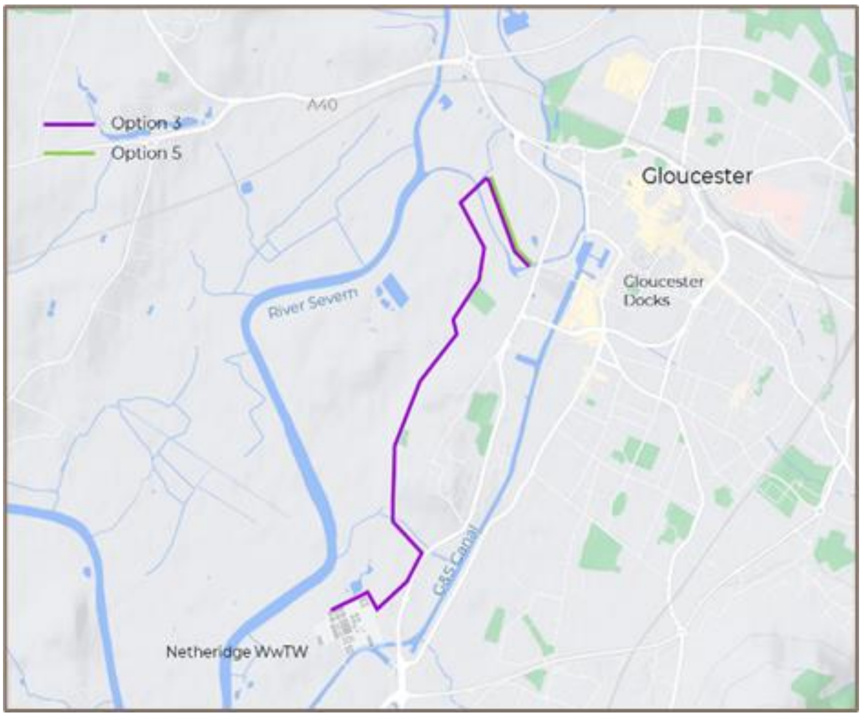


Figure 1-7 - Option 3 Discharge Location



OPTION 4 – GLOUCESTER AND SHARPNESS CANAL

Option 4 comprises:

- A pipeline (approximately 400m in length) from Netheridge WwTW to the Gloucester and Sharpness Canal.
- Treatment to comprise:

- Primary total phosphorus removal (Iron based coagulant dosing into the existing ASP)
- Nitrification (MBBR)
- Tertiary total phosphorus removal (CoMagTM)
- Micropollutants removal (ozonation, BAF units, GAC and ion exchange)
- Disinfection (Ultraviolet (UV))

Figure 1-8 - Option 4 Treatment Summary

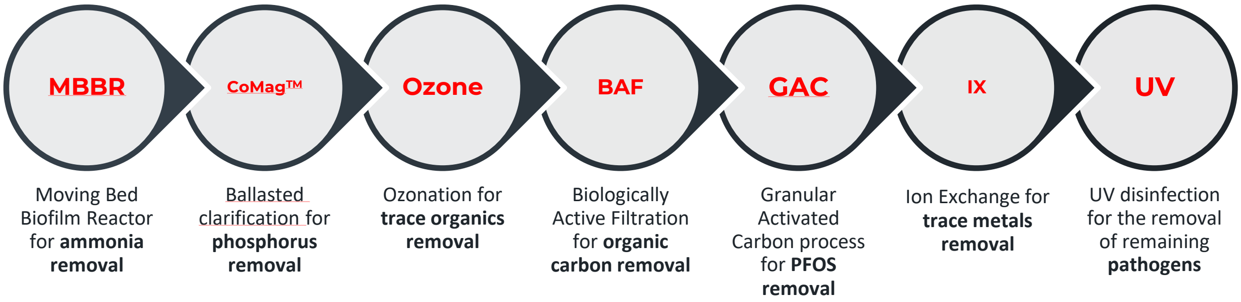


Figure 1-9 - Option 4 Discharge Location

OPTION 5 – SOUTHWEST REGION BRANCH PIPELINE

Option 5 comprises:

- Additional pipeline for diversion of flows from the Netheridge to Deerhurst (or Haw Bridge) pipeline for discharge to the East Channel of the River Severn downstream of the intake for Gloucester Docks. This branch will follow the same route as Option 3.
- The treatment process will be identical to option 3.

Figure 1-10 - Option 5 Treatment Summary

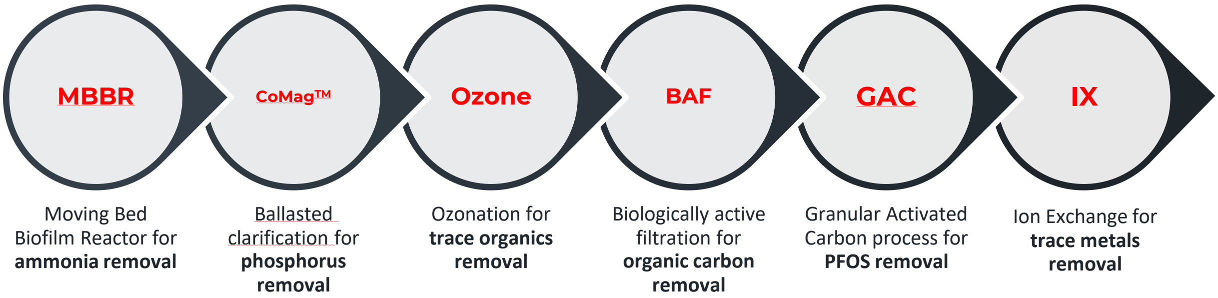


Figure 1-11 - Option 5 Discharge Location

1.5 REPORT REQUIREMENTS AND SCOPE

The scope of work for the Netheridge Process Basis of Design Report included:

- Gate 1 Options appraisal.
- Options Development including:

- Detailed development of treatment options
- Site layouts
- Power requirements
- Development of an operational strategy
- Carbon cost estimation
- Biodiversity Net Gain opportunities
- Requirements for investigations and studies in the next phase of the project
- Incorporate feedback from the environmental teams and the land planning teams

- Capital and operational cost estimation, including net present value (NPV), optimism bias and costed risk assessment

This report describes the existing treatment process and its operational performance before leading on to how 35 MLD will be diverted to the new treatment process prior to transfer to the point of discharge.

The derivation of water quality requirements is described in Section 4.1 which then informs treatment process selection in Section 5. An outline control philosophy in Section 9 describes how each process will operate.

1.6 APPROACH AND METHODOLOGY

The purpose of this Process Basis of Design Report is to present the known data, assumptions, treatment selection and design used to provide robust solutions that can achieve the assumed permit requirements for each of the four discharge locations. Where applicable, the treatment technologies identified at Gate 1 have been carried forward to Gate 2. Gate 2 basis of design assumptions are described in Section 4.2. Upon review of flow through the works and more recent modelling and permitting information, the process requirements have changed.

There is scope for refinement of the design in Gate 3 when the permit requirements are more defined, and the demands of the STT are better understood. Alternative process technologies are also presented to ensure the best treatment fit can be achieved.

2 NETHERIDGE WWTW

2.1 TREATMENT PROCESS AND SITE LAYOUT

Netheridge WwTW serves a population equivalent (PE) of 229,628 (2020 PE data) and comprises three fine inlet screens (duty/assist/assist), grit removal (detritor), four identical radial type primary settlement tanks, ASP lanes and six radial FSTs before discharge to the River Severn to the north of the site boundary. Figure 2-1 shows the site layout.

Figure 2-1 - Netheridge WwTW Site Layout

[redacted]

2.1.1 INLET WORKS

The inlet pumping station comprises five DWF pumps (duty/assist/assist/assist/standby) which transfer flow to the main process. The inlet pumping station also contains four unsettled storm screen pumps that transfer unsettled storm flow directly to the River Severn via a set of screens at flows greater than 2,720 l/s.

Screenings from the DWF fine inlet screens are dewatered by duty/ standby double screwed dewatering units, which also receive storm screenings, and are transferred to a skip for offsite removal.

Trade imports enter the process just upstream of the inlet screens. Liquors from the primary gravity belt thickeners (GBT) combine with the imported sludge stream and the contents of the trade waste blending tank are pumped to the main process downstream of the grit removal plant. The grit removal plant (detritor and reciprocating rake) operates well with little evidence of grit in the PSTs, however, may soon be coming to the end of its asset life (construction was in the 1980s).

2.1.2 PRIMARY SETTLEMENT TANKS

The Primary Settlement Tanks (PSTs) PSTs are fed from a central distribution chamber. The peripheral weir of each PST has no v-notches. There were no obvious signs of short circuiting or suspended solids carry over during a site visit. Each PST has a duty/ standby desludge pump which operates on a timer and sludge is generally removed at 1% dry solids content to prevent significant sludge accumulation but can stress the sludge handline process. Centrate returns from the sludge centrifuges are returned to the PST distribution chamber.

2.1.3 ACTIVATED SLUDGE PLANT (ASP) FEED

PST effluent combines with the return activated sludge (RAS) removed from the FSTs, is lifted by three Archimedes screws before even distribution to each ASP lane by a common distribution chamber. No.1 screw is inverter-driven; however, the other screws operate as direct online (DOL). There is a scheme to change all screws to be inverter driven.

2.1.4 ASP LANES

Each ASP Lane has an anoxic zone. The mixers in the anoxic zones were offline during the site visit but there is an intention to bring these back into operation. Three blowers provide air to the ASP fine bubble diffused aeration (FBDA) to support biological activity and the speed and flow control valve positions are controlled via ammonia monitors. The age and efficiency of the diffusers is unknown.

The oversizing of the ASP lanes means taking one out for maintenance does not cause detriment to final effluent quality removing the need for temporary treatment. Each lane is taken offline every 3 years to undertake maintenance. During the site visit on 14/12/2021, there were no reported issues with the ASP process.

2.1.5 FINAL SETTLEMENT TANKS (FST)

Effluent from the ASPs flows via gravity to the FST distribution chamber. Each FST receives flow from each ASP lane (there is no designated FST for each ASP lane). The RAS/surplus activated sludge (SAS) split is achieved simply by operating the SAS pumps on a timer, and control with valving operation has been removed. The FSTs generally perform well, however there was one reported occurrence where the sludge blankets were lost and took a whole sludge age to recover. This was believed to be caused by the drip feed trade import pumps operating at 100%, but its composition was unknown.

Final effluent from the FSTs flows via gravity to a wash water cooling pumping station. This houses three pumps to transfer final effluent to the sludge dryers for cooling, and wash water lift pumps to supply the wash water booster tank. The wash water booster set positioned closer to this pumping station provides wash water at 7 bar for the primary sludge screens, screenings handling, GBTs, foam control in the centrifuges. There is no flow meter on the wash water system to identify the demand.

2.1.6 SLUDGE TREATMENT

The sludge treatment area is located to the east of the site. Netheridge WwTW is a strategic sludge treatment centre and processes sludge from over 100 regional WwTWs. It also processes trade waste which are tankered to site for treatment by the digestion stream, Combined heat and power (CHP) engines were installed in AMP5.

Imported wastes accepted for delivery are discharged into one of two import tanks. One import tank is for low strength wastes, the other for high strength wastes. Strength refers to the ammonia and chemical oxygen demand (COD) within the waste stream and influences how the waste streams are blended prior to the digesters.

Indigenous sewage sludge is removed from the primary settlement tanks then pumped into a holding tank and mixed with thickened surplus activated sludge (SAS) and imported sludge.

Pre-mixed wastes are transferred via pipework to one of four primary anaerobic digesters. The contents of the anaerobic digesters are mixed while the contents undergo anaerobic degradation. The anaerobic digestion process gives rise to biogas (largely methane), which rises through the digester for capture and transfer to the adjacent floating roof gas storage tank for utilisation on site. The anaerobic digesters operate as a continuous process with sludge being added and treated sludge extracted. Sludge is pumped to open pathogen kill tanks (secondary sludge tanks) for 14-day storage. Sludge extracted from the pathogen kill tanks is then transferred to the buffer tanks, before dewatering occurs by centrifuge. Supernatant liquor is returned to the head of the works, and sludge cake is transferred via conveyors to the sludge cake pad.

Due to levels of siloxanes within the biogas, the biogas is passed through a GAC filter system, and the filtrate is returned to the head of the works. The site has two gas engines for the combustion of biogas and generation of electricity used both within the site and exported to the national grid.

The site is well served by a network of roads that permit tanker movement. There are no available spare or abandoned tanks on site that could be utilised as part of the new treatment process.

There are no reports of the tidal influence of the River Severn causing problems at the final effluent flume. However, it can back up along the unsettled storm outlet at the head of the works.

During the site visit on 14/12/2021 there were no reported issues with fats, oils and greases (FOG) or grit handling. Wash water is critical for the sludge process, and there have been reports of wash water deficiency during summer months believed to be due to inefficient equipment rather than low flow. Final effluent is also used for cooling water in the sludge drier plant.

The proposed treatment stream for the STT project will take a portion of final effluent from downstream of the wash water cooling pumping station given the high dependency on wash water. Final effluent is sampled from the final effluent flume to the North of site. The monitoring certification scheme (MCERTS) meter is also located along this flume, confirmed by site operations. Therefore, the existing final effluent quality will be used as a basis of design. The existing process has not been modelled to account for any proposed population growth or potential improvement projects which could affect final effluent quality.

2.2 EXISTING FINAL EFFLUENT QUALITY

The sanitary requirements of the existing final effluent permit are shown in Table 2-1. This data was provided by Atkins and consists of 14 spot samples taken at monthly intervals between 10/12/2020 and 12/10/2021, except for April 2021 and June 2021 where two samples were taken each month.

Table 2-1 – Netheridge Sanitary Permit Requirements

Determinant	Sanitary Consent	Netheridge Final Effluent (Average)	Netheridge Final Effluent (Maximum)
Biological oxygen demand (BOD) (milligrams per litre (mg/l))	25	5.7	13
Ammonia (mg/l)	15	0.9	4.6
Total Suspended Solids (TSS) (mg/l)	45	10	65

3 EFFLUENT VOLUMES AND FLOW DIVERSION

It is proposed to divert flow from downstream of the existing wash water cooling pumping station to a new tertiary treatment process dedicated to water transferred as part of the STS/STT project.

Existing STS documents state that the capacity of the Netheridge SRO is up to 35 MLD and this figure has been taken as a design criterion for Gate 2. The original assessments that resulted in the selection of this figure are not available for review. Analysis of historic flow data indicates that this flow rate is not guaranteed to be available from the upstream process. Further assessment is needed to model future flows including an end-to-end assessment of the upstream process to transfer discharge point.

3.1 35 MLD FLOW AVAILABILITY

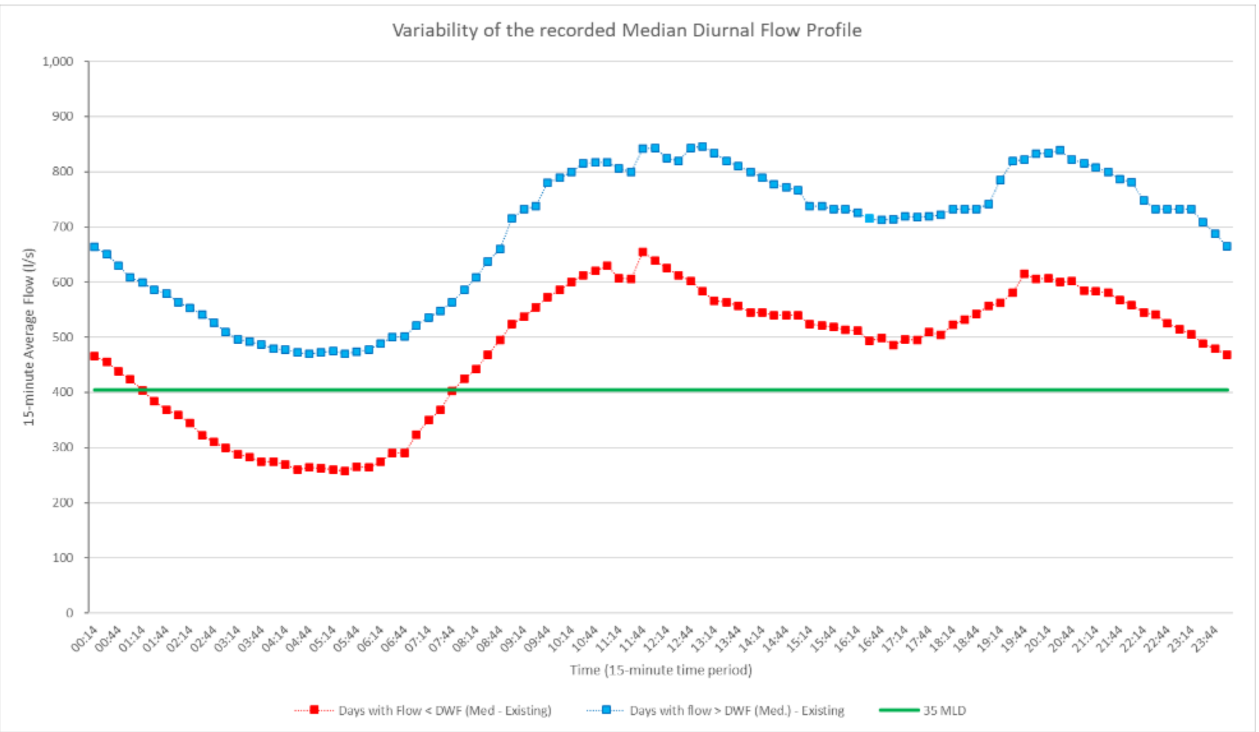
During a site visit in December 2021, it was reported that summer flows can fall below 405 l/s (the instantaneous flow for 35 MLD), and the works may not be able to provide 35 MLD at a constant rate, particularly in summer months when it is most likely to be needed.

A dataset of Netheridge WwTW fifteen-minute average flows recorded by the site MCERTS flowmeter (ident ID E1426 & E127930) which measures final effluent flow (confirmed by site operators) has been reviewed. The dataset extends over a period from the 1st of January 2017 through to the 31st of December 2021, a total of 1,826 days. The period from the 8th through to the 29th of October 2019, 22 days, has been discounted due to an apparent failure of the MCERTS meter (or extended works outage), giving a total of 1,804 days.

Within this timeframe there were 580 days where the recorded flow was below the DWF of 42.8 MLD. Of these, there were 38 days where the recorded flow was below 35 MLD and flow on 4 of these was recorded below 20 MLD. On inspection of the data for the four days below 20 MLD, no pattern that could reasonably be attributed to an instrument failure was evident. These low-flow days may reflect actual flows through Netheridge WwTW due to extended dry periods or due to works outages. These figures are an approximation, the actual number of days where available final effluent for Netheridge SRO treatment is below the target may be higher.

The diurnal flow profile for days with final effluent flow below DWF (42.8MLD) verses days where flow exceeds DWF is presented in Figure 3-1. Importantly, the variability of flow (minimum and maximum values) across the period reviewed shows the range of flow, at each 15-minute time interval, is wide. It is therefore unclear the extent to which an average diurnal flow is predictable; further, the volume of balancing storage necessary to smooth out this variability of flow may be significant.

Figure 3-1 - Variability of the recorded median diurnal flow profile



The red trend represents the median diurnal flow profile of days with flow less than dry weather flow (42.8 MLD (495 l/s)), when there is most likely to be a demand for flow by STT. The blue trend represents the median diurnal flow profile of days with flow greater than DWF. The 'typical' diurnal profile shown by the medial flows could exist anywhere between the lower (min) and upper (max) chart lines. The average difference between instantaneous flows of days where cumulative flow is less than DWF, and days where cumulative flow exceeds DWF is 227 l/s. Figure 3-1 shows flow to the tertiary treatment system should follow the diurnal flow pattern, and at times will take all flow prior to the existing outfall.

3.2 FLOW PROFILE

The median of MCERTS fifteen-minute flow data shows an ‘average’ diurnal profile with lower flow through the early hours and higher flow in the afternoon. Assuming a constant flow rate, a transfer of 35 MLD would require an effluent inflow of 405 l/s. There are only 391 days (22%) where the minimum instantaneous flow rate is above this.

In response to the variability in available final effluent, the STS SRO treatment process will need to either vary the treatment and transfer flowrates (following the available final effluent flow), buffer a volume of final effluent to smooth out the varying flows, or a combination of both.

Considering the impact of balance storage alone, with a constant flow of 405 l/s to the STS SRO treatment a balancing capacity of approximately 9 megalitres (ML) would be required when considering only those days where the recorded flow exceeds 35 MLD. Including days with flow below 35 MLD, i.e., balancing flows across one or more days, could see the storage volume needed exceed 100 ML; this figure is driven by an extended period of low flow recorded between 18th June to 3rd July 2018. The validity of this data should be reviewed at Gate 3. This volume of storage is considered to be economically onerous and an approach utilising buffering storage alone to manage variability in final effluent flow is discounted.

Consequently, the abstraction of final effluent from the Netheridge WwTW discharge for STS SRO treatment and transfer will, by necessity, need to fluctuate in response to the variability in the Netheridge WwTW effluent flow.

Considering a varying flow range only, based on the MCERTS 15-minute average flow data, a maximum flow to the STS SRO treatment of approximately 880 l/s (73.44 MLD) would have been required to deliver 35 MLD on all days except those 38 days (within the assessed period of 1,826 days) where the inflow to Netheridge was below 35 MLD. It is important to add that the demand for water in the Southeast is likely to be during summer months, and potentially when the catchment serving Netheridge is in drought conditions. This could lower the proportion of days where 35 MLD can be achieved.

The minimum flow to the STS SRO treatment has no direct impact upon the cumulative volume that would have been treated. This presumes, at flows below the STS SRO treatment minimum, final effluent would be captured within the initial pumpstation wet-well and subsequently passed to treatment by intermittent pump-down operation. The direct impact that the minimum STS SRO treatment flow has is on the frequency and duration of periods where intermittent stop-start flow will be seen.

Designing and operating a treatment and transfer scheme to precisely match the variation in incoming flow is considered highly impractical, with the required flow rate ranging to 880 l/s. Amongst other issues, this would result in an uneconomical overcapacity in the treatment process and a lack of process stability.

To derive a diurnal flow profile to be used for process design, the profile across days from each of 2019, 2020 and 2021 were selected randomly from a pool of the driest days in those respective years to allow review of the diurnal profile in typical dry conditions, refer to Figure 3-2, Figure 3-3 and Figure 3-4.

Figure 3-2 - Dry Day Flow Profiles 2019

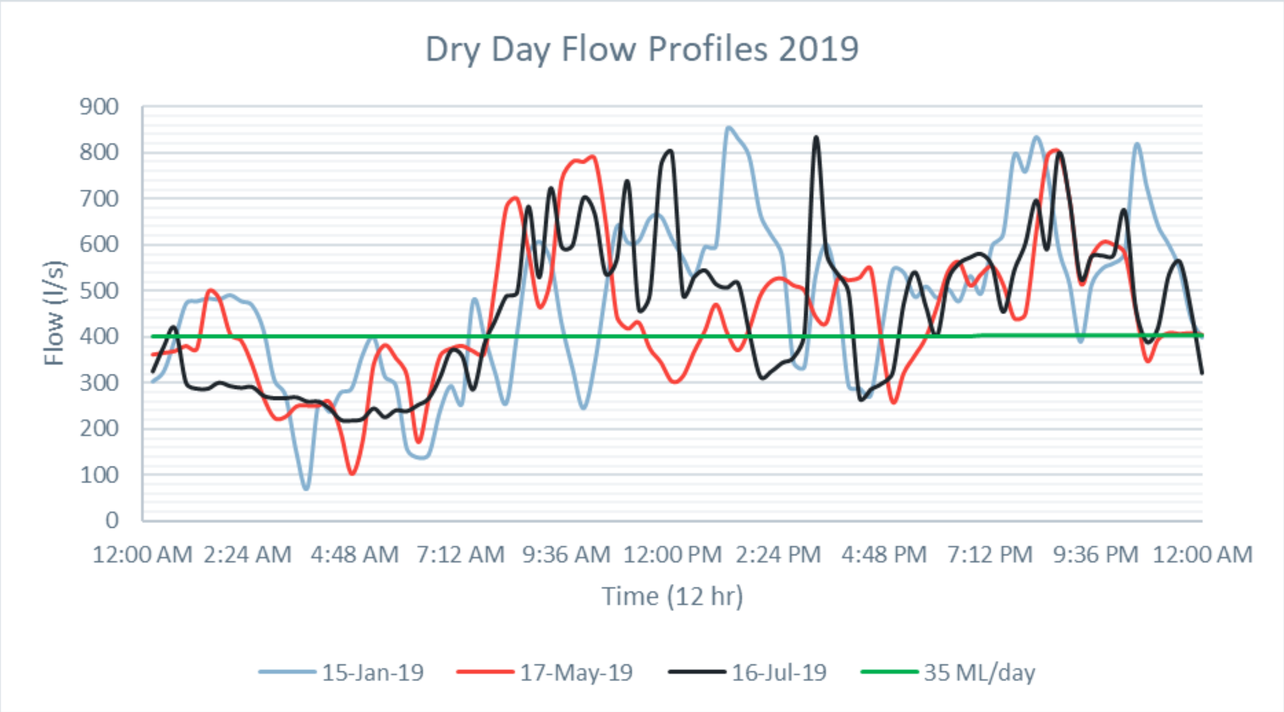


Figure 3-3 - Dry Day Flow Profiles 2020

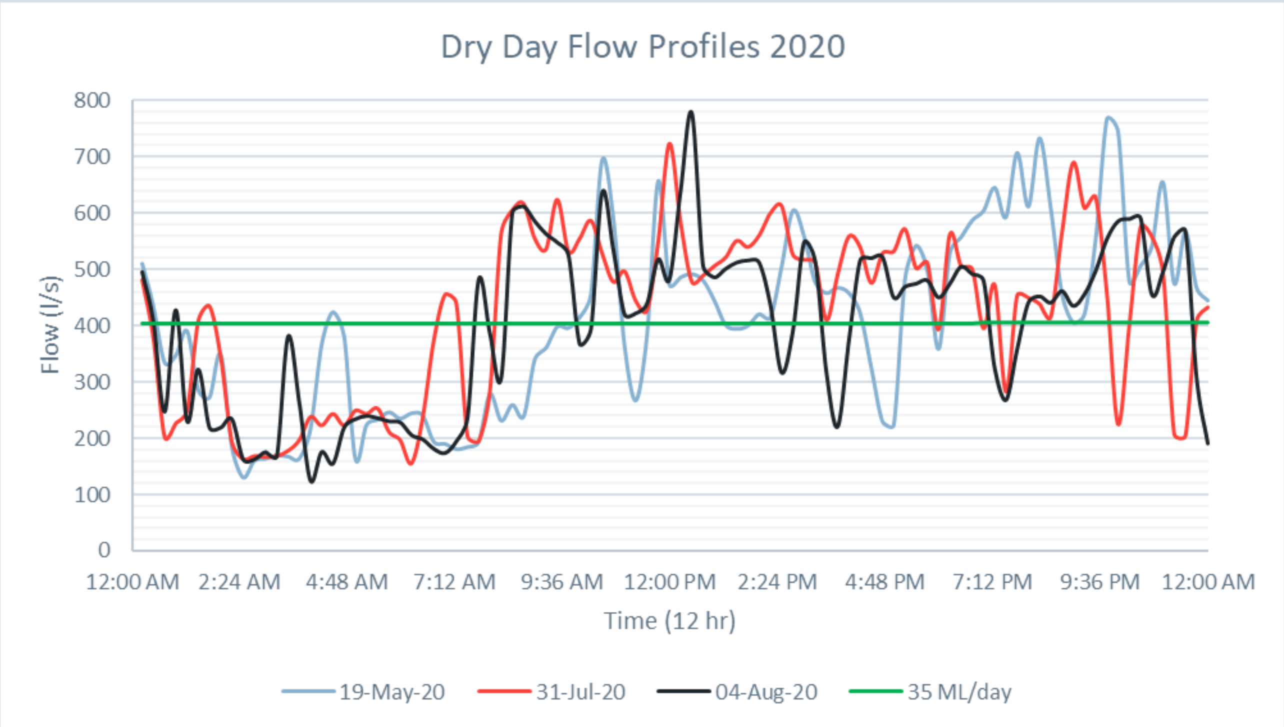
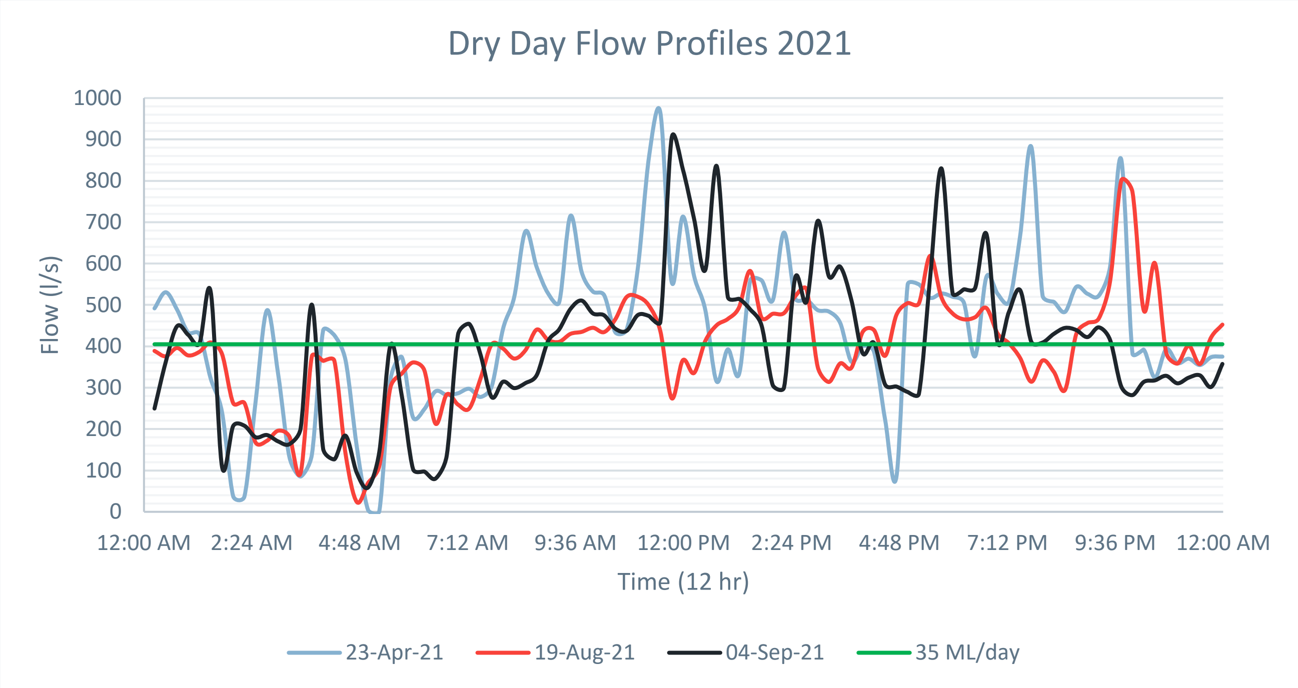


Figure 3-4 - Dry Day Flow Profiles 2021



The cumulative volume for the randomly selected dry days is shown in Table 3-1.

Table 3-1 – Dry day cumulative flow volumes

Date	Cumulative Flow (MLD)
15/01/2019	41
17/05/2019	38
16/07/2019	39
19/05/2020	35
31/07/2020	37
04/08/2020	35
23/04/2021	38
19/08/2021	**34**
04/09/2021	**34**

Table 3-1 shows that of the dry days selected at random, cumulative volumes can decrease below 35 MLD.

The dry day flow was combined to create a ‘dry day diurnal profile’ as shown in Figure 3-5 and Table 3-2. The flows range used in the STT SRO treatment process design was 200 l/s to 550 l/s.

Figure 3-5 - Proposed dry day diurnal profile

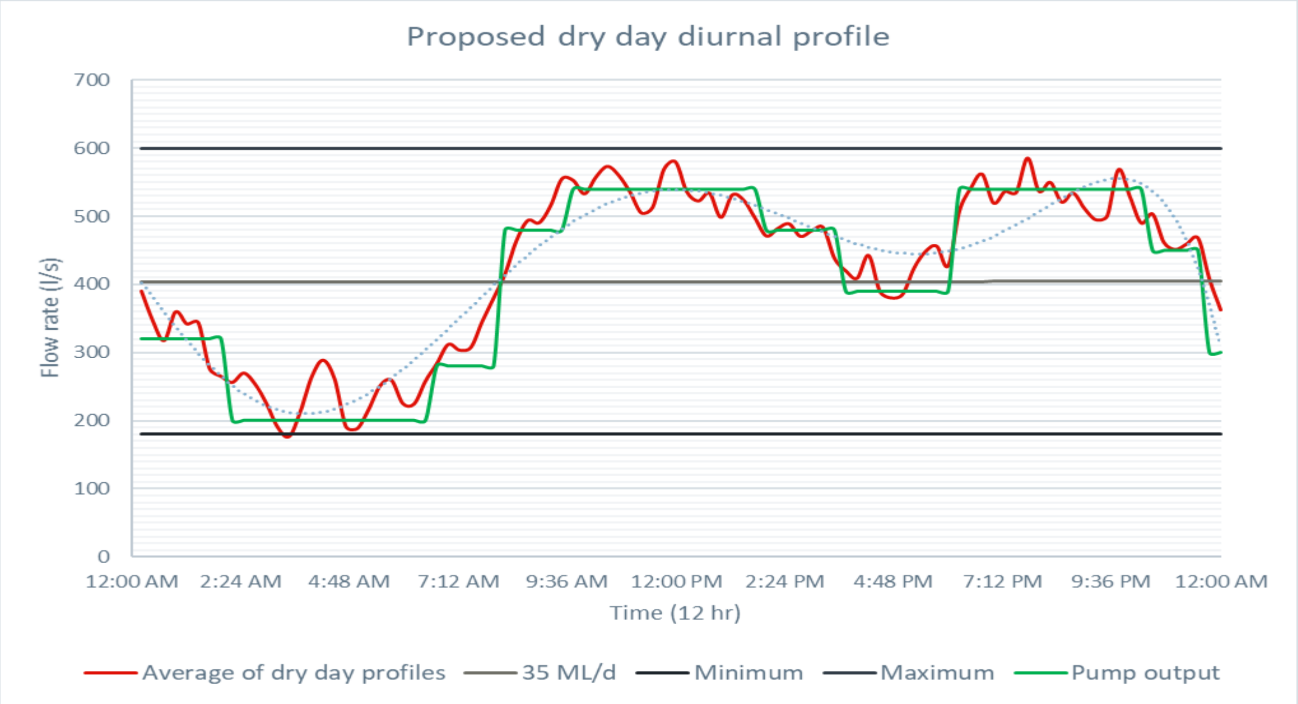


Table 3-2 – Proposed Dry Day Diurnal Profile with Cumulative Volume

Time	Flow (l/s)	Cumulative Volume (MLD)
00:00 – 02:00	320	2.3
02:00 – 06:30	200	5.2
06:30 – 08:00	280	6.7
08:00 – 09:30	480	9.3
09:30 – 12:00	540	14.2
12:00 – 13:45	540	17.7
13:45 – 15:30	480	20.7
15:30 – 18:00	390	24.2
18:00 – 22:15	540	32.7
22:15 – 23:30	450	34.7
23:30 – 00:00	300	35.2

The flow range of 200 l/s to 550 l/s was considered as an acceptable compromise between following a dry day diurnal profile and reasonable flow turn down through process units.

Larger flow operating ranges could require multiple treatment streams for different flow ranges leading to control complications and this drives the requirement for further flow modelling.

Due to losses in the tertiary treatment process (sludge and backwash water for the GAC and BAF units), the actual volume of final effluent requiring diversion for transfer to STT must be greater than 35 MLD. This will increase the set points of the diurnal profile shown in Table 3-2, but based on current losses, this is expected to be approximately 1,540 cubic metres per day (m^3/day) (4.4% of 35 MLD (18 l/s)). Actual losses will be confirmed during detailed design, but because the equipment is designed for flows up to 550 l/s (47.5 MLD), the process will be able to handle the higher flow demand.

3.2.1 BUFFERING CAPACITY

With a 200 to 550 l/s flow range, without any buffering capacity, the number of days where 35 MLD would not have been deliverable increases from a base of 38 days to approximately 122 days (6.8%), an increase of 84 days (4.7%). Further, the number of days where stop-start operation during low inflow would be seen, increases to 985 days (54.6%).

Analysing 15-minute data over the 5-year period showed occasions where the average flow falls below the minimum treatment flow (200 l/s), the frequency and duration of such periods is variable however there are a total of 1,007 (55%) days where this occurs.

The buffering capacity required to maintain the STS SRO treatment process at or above the minimum flow (200l/s) is approximately 9 megalitres (ML), considering all days with recorded final effluent flow greater than 18.28 MLD (200l/s for 24 hours). There are four days where the recorded flow is below this volume; further analysis of the dataset and its context would be required to clarify if these days represent erroneous data or extreme flows during extended dry periods. Excluding all days where flow is below 35 MLD (20 days), a buffer capacity approximately 4 ML may be needed; however, the maximum duration of 'starved' flow operation may impact upon the treatment process operation requiring manual intervention by STW operatives.

With an STS SRO treatment flow ranged from 200 to 550 l/s, and an operational philosophy that follows the final effluent flow, the approximate buffering storage required to limit the number of days where 35 MLD is not delivered, to the base twenty days, would be approximately 1.5 ML. However, it is important to note that such variable treatment flow (as shown by the diurnal flow profile analysis) may not be aligned with the stable operation of the proposed STS SRO treatment process.

Therefore, based upon the Netheridge MCERTS dataset, a combination of final effluent capture and storage ahead of an Netheridge SRO treatment process with a variable flow rate may be necessary to maximise the probability of delivering 35 MLD across varying diurnal profiles recorded between 2017 and 2020.

It is proposed that the availability of final effluent flow is further reviewed at Gate 3 to confirm the viability of the Netheridge STS SRO to provide 35 MLD, and the requirement for storage capacity based on the validity of MCERTS data, which must be verified by STW.

3.3 GROWTH

Flow summary data of the PR24 Design Envelope Confirmation (DEC) received from STW in February 2022 indicates a domestic population increase at Netheridge from 173,118 to 192,583 between 2020 and 2038. The impact on dry weather flow is shown in Table 3-3. This data sheet advises not to apply trade figures as per design manual DM201-01B.

Table 3-3 – Population equivalent increase at Netheridge WwTW and effect on dry weather flow

Parameter	2020	2038	Increase	Percentage Increase
Total Domestic Population (P)	173,118	192,582	19,464	11.2%
Consumption (Cubic metres per person per day (m^3/hd/day)) (G)	0.16	0.16	0	-
Infiltration (m^3/day) (I)	13,029	13,761	732	5.6%
Trade Effluent (m^3/day) (E)	0	0	0	0%
Dry Weather Flow (MLD) (PG + I + E)	40.7	44.6	3.9	9.4%

The increase in population leads to an increase in dry weather flow of 3.9 MLD. To determine how growth will influence the ability to provide 35 MLD, the instantaneous dry weather flow (44.5 l/s) was added to the existing flow data points. Only dry weather flow was considered because the demand for the supplementary flow from Netheridge is likely to be in summer months when the weather is drier. The effect of applying this dry weather flow increase is shown in Table 3-4.

Table 3-4 – Impact of growth on flow at Netheridge WwTW

Item	Existing	2038
Days where flow is less than DWF	580 (32%)	441 (24%)
Days where flow is less than 35 MLD	38 (2.1%)	16 (1%)
Days where flow is less than 20 MLD	4 (0.2%)	3 (0.2%)
Days where minimum instantaneous flow is greater than 405 l/s	391 (22%)	445 (25%)
For flows between 200 l/s and 550 l/s, failure to meet 35 MLD	122 (6.8%)	21 (1.2%)
Buffering Capacity (only for days where flow is greater than 200 l/s for 24 hours)	9 ML	7.2 ML
Buffering capacity (excluding all days where flow is below 35 MLD).	4 ML	2.8 ML

The additional growth halves the percentage of days where flow is less than 35 MLD from 2.1% to 1% for all days analysed and decreases the percentage of days that fail to meet 35 MLD for flows between 200 l/s and 550 l/s from 6.8% to 1.2%. Therefore, the increase in dry weather flow does not completely eliminate the chance of 35 MLD not being achieved over 24 hours.

The number of days where minimum instantaneous flow is greater than 405 l/s increases from 22% to 25%, so a variable feed to the Netheridge SRO treatment is still likely to be required.

.The effect of growth on medial diurnal flow patterns is presented in Figure 3-6.

Figure 3-6 - The effect of growth on medial diurnal flow profiles

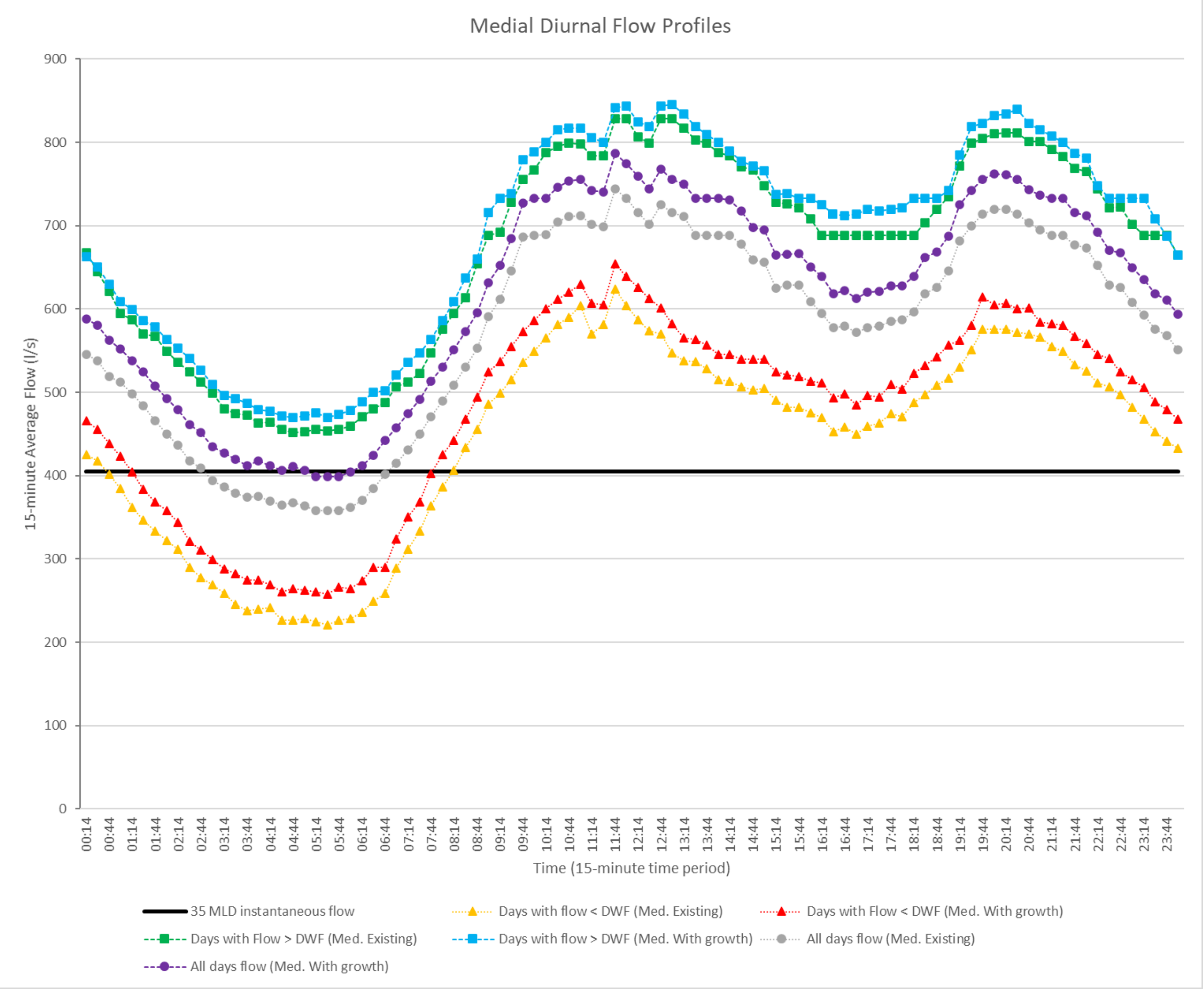


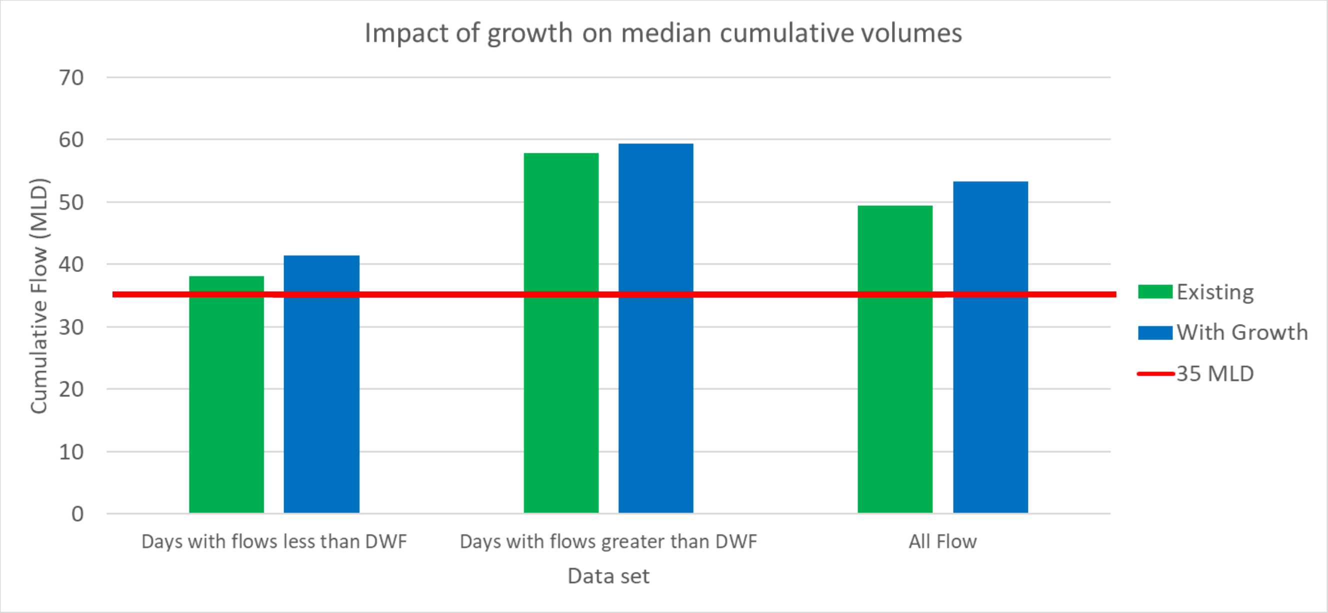
Figure 3-6 compares existing medial diurnal flow data with growth applied to the existing medial diurnal flow. There is a small change to medial flows for days with flow greater than DWF and remains consistent in achieving 35 MLD.

For all flow data, the minimum medial flow increases to 399 l/s, just 6 l/s below the required set point for 45 minutes per day. The cumulative medial total increase from 38 MLD to 50 MLD.

For days with flows less than DWF, the medial flow increases on average by 35 l/s with growth applied, although flow still remains less than the instantaneous requirement to provide 35 MLD during the night (01:00 to 08:00). Therefore, it still remains likely that during summer months when the STS flow demand is likely to occur, a variable flow to the tertiary treatment plant will be required.

The effect that growth has on medial cumulative daily flows is presented in Figure 3-7

Figure 3-7 - Effect of growth on median cumulative daily flow volumes



Under conditions where daily volume is less than dry weather flow, growth increases the median cumulative daily volume from 38 MLD to 41 MLD, adding more confidence towards achieving the 35 MLD target. When growth is applied to all flow data, the median flow increases from 50 MLD to 53 MLD.

In summary, the predicted growth will add more confidence towards achieving 35 MLD, however a variable flow that follows the diurnal flow profile will remain a requirement, because growth too small to guarantee a constant flow of 405 l/s through the works.

4 PROCESS DESIGN BASIS AND ASSUMPTIONS

4.1 RECEIVING WATER STANDARDS

4.1.1 RIVER SEVERN

A preliminary screening exercise has been undertaken of existing Netheridge final effluent discharged to the proposed outfall location at Deerhurst. This data consisted of 14 spot samples of final effluent discharged from Netheridge WwTW taken at monthly intervals between 10/12/2020 and 12/10/2021, except for April 2021 and June 2021 where two samples were taken each month. The existing discharge is into the tidal River Severn whereas the Deerhurst section is fresh water. The permit requirements will therefore be more onerous for discharge into the Deerhurst section. The screening considers 35 MLD transferred as a daily effect, and not the diurnal variation between 200 l/s and 550 l/s required to meet the 35 MLD demand due to the nature of incoming flow at Netheridge.

A total of 36 chemicals of concern in the existing Netheridge final effluent were highlighted in sampling data received at the start of Gate 2 from Atkins that would not meet likely permitting conditions.

The screening results at Deerhurst reduced the number of chemicals requiring removal to five. Of these five, four are micropollutants requiring removal because they are considered an addition of a new substance to the river. These organics are typically used as pesticides or herbicides and given the geography of the surrounding area (arable farmland) it is unlikely that these are absent from the river. It is instead suspected that they have not be detected due to the limit of detection (LoD) of the analytical method. This requires additional discussion with regulatory bodies but, for the purpose of ensuring sufficient treatment to allow discharge at Deerhurst, it is assumed their removal is required. In the absence of a confirmed permit, assumptions have been made with regards to sanitary requirements. The assumed discharge requirements for options 1 and 2 are shown in Table 4-1.

It is further assumed that the discharge requirements to Haw Bridge will be the same as Deerhurst.

Table 4-1 – Assumed permit requirement for discharge options 1 and 2.

Determinant	Assumed permit requirement	Comment
BOD (mg/l)	Unknown	Assumed effluent from the new tertiary treatment process will comply with a tightened BOD permit due to increased treatment and solids capture.
Ammonia (mg/l) (95th percentile)	1	Assumption in the absence of a confirmed permit. Based on the permit of WwTWs (Tewkesbury) discharging into a similar stretch of the River Severn. Netheridge provides a larger contribution than these treatment works, and this value may be liable to change.
TSS (mg/l)	Unknown	No modelling performed. Assumed the new tertiary treatment process will comply with a tightening of the existing TSS permit.

Determinant	Assumed permit requirement	Comment
pH	6-9	Assumed as part of typical permit requirements
Total Phosphorus (mg/l) (12-month average)	0.2	Assumed to prevent impediment towards good water framework directive (WFD) status.
Iron (mg/l) (Maximum)	4	Assumed an iron permit will be imposed because of coagulant dosing for phosphorus removal.
2-4, dichlorophenol microgram per litre (µg/l) (Maximum)	0.02 (Limit of detection)	Chemical not detected in 14 samples taken of the river at the proposed discharge location between 2020 and 2021. Therefore, it has been classified as an addition of a new substance into the river. The target concentration is assumed to be the limit of detection.
Chlorothalonil (µg/l) (Maximum)	0.035 (Limit of detection)	Chemical not detected in 14 samples taken of the river at the proposed discharge location between 2020 and 2021. Therefore, it has been classified as an addition of a new substance into the river. The target concentration is assumed to be the limit of detection.
Nonylphenols (4-nonylphenol technical mix) (µg/l) (Maximum)	0.04 (limit of detection)	Chemical not detected in 14 samples taken of the river at the proposed discharge location between 2020 and 2021. Therefore, it has been classified as an addition of a new substance into the river. The target concentration is assumed to be the limit of detection.
Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol) (µg/l) (Maximum)	0.01 (Limit of detection)	Chemical not detected in 14 samples taken of the river at the proposed discharge location between 2020 and 2021. Therefore, it has been classified as an addition of a new substance into the river. The target concentration is assumed to be the limit of detection.
Perfluorooctane sulfonic acid and its derivatives (µg/l) (Maximum)	0.0002 (Limit of detection)	Addition will cause impediment towards achieving target river status. The required concentration is assumed to be the limit of detection.

The required removal is shown in Table 4-2. BOD and TSS have been removed based on the assumption the tertiary treatment process will achieve any enforced permit condition. Iron has also been removed because it is added to the process in the form of iron-based coagulants and thus meeting the permit will be based on good process control rather than a removal process.

Table 4-2 – River Severn water quality and removal requirements

Determinant	Assumed permit requirement	Netheridge Effluent (Average/ Max)	Required removal (%)
Ammonia (mg/l)	1	1.24 / 5.9	19% / 83%
Total Phosphorus (mg/l)*	0.2	1.2 / 1.98	83% / 90%

Determinant	Assumed permit requirement	Netheridge Effluent (Average/ Max)	Required removal (%)
2-4, dichlorophenol (µg/l)	0.02	0.022 / 0.05	9% / 60%
Chlorothalonil (µg/l)**	0.035	0.035 / 0.035	0% / 0%
Nonylphenols (4-nonylphenol technical mix) (µg/l)	0.04	0.535 / 1.09	93% / 96%
Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol) (µg/l)	0.01	0.011 / 0.02	9% / 50%
PFOS and its derivatives (µg/l)	0.0002	0.0065 / 0.0101	70% / 80%

**Assuming the primary dose will remove 70% of the total phosphorus prior to the new tertiary treatment process. The average total phosphorus concentration in the existing effluent is 4.0 mg/l, maximum is 6.6 mg/l.*

*** The concentration of chlorothalonil in the final effluent at Netheridge was measured at the limit of detection. The requirement to remove chlorothalonil when it is already measured at the limit of detection requires discussion with regulatory bodies.*

It should also be noted that this assumed PFOS permit requirement is less than the 0.01 µg/l specified in the guidance on the Water Supply (Water Quality) Regulations 2016 specific to PFOS and perfluorooctanoic acid (PFOA) concentrations in drinking water. This should also be discussed with regulatory bodies.

With regards to treatment, no treatment is 100 % selective and so other chemical species will be removed with any installed process.

4.1.2 EAST CHANNEL OF THE RIVER SEVERN

For discharges into the East Channel, no screening exercise or dilution modelling has been undertaken, and no specific permit requirements defined. Therefore, it is assumed the same sanitary permit requirements for the River Severn will apply for discharge to the East Channel (Table 4-3).

Table 4-3 – East Channel Sanitary Permit Requirements

Determinant	Assumed permit requirement	Comment
Total Phosphorus (mg/l) (12 month average)	0.2	Assumed
Ammonia (mg/l) (95^{th} percentile)	1	Assumed
BOD (mg/l)	Unknown	Assumed that the proposed tertiary treatment processes will comply with any BOD permit requirement imposed

TSS (mg/l)	Unknown	Assumed that the proposed tertiary treatment processes will comply with any BOD permit requirement imposed

The Netheridge final effluent sampling data received in October 2021 and East Channel environmental quality standards (EQSs) indicated additional metals and micropollutants that will not meet likely discharge permit conditions without additional treatment. These micropollutants and the required removal percentage are shown in Table 4-4.

Table 4-4 – East Channel Assumed Micropollutant Water Quality Requirements

Determinant	Limit of detection	East Channel Long Term Mean Target	East Channel Short Term Maximum Target	Netheridge effluent Average	Netheridge effluent maximum	Required percentage removal (mean)	Required percentage removal (maximum)
2,4-dichlorophenol (µg/l)	0.02	0.02	0.02	0.022	0.05	9%	60%
Chlorothalonil (µg/l) *	0.035	0.035	0.035	0.035	0.035	0%	0%
Nonylphenols (4-nonylphenol technical mix) (µg/l)	0.04	0.04	0.04	0.54	1.09	93%	96%
Perfluorooctane sulfonic acid (PFOS) and its derivatives (µg/l)	0.0002	0.00065	0.0033	0.0065	0.0101	90%	67%
Chromium (III) dissolved (µg/l)	1	1.018	1.2	1.60	7.9	36%	85%
Glyphosate (µg/l)	0.1	0.17	0.37	0.79	1.3	78%	72%
Mecoprop (µg/l)	0.02	0.025	0.06	0.12	0.3	79%	80%
Permethrin (µg/l)	0.001	0.001	0.003	0.002	0.005	50%	40%
Triclosan (µg/l)	0.01	0.01	0.01	0.02	0.04	50%	75%
Cypermethrin (µg/l)	0.00008	0.00008	0.00028	0.00019	0.0005	58%	44%
Dichloromethane (µg/l)	1	1	Not applicabl e (N/A)	2	5	50%	N/A
Hexabromocyclododecane (HBCDD) (µg/l)	0.00014	0.000147	0.00018	0.0008	0.00185	82%	90%
Lead dissolved (µg/l)	0.09	Annual average EQS requires calculation using BLM tool. Not currently assessed.	2.5	0.69	2.9	N/A	14%
Mercury dissolved (µg/l)	0.001	n/a	0.04	0.01	0.052	N/A	23%

Determinant	Limit of detection	East Channel Long Term Mean Target	East Channel Short Term Maximum Target	Netheridge effluent Average	Netheridge effluent maximum	Required percentage removal (mean)	Required percentage removal (maximum)
Nickel dissolved (µg/l)	0.5	Annual average EQS requires calculation using BLM tool. Not currently assessed.	2.5	2.63	3.9	N/A	36%
Pentachlorophenol (µg/l)	0.02	0.02	0.02	0.023	0.03	13%	33%
Terbutryn (µg/l)	0.02	0.02	0.02	0.03	0.07	33%	71%
Tributyltin compounds (as tributyltin cation) (µg/l)	0.00003	0.0000482	0.00012	0.00005	0.00011	4%	0%
Boron total (µg/l)	12	59	N/A	127	170	54%	N/A
Chloride (mg/l)	0.1	42	N/A	82	140	49%	N/A
Dibutyl phthalate (µg/l)	0.02	0.02	0.02	0.03	0.1	33%	80%
Diethyl phthalate (µg/l)	0.02	0.02	0.02	0.02	0.02	0%	0%
Diflubenzuron (µg/l)	0.001	0.001	0.001	0.001	0.001	0%	0%
EDTA (µg/l)	100	100	100	166	337	40%	70%
Fluoride (mg/l)	0.01	0.162	0.22	0.163	0.20	1%	0%
Mancozeb (µg/l as carbon disulphide (CS_2))	0.1	0.1	0.1	0.21	0.70	52%	86%
Maneb (µg/l CS_2)	0.1	0.1	0.1	0.14	0.50	29%	80%
Sulphate (mg/l sulphate (SO_4))	0.1	57.4	N/A	101	140	43%	N/A
Tributyl phosphate (µg/l)	0.02	0.02	0.02	0.08	0.61	75%	97%
Triphenyltin (TPT) compounds (as tryphenyltin cation) (µg/l TPT)	0.002	N/A	0.002	0.003	0.02	N/A	90%

**Further clarification is required to confirm removal requirements if these micropollutants are already at the limit of detection.*

4.1.3 GLOUCESTER AND SHARPNESS CANAL

The Gloucester and Sharpness Canal is a drinking water protected area (DWPA). Water is abstracted from the canal by Bristol Water for treatment at Purton water treatment works (WTW) approximately 19.5 km downstream of Netheridge WwTW.

Engagement between STW and the DWI/EA has begun, but there has been no confirmed water quality requirement. Therefore, it is assumed that the same ammonia and total phosphorus permit requirements will apply, as will a robust treatment process to remove micropollutants, similar to the East Channel treatment proposal. This could change depending on the results of environmental modelling, assessment of water safety planning risks for Purton WTW and permit discussions with regulatory bodies. A requirement to provide disinfection is also assumed (but must be confirmed) to comply with drinking water protected area discharge requirements.

4.2 BASIS OF DESIGN ASSUMPTIONS

The assessment of environmental permit conditions for the Netheridge and wider SRO projects is being undertaken by an independent consultant. Several discussions have taken place with the EA and SRO partners, but the exact permit and discharge parameters will not be determined until after the design report is completed. The assumed required permit conditions are detailed in section 4.1.

In the absence of confirmed permit requirements, several design assumptions, based largely on worst-case scenarios, have been made to enable the concept design to be progressed:

- Any new biological treatment and tertiary solids removal processes will require continuous flow.
- The range (minimum, average and maximum) of ammonia concentrations in the existing final effluent will be used to size the biological treatment stage required for tertiary ammonia removal.
- Nitrification will be provided to achieve a 95th percentile ammonia concentration of 1 mg/l regardless of the discharge location.
- Two-point iron based coagulant dosing is required to achieve an assumed 0.2 mg/l total phosphorus consent for the effluent provided to supplement water abstracted for STT.
- The tertiary solids removal process, as proposed at Gate 1, will be carried forward as the tertiary phosphorus process (for costing purposes). Alternative processes are offered in the relevant sections.
- The tertiary solids removal process will require potable water for polymer make up which will be available on site.
- BOD, TSS and Total Iron permit requirements are achievable through the provision of a tertiary solids removal process.
- Advanced treatment processes will be used to remove organics and PFOS for discharges to the River Severn.
- Discharges to the East Channel and Gloucester and Sharpness Canal will require metals removal in addition to organics and micropollutants.
- UV disinfection will be required for discharge to the Gloucester and Sharpness Canal in order to comply with drinking water abstraction standards.
- Proposed thermal hydrolysis process (THP), DWF and phosphorus removal projects for the existing treatment process have been ignored in the development of treatment options in this report due to the uncertainty of their progression at the time of writing. The progress of these proposed projects must be reviewed at Gate 3 to ensure a holistic approach to all improvement projects at Netheridge WwTW is undertaken. A potential DWF project highlights hydraulic capacity in the existing works may be an issue.
- The existing SAS thickening plant can accommodate the additional sludge produced as part of ferrous sulphate dosing into the ASPs.

- A new sludge thickening plant will be provided to thicken sludges from new treatment processes installed as part of the STS SRO project.
- Flow will range around a diurnal profile that is configured on the 'typical' flow profile and delivers 35 MLD.
- Flow to SRO treatment will range from 200l/s to 550l/s.
- There will be adequate storage of final effluent (FE) to enable stable and consistent flow particularly when the final effluent flow falls below 200l/s.

4.3 OPERATING PHILOSOPHY

As per Section 1.2, the operational control will be based on the STT SRO requirements:

- 17 days' notice before transfer.
- 20 days minimum transfer of flow.
- Sweetening flow of 20 MLD.
- Full transfer flow of 35 MLD.

The Gate 2 concept design is based upon Netheridge SRO operating criteria outlined in section 1.2, based upon the STT SRO requirements:

- The STT SRO scheme will provide at least 17 days' notice of the intent to begin transfer of flow to the River Thames.
- The STT SRO will operate for a minimum of 20 days once fully operational.
- The STS SRO will be dispatched at 35 MLD when the STT SRO scheme is operational.
- The STT SRO scheme will provide at least 17 days' notice of the intent to dispatch 20MLD sweetening flows.
- The STS SRO will be dispatched at 20 MLD when the STT SRO is NOT operational but when levels in the River Severn are below 'hands off' flow' (HOF) and 'sweetening' flows from the River Severn cannot be abstracted without augmentation from other sources.

The operation of the Netheridge SRO treatment will also adhere to the following constraints that are a consequence of the Netheridge WwTW flow profile or proposed treatment technologies:

- The Netheridge SRO treatment will remain in continuous operation to maintain the treatment biospheres in readiness (the option to shut down the treatment processes has been discarded at Gate 2 concept design by STW; this constraint should be reviewed at Gate 3).
- The Netheridge SRO treatment is sized for an average diurnal profile and would require final effluent flow to remain within a 200l/s to 550l/s range, excepting for short sub-200l/s periods.
- Variable speed pumps will be installed to ensure that flow can be controlled to each process and stable operation is achieved.
- The Netheridge WwTW final effluent flow follows a varying and unpredictable diurnal profile. There is a risk the Netheridge SRO treatment could require adequate buffering storage of final effluent to operate to a fixed diurnal profile (this constraint should be investigated at Gate 3 in more detail).
- During periods when the STT SRO is not dispatched (transfer or sweetening) the treated final effluent will be returned to the Netheridge WwTW outfall channel for discharge.

- During periods when the STT SRO is not dispatched (transfer or sweetening) the Netheridge SRO treatment will operate at its minimum flow of 200 l/s or 17.3 MLD.
- Both the pre-ASP ferric dosing and the STS final-effluent treatment will operate continuously to maintain the treatment units in operational readiness (e.g., the biospheres).

4.3.1 RISKS

4.3.1.1 17 Day start up period

The technologies presented should be capable of increasing flow throughput during this 17-day period, however the flow should be increased incrementally to prepare the treatment for full flow. This will shorten the period for performance testing at full flow to 14 days.

There is also a risk regarding loading during this start up period. In theory, by increasing the flow by 1.75 times, loads should increase by 1.75 times, which could cause process upset if the system is overloaded quickly. Conversely, there is also a risk that the loads required to test the system at maximum loading are unavailable, so performance validation at full loading cannot be confirmed prior to transfer. This could also have implications on procurement items, such as contractual arrangements and performance guarantees.

Equipment failure, chemical availability and process instability all pose risks to achieving water quality requirements and performance validation. It is recommended further work at gate 3 is undertaken to develop a robust and rigorous strategy to increasing flow and validating performance.

5 TREATMENT APPROACH FOR RIVER SEVERN (OPTIONS 1 AND 2)

This section describes the basis of design and treatment process for discharge to the River Severn at Deerhurst and Haw Bridge. It is assumed the permit requirements are the same for both discharge locations as discussed in Section 4.

5.1 PRIMARY PHOSPHORUS REMOVAL

5.1.1 FERROUS SULPHATE

To achieve the assumed 0.2 mg/l total phosphorus permit, two-point chemical dosing is required in accordance with STW design standards to reduce the risk of non-compliance with a low permit requirement. The first dose removes the majority of the phosphorus and the second (upstream of the tertiary solids removal process as described in section 5.3) to trim the phosphorus removal to the required limit.

The Gate 1 option to dose ferrous salts into the ASP has been carried through to Gate 2. The oxygen available in the ASP oxidises the ferrous ions to ferric ions which then act to coagulate solids. The oxygen demand is so small that it can be ignored according to STW standards. The advantages of using ferrous over ferric salts include a lower oxygen load on the ASP and cheaper whole life costs compared to ferric salts. The Gate 1 report states that a well-run ASP and clarification system can achieve a 1 mg/l total phosphorus concentration when operated in this arrangement.

To comply with the STW design manual (DM0201-05A), ferrous sulphate will be dosed into the recycled activated sludge (RAS) stream (Figure 5-1) rather than into the effluent post primary settlement to avoid intrusive work and the potential for temporary treatment. Dosing directly into the turbulent RAS stream, together with vigorous mixing in the ASP, removes the requirement for rapid mixing and reduces the magnitude of competing reactions that could occur when dosing upstream of the primary settlement tanks. By dosing into the RAS stream, the iron dose is better focused on targeting phosphorus. Ferrous sulphate is delivered as a solid, and so will require saturators to facilitate the dose, but is cost effective when dosed into sites serving a population equivalent greater than 100,000 PE.

The ferrous sulphate control will be flow paced to deliver an assumed mass of iron which can be optimised during commissioning with the support of jar testing. The requirement of 1,830 kilograms per day (kg/day) has been derived using the following parameters:

Table 5-1 – Primary Ferrous Sulphate Design Parameters

Parameter	Value	Comment
Flow to ASP lanes	52,045 m^3/day	Average MCERTS flow between 2017 and 2021
Phosphorus Load	377 kg/day	From 2020 population equivalent data. Assuming no removal in the PSTs.

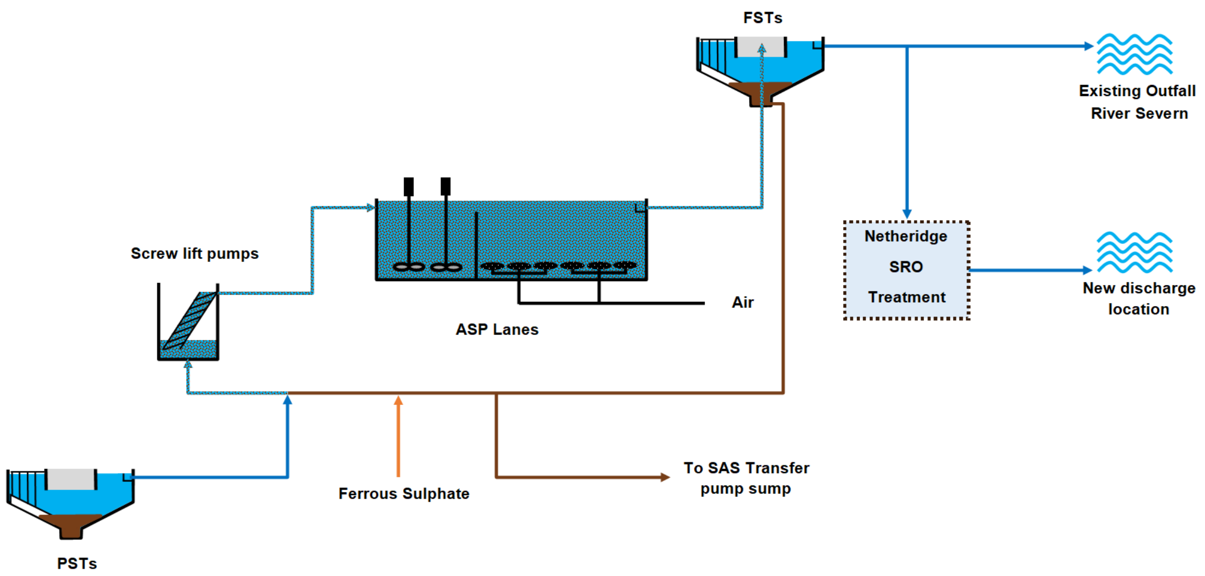
Parameter	Value	Comment
Iron requirement	1.8 g/g Phosphorus	From STW design specifications
Mass of iron required	679 kg/day	
Percentage of iron in product as delivered	37%	
Mass of ferrous sulphate per day	1,830 kg/day	
Storage requirement	28 days	
Storage volume	14 cubic metres (m^3)	
Increase in SAS production	20%	From STW design specifications
Additional surplus solids	1,454 kg/day	Additional SAS load
Assumed dilution for dosing (wt.%)	50%	Assumption
Potable water requirement (kg/day)	1,982 kg/day	

Section 4.1.1 explains that 70% of the crude total phosphorus will be removed by primary chemical dosing. The parameters listed in Table 5-1 have been derived based on 100% of incoming total phosphorus for high level optioneering and costing.

The primary ferrous sulphate dose will remain online even when there is no demand from STT to not shock the biological activity in the ASP which will have become accustomed to ferrous sulphate addition.

The ferrous sulphate dose may change depending on jar testing results and site performance during commissioning optimisation.

Figure 5-1 - Ferrous sulphate dosing schematic



5.1.2 RISKS

5.1.2.1 Additional solids production

Dosing into the ASP will increase the percentage of inert solids in the mixed liquor, usually by 10%. The mixed liquor suspended solids (MLSS) will need to be increased accordingly in order to maintain the same treatment level. A simple analysis on the impact of ferrous sulphate dosing on FST hydraulic and solids loading based on the theoretical requirement is shown in Table 5-2.

Table 5-2 – Changes to FST hydraulic and solids loading due to ferrous sulphate dosing

Parameter	Existing	Change due to ferrous sulphate dosing
MLSS (mg/l)	2,660 mg/l	2,926 mg/l
Hydraulic loading rate at average flow	0.9 cubic metres per square metre per hour ($m^3/m^2/hr$)	1.0 ($m^3/m^2/hr$)
Hydraulic loading rate at full flow to treatment (FFT)	1.4 ($m^3/m^2/hr$)	1.5 ($m^3/m^2/hr$)
Solids loading rate at average flow	2.5 kilograms per square meter per hour ($kg/m^2/hr$)	2.8 ($kg/m^2/hr$)

The existing ASP FSTs and SAS handling facilities can be modelled in more detailed in Gate 3 to determine the suitability of dosing into the RAS stream.

5.1.2.2 pH

Chemical phosphorus removal depletes alkalinity and potentially reduces pH which could impact on biological performance.

5.1.2.3 Diffuser Fouling

Chemical dosing into the ASP can increase diffuser fouling. This will impact on treatment performance and potentially the final effluent ammonia concentrations. The diffuser service life may also be reduced.

5.1.2.4 Potable water

Potable water is required for the ferrous saturator. The existing potable water supply struggles to meet site demand and can restrict the domestic supply locally. If ferrous sulphate is to be used, then the works potable water supply will require an upgrade.

5.1.2.5 Ferrous Sulphate handling hazards

Ferrous sulphate may cause skin and eye irritation and if inhaled can cause headaches, nausea and respiratory irritations due to hydrolysis to form sulphuric acid. Fumes or mists may cause irritation to or burns to skin.

Eye wash and safety showers are included as part of proposal.

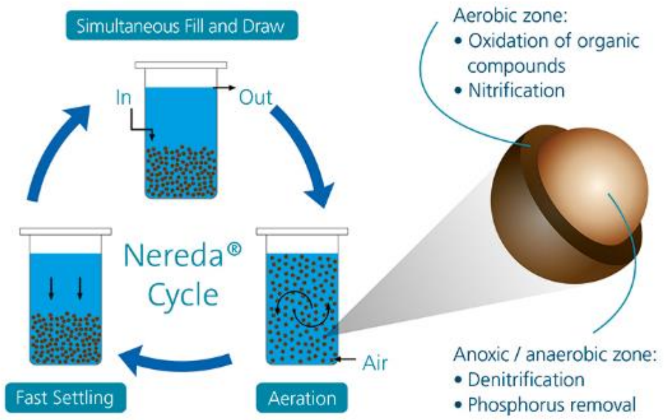
5.1.3 OPPORTUNITIES

5.1.3.1 Upgrading the existing ASP to permit phosphorus removal

Nereda®

A biological phosphorus removal technology that has been applied at other STW assets is Nereda®. Nereda® is a granular biomass process that operates like a sequential batch reactor (SBR) comprising a three-step cycle: simultaneous fill and draw, aeration, and fast settling. The purifying biomass grows naturally as a compact aerobic granular sludge with very good settlement properties. Within the granules, aerobic and anoxic/anaerobic zones co-exist which enable nitrification, denitrification and biological phosphate removal to occur simultaneously. How the Nereda® process works is depicted in Figure 5-2.

Figure 5-2 - Simplified diagram of the Nereda® process (Royal Haskoning DHV)



The short period between initiating settlement and sludge draw-off means the desirable fast settling granules are retained whilst the granules with poor settlement properties are removed and consolidated in a sludge tank.

The advantages of an Nereda® process are the small footprint (25% of conventional biological nutrient removal (BNR) activated sludge processes) or in the case at Netheridge, the ability to increase treatment capacity whilst maintaining the same footprint. If a phosphorus consent is imposed on the existing discharge to the River Severn, biological phosphorus removal could become an economically attractive alternative considering the cost and diminishing availability of iron-based coagulants. Conversion of the existing ASPs into a Nereda® process would increase treatment capacity and remove the requirement of downstream clarifiers. The redundant FSTs could be utilised to provide additional storm capacity.

The developers of Nereda® (Royal Haskoning DHV) also provide resource recovery processes that can be coupled with Nereda®. One recovery process can harvest a biopolymer from the Nereda® granules to replace fossil-based material, and another for phosphate recovery.

Nereda® on its own can be used to achieve a total phosphorus concentration of 1 mg/l. Combined with a tertiary solids removal unit, a 0.5 mg/l permit can be achieved. With coagulant dosing and tertiary solids removal unit, 0.2 mg/l total phosphorus permits can be achieved (according to supplier website). A trim dose of ferric sulphate may still be required to achieve the low phosphorus concentration.

BioMag®

BioMag® was installed at a wastewater treatment works in North America to achieve a total phosphorus concentration of less than 0.2 mg/l. The BioMag® system infuses magnetite as a weighting agent into biological floc which achieves rapid and reliable settling (settled sludge volume index (SSVI) of less than 50). The high specific gravity and strong affinity for biological solids means the settlement rate can be increased and provide an increase in biological capacity (MLSS) of two to three times leading to a more robust process within the same footprint.

BioMag® was originally discounted for Netheridge because there would be a requirement to convert all of the ASP lanes, meaning all 115 MLD (FFT) would be treated proving more expensive than providing a system that can be operated intermittently for only 35 MLD. However, if most of the tertiary treatment processes need to remain online, conversion of the ASP process to BioMag® would provide future proofing against any potential total phosphorus permit imposed on the existing process and potentially eliminate the need for CoMag™ further downstream

The BioMag® process could require a trim dose of ferric sulphate to achieve the low total phosphorus concentration. A system diagram of BioMag® is shown in Figure 5-3.

Figure 5-3 – BioMag® System Diagram

5.1.4 ALTERNATIVE CHEMICALS

5.1.4.1 Ferrous chloride

Ferrous chloride can be dosed into the ASP in lieu of the ferrous equivalent removing the need for saturators and a potable water supply. Ferrous chloride has an iron concentration of 8.5 to 13.4% as delivered and so transport based financial and carbon costs are greater. Ferrous chloride presents a series of health and safety risks because it is corrosive and high-level exposure can cause discolouration of the eyes and damage to the liver. Unlike ferrous sulphate, ferrous chloride is not compatible with 304/316 stainless steel.

5.1.4.2 Ferric sulphate

Ferric sulphate can be dosed into the crude sewage upstream of the PSTs. The advantage is this will allow the use of a single phosphorus removal chemical on site (ferric sulphate is used for tertiary phosphorus removal process). Dosing into the crude may reduce BOD loading onto the ASPs, increasing the aeration capacity to provide additional ammonia removal potentially removing the need for the tertiary MBBRs (subject to modelling), may have a beneficial impact on FST solids handling capacity and optimise the sludge age to improve biological performance. Dosing into the RAS stream has been preferred in this instance due to the potential for competing reactions to occur when dosing raw sewage, which could lead to an increased coagulant dose to achieve the same degree of phosphorus removal.

The additional sludge produced would be removed by the PSTs and may increase the potential biogas production in the anaerobic digestion plant on site.

5.1.5 CARBON IMPACT

With respect to carbon emissions, powdered ferrous sulphate has been proposed to minimise vehicle deliveries to site. Applying a low carbon phosphorus treatment process for primary phosphorus removal would be difficult to incorporate; particularly due to the land required, lack of redundant equipment that could be utilised and need to treat all flow (up to 115 MLD).

Carbon impact could be minimised if the ASP lanes were converted into a biological nutrient removal process. This could reduce the need for tertiary ammonia and phosphorus removal. The phosphorus captured in the biological sludge could be recovered as described in 5.1.3.

The carbon impact could also be further minimised by applying the primary phosphorus dose upstream of the PSTs. This would remove some of the organics prior to ASP and potentially increase the available treatment capacity of the biological process and contribute towards future proofing against any tightening of BOD or ammonia permits without the need to invest in the provision of additional capacity.

5.1.6 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

- Confirm total phosphorus permit requirements with regulatory bodies.
- Model the existing ASP and FST processes to confirm in more detail the impact of ferrous sulphate dosing into the RAS stream.
- Undertake a detailed review of existing SAS handling capacity and confirm available capacity.
- Detailed alkalinity consumption model encompassing future growth.
- Quantify the existing potable water supply and available capacity.
- An assessment of the existing primary sludge handling capacity.
- Review the benefit dosing upstream of the PSTs could offer with regards to ammonia loading on the existing ASPs. Dosing upstream of the PSTs could reduce BOD loading onto the ASP, increasing the aeration capacity for improved ammonia removal, and help to optimise the sludge age for improved biological performance. A long sludge age can lead to secondary releases of phosphorus and lower food to mass ratios, decreasing the treatment capacity. However, dosing upstream of the PSTs could lead to an increased coagulant dose to achieve the required phosphorus removal, given the complexity of competing reactions with organics. Primary sludge handling should also be reviewed.
- Review the progress of the proposed THP project with STW, and what impacts dosing chemical upstream of the PST for phosphorus removal would have on SAS volatile solids destruction as

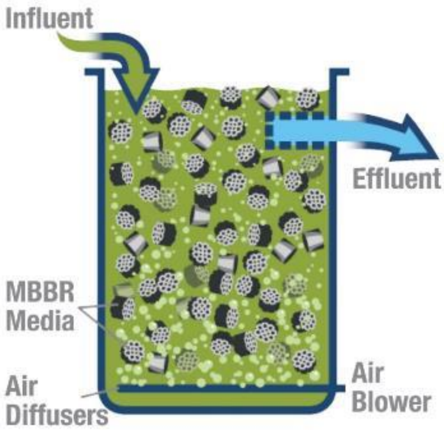
part of a holistic approach to improvement projects at Netheridge WwTW. The provision of a THP could require a liquor treatment plant (LTP) to remove ammonia, and therefore remove the requirement for a tertiary nitrification process.

5.2 AMMONIA REMOVAL

5.2.1 MOVING BED BIOFILM REACTORS (MBBR)

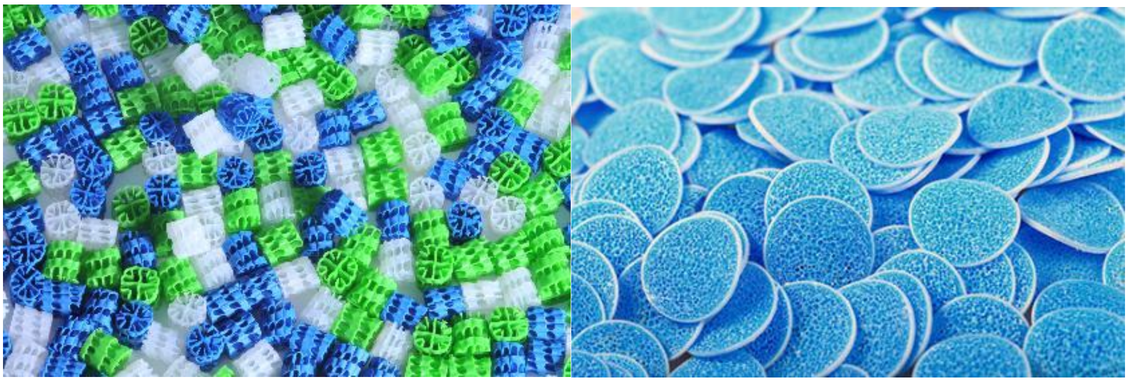
Feed to 5no. MBBR units will be provided by a set of pumps fed from the final effluent flume at a flow between 200 l/s and 550 l/s depending on water availability. The final effluent will enter the MBBR distribution chamber and be evenly distributed across the units which will provide ammonia removal to achieve the assumed 1 mg/l permit requirement. The MBBR process is portrayed at high level by Figure 5-4.

Figure 5-4 - MBBR Details (Bioetp)



MBBR uses thousands of plastic carriers (media) which occupy between 50 and 70% of the tank volume and support aerobic biofilm accumulation to metabolise the ammonia. The media density is similar to that of water to ensure good mixing through the fluid. Examples of MBBR media are shown in Figure 5-5

Figure 5-5 - Examples of MBBR Media (Lenntech / ecologix systems)



Blowers provide air to the coarse bubble aeration grid to mix the tank contents, keep the media moving and provide oxygen. The blower operation will be controlled by dissolved oxygen monitors in each basin with a minimum set speed to maintain the media in suspension. Prior to leaving the tank, the treated water will pass through a sieve to retain the media. MBBRs occupy a smaller footprint compared to traditional biological treatment systems due to the maximised surface area of the media permitting biological growth.

Table 5-3 – MBBR Design Parameters

Parameter	Value	Comment
Flow	1,458 cubic metres per hour (m^3/hr) (avg) / 1,980 m^3/hr (peak)	
Ammonia permit condition	1 mg/l	Assumption
Ammonia target concentration	0 mg/l	
Feed ammonia concentration	1.24 mg/l (avg). 5.9 mg/l (max)	Existing sampling data used.
BOD mass requiring removal	455 kg/day	Using maximum BOD concentration and average flow
Ammonia mass requiring removal	206 kg/day	Using maximum ammonia concentration and average flow
Actual Oxygen Requirement (AOR)	1,203 kilograms of oxygen per day (kgO_2/day)	
Standard oxygen transfer requirement	2,022 kgO_2/day	
Blower requirement	7,814 m^3/hr	
Blower arrangement	Duty/ Assist/ Standby	
Retention time at full flow to treatment	2 hours	Assumption
Volume required	3,960 m^3	To provide retention time at FFT.
Number of reactors	5	
Top Water Level	5 metres (m)	Assumption
Tank width	8 m	
Tank Length	20 m	

When there is no demand from STT, the MBBR process would have to remain online. This is a STW preference, as there is a risk that the 17-day notice requirement is too short to bring the process online ready for transfer from an offline state.

An advantage of the MBBR is the ability to adjust to varying flows and loads. This makes it an appropriate solution for the design flow variation (200 l/s and 550 l/s) in order to achieve a cumulative 35 MLD to Deerhurst.

The MBBRs will be positioned on the available land to the west of the FSTs. The area of land between the storm tanks and FSTs was deemed unsuitable given footprint restrictions, buried services and the requirement to remove a large amount of earth to level the ground.

5.2.2 RISKS

5.2.2.1 Ammonia loading

The MBBR units are sized to achieve a 2-hour retention time at FFT. Given the performance of the existing ASP process achieving low ammonia concentrations, there is a risk the ammonia loading onto the MBBR process is too low to develop a healthy biomass.

5.2.2.2 BLOWER TURNDOWN

There is a risk that the blower turndown is not possible given the range of flows and low loading as a result of the current ASP performance. The blower arrangement may require more than one assist and should be reviewed at the next stage of the design process using additional data collected on the existing ASP performance.

5.2.3 OPPORTUNITIES

5.2.3.1 Existing ASP performance

Whilst the existing ASP process has proven to comfortably achieve the current 15 mg/l permit, the ASP process is not designed to achieve a 1 mg/l permit. Therefore, once any projected growth materialises, there will be a reduction in ammonia removal, and final effluent ammonia concentrations may approach 15 mg/l. The MBBR process is provided for robustness and future proofing, but there may be opportunity to defer construction/ commissioning until the requirement materialises.

5.2.4 ALTERNATIVE TECHNOLOGIES

Alternative compact processes

Alternative compact processes include BAF and SAF plants. The former is not considered sufficiently robust at this stage of the process for the variation in flow required although suppliers may take a different view. At this scale, SAF plants are unlikely to be viable.

5.2.4.1 Upgrade the existing works

To save on construction costs, the existing ASP lanes could be upgraded with more efficient diffusers (and potentially larger blowers) to improve oxygen transfer and ultimately increase the capacity for nitrification. As discussed in section 5.1.4, dosing ferric sulphate into the crude sewage could reduce organic loading onto the ASPs, increasing the capacity for nitrification. These interventions could also provide future proofing against a tightening of the existing ammonia permit.

5.2.4.2 Tertiary Nitrifying Filters

Tertiary nitrifying filters could be provided as a low carbon option to achieve the proposed ammonia concentration. The pumped effluent will flow downwards through the media usually blast furnace slag media although plastic media provides another option. They do not require mechanical aeration only a passive aeration system. Given the requirement to maintain flow through the biological treatment process, tertiary nitrifying filters could provide an attractive low carbon alternative.

5.2.4.3 Floating Wetlands

Floating wetlands are a low carbon technology used to remove contaminants and nutrients. Floating wetlands rely on natural processes to biologically filter water as it passes through the long roots of the floating islands that spread down into the water depth to create a dense column with large surface area. The ammonia is taken up by microbes and plants, but also by the biofilm that develops on the roots. The aquatic plants above and below the surface take up and remove these elements into the plant or material biomass.

Floating wetlands do not clog as conventional constructed wetlands do which eases operation and maintenance requirements. Floating wetlands also bring biodiversity benefits as well as potential social benefits such as cycle paths bird watching activities and educational centres.

The flow pattern through floating wetlands will need to be considered carefully to avoid bypassing or short circuiting. The passive process means it is difficult to apply subtle operational changes to optimise performance.

The anticipated ammonia removal performance at a floating wetland site used for the treatment of water abstracted from a river prior to a water treatment works is 16% to 58% which. At the current ammonia concentrations in the final effluent, this is acceptable, but not if the concentrations approach 15 mg/l. In addition, it is anticipated a nitrate removal of 23-60% and TSS removal of 50% could be achieved.

Evidence within literature has demonstrated beneficial reductions in PFAS and pesticides as well as BOD and chemical oxygen demand (COD).

The floating wetlands also require a large footprint which may not be available within the existing site boundary.

5.2.5 CARBON IMPACT

The MBBR process has been chosen for costing because it is a robust process proven to achieve low ammonia concentrations. However, it does have a large carbon impact when compared to biological trickling filters. If after Gate 3 investigations into the existing ASP it is proven that tertiary ammonia removal is required, biological filters should be considered to remove the requirement for typically energy inefficient aeration, providing they can achieve the ammonia permit.

5.2.6 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

- Confirm with regulatory bodies the requirement for ammonia removal at the proposed discharge locations.
- Confirm the existing ASP capacity and when it will be met in relation to expected growth.
- Discuss the progress of the potential THP project on the existing works and identify the implications this will have on the existing ASP and proposed MBBR of the SRO treatment process.

5.3 TERTIARY PHOSPHORUS REMOVAL

5.3.1 COMAG™

To achieve the assumed 0.2 mg/l total phosphorus consent, Evoqua's CoMag™ process has been considered for Gate 2 in continuation from Gate 1. This technology is being applied at other SROs and is preferred by STW on larger applications compared to filtration techniques that will generate a large backwash returns volume. CoMag™ has been proven to achieve low phosphorus permits at Finham WwTW (0.1 mg/l) and Cannock WwTW (<0.1 mg/l).

CoMag™ is a tertiary solids removal process based on conventional coagulation and flocculation, but uses magnetite, an inert, high specific gravity (5.2), finely ground, non-abrasive iron ore as a ballast. The system can process a wide range of flows and loads with little effect on contaminant removal performance.

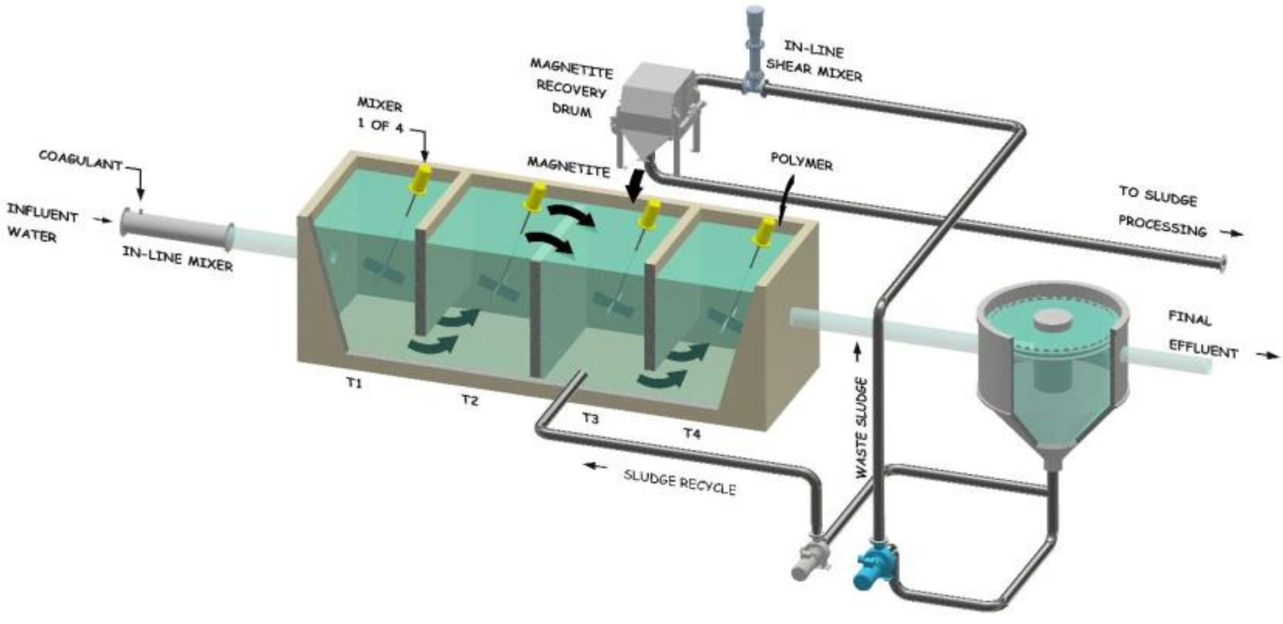
The magnetite infuses into the metal hydroxide floc increasing its specific gravity and improving the settlement rate leading to greater solids capture and a smaller footprint requirement compared to conventional clarification tanks.

A coagulant (ferric sulphate) is dosed at a fixed iron concentration as mg/l into a static mixer. The iron concentration is adjustable via the CoMag™ human machine interface (HMI), and the software converts the required iron concentration into a ferric sulphate dose as litres per hour (l/hr) according to flow to the CoMag™ plant.

The static mixer provides flash mixing with tanks 1 and 2 allowing for gentle mixing and the development of large metal hydroxide floc. In tank 3, magnetite is added to the floc via three means. Recycled sludge, magnetite recovered by the mag drum during the sludge to waste cycle, and virgin magnetite added manually. Magnetite is added to maintain a magnetite concentration in the system which is determined during commissioning and optimisation.

In tank 4, a polymer solution is added to the effluent using a carrier water and gently mixed to promote large floc development before discharge via gravity to the clarifier. A summary of the CoMag™ treatment process is shown in Figure 5-6.

Figure 5-6 - Summary of the CoMag^{TM} process (Evoqua)

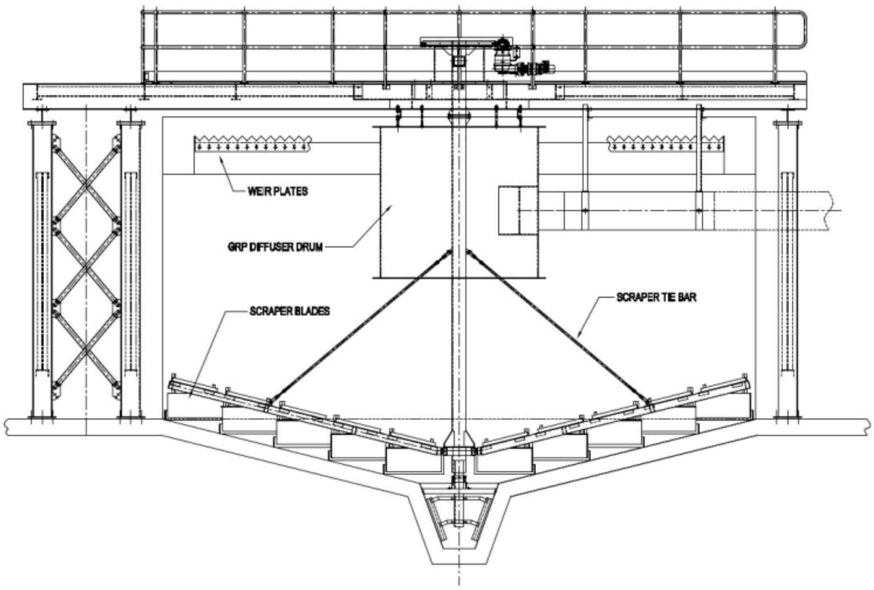


Summary of CoMag^{TM} Treatment Process

Within the clarifier, the magnetite infused solids are separated from the effluent water. The high degree of separation of solids from the effluent ensures low phosphorus concentrations leaving the clarifier. Settled sludge is recycled continuously to tank 3 and periodically removed from the process via the magnetite recovery drums. An inlet baffle is located in the centre well to dissipate the energy of the conditioned effluent as it enters the clarifier. Energy dissipation is key to prevent sludge blanket pluming and solids loss.

The clarifier will comprise a fixed bridge, centre drive, side feed and CoMag^{TM} sludge collection mechanism. The tank floor slope should be a minimum of 14 degrees (°), which is steeper than STW specifications but encourages sludge collection and magnetite recovery. A typical CoMag^{TM} clarifier is shown in Figure 5-7.

Figure 5-7 – CoMag^{TM} clarifier design (Evoqua)



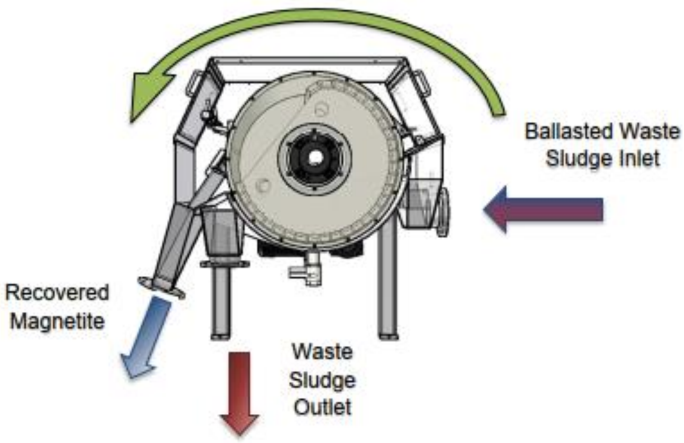
Typical CoMag™ Side-Feed Process Clarifier

Sludge is recycled from the clarifier back to the reaction tanks by centrifugal pumps acting as duty/ standby. Recycling sludge increases nucleation sites, enhances precipitation kinetics and promotes floc sweep. This leads to improved solids removal and more efficient chemical use. Each pump is fitted with semi-open impellers to prevent blockages.

A continuous final effluent water supply of up to 9 litres/minute (l/min) per pump is required to provide a seal flush around the pumps.

The magnetite ballast is recovered from the waste sludge magnetically (>95% reported, to be confirmed during commissioning) and returned to the treatment system. Waste ballasted sludge is fed to the drum through an overflow weir. The sludge flows down through the drum and wastewater sludge is separated from the magnetite. The magnetite recovery drum is shown in Figure 5-8.

Figure 5-8 - Magnetite Recovery Drum



Recovered magnetite can be reused without any loss in its effectiveness as a ballast.

The key design parameters of the CoMag™ Process are shown in Table 5-4.

Table 5-4 – CoMag™ Design Parameters

Key design parameters	Preliminary Design
Flow (Min/ Avg/ Max)	17,000 m^3/day / 35,000 m^3/day / 47,520 m^3/day
Average Influent Suspended Solids	110 mg/l
Peak Influent Suspended Solids	150 mg/l
Average Influent Total Phosphorus	4.0 mg/l
Peak Total Phosphorus	6.6 mg/l
Average Influent Ortho Phosphorus	3.5 mg/l
Feed pH range	6 – 8

Two items to note in Table 5-4 are the influent suspended solids concentrations and the influent phosphorus concentrations. The high influent suspended solids allows for sloughing from the MBBR and is the main contributor to the high waste sludge volume of the CoMag™ process. With respect to phosphorous concentration, the plant was designed under the assumption that it would remove all phosphorus with no primary phosphorus removal. Since the proposal, primary phosphorus removal has been added in discussion with the STW process design excellence which will reduce the total phosphorus concentration in the feed to 1.2 mg/l under average conditions assuming a 70% removal rate.

This reduction in feed concentration will not impact unit sizing since this is based on flow. The iron dose of 10 mg/l is likely to remain the same as molar ratios between iron and phosphorus increase as the phosphorus concentration decreases, and so the operational expenditure (OPEX) cost associated with ferric sulphate dosing will remain the same.

Any changes in recognised OPEX costs will not be realised until jar testing, commissioning and optimisation has been undertaken.

The preliminary process configuration is outlined in Table 5-5

Table 5-5 – CoMag™ Preliminary Process Configuration

Parameter	Preliminary Design
Number of reaction tank streams	1 x 100%
Number of reaction tanks per stream	4
Preliminary Reaction Tank Volume (per tank)	73 m^3
Reaction Tank Dimensions (per tank)	4.2 m x 5.3 m (Depth x Height)
Number of clarifiers	1 x 100%

Parameter	Preliminary Design
Clarifier type	CoMag™ Clarifier
Clarifier diameter	12 m
Clarifier side wall depth	4.27 m
Clarifier floor slope	14°
Magnetite recovery drums	1

Operating consumables for the CoMag™ process are shown in Table 5-6.

Table 5-6 – CoMag™ operating consumables

Parameter	Value
Magnetite initial charge	11,010 kg
Daily magnetite top up (loss)	53 kg/day
Coagulant dose (as iron)	10 mg/l
Daily average coagulant consumption (at 12.5% iron)	1,806 l/day
Polymer dose (100% active ingredient)	0.8 mg/l
Daily average polymer consumption (100% active ingredient)	28 kg/day
Daily power consumption	200 kilowatt hours per day (kWh/day)
Daily waste sludge production (Average)	1,032 m^3/day

5.3.2 ANCILLARY EQUIPMENT

5.3.2.1 Ferric sulphate dosing

Dosing will be flow paced to achieve an iron concentration adjustable via the HMI. This equates to a dose of between 34 and 153 l/hr due to the variation in feed flow caused by the dry day diurnal flow. The actual dose required will be confirmed by jar testing and optimised during commissioning.

Delivery will be by tanker via an offloading area with interceptor and interlocking system to prevent delivery until flow to site drainage is isolated and diverted to the interceptor as per Severn Trent standards. The ferric sulphate storage tanks, providing a combined 54 cubic metres (m^3) capacity, will be high density polyethylene (HDPE) or other suitable plastic and installed in a concrete bund. Two tanks will be provided with a balancing line so each can be taken offline for maintenance.

The dosing pumps will be arranged as duty standby and, due to the process criticality, duty standby dosing lines. The ferric sulphate storage tanks and dosing equipment can be constructed off site and assembled upon receipt of delivery to reduce construction health and safety risks.

5.3.2.2 Polymer dosing

The type of polymer used for the CoMag^{TM} process will be decided using jar tests and the STW preferred polymer supplier. It is assumed that a powdered polymer make-up system will be used.

The polymer will be diluted to 0.3% using potable water. This equates to a polymer solution flow rate of 292 to 1,485 l/hr due to the large range of flows that require treatment. Carrier water (CoMag^{TM} effluent) is added to improve dispersion and mixing at the point of application in T4. The carrier water is typically set to a ratio of 1:4 against the polymer solution requirement.

The polymer dosing system will comprise a dry powder storage room with transfer to a 'wet make up room' via eductor and blower. The quantity of polymer to make a 0.3% solution in the makeup tank can be calculated, and the feed screw and blower timers can be calibrated and set during commissioning.

From the make-up tank, after a mixing period that is programmable via the HMI, the polymer is transferred to a storage tank from which the duty standby dosing pumps draw. The system will comprise duty/ standby dosing pumps and pipework given the criticality of polymer to successful process performance.

The polymer makeup system will be a batch process. The transfer of a makeup batch to the storage tank will initiate a makeup according to level in the makeup tank.

Polymer will be delivered to site via a heavy goods vehicle (HGV) and offloaded on pallets. If a big bag system is required, then a large kiosk with lifting system for loading will be required. Storage will require a temperature and humidity-controlled kiosk to prevent moisture exposure and polymer handling problems.

The polymer dose to the CoMag^{TM} system will be flow paced to achieve a concentration of active ingredient, set via the HMI. The polymer make-up concentration is also set via the HMI, and the software will convert this into a flow rate to control pump speed. Given the viscous nature of the polymer, it is not recommended to convert the pump speed into a flow rate.

5.3.2.3 Magnetite storage kiosk

A 4m x 4m kiosk will provide storage for the bags of magnetite delivered to site on pallets. The kiosk will be placed local to the four main CoMag^{TM} tanks with a permitted route for a trolley to safely transfer the bags from the kiosk to the loading winch of tank 3.

5.3.2.4 Potable water supply

Potable water is required for the polymer make up of 13 m^3/day under average conditions. Potable water will need to be provided either from the existing potable water distribution network, or independent booster set.

5.3.2.5 Final Effluent booster Set

A final effluent booster set is required to provide carrier water to the polymer dosing system of 37 to 51 m^3/day, and to the sludge recycle pumps to provide seal flush water at 9 litres per minute. It is proposed to take the effluent from downstream of the ozone pumping station via an independent booster set with duty standby pumps and close coupled storage tank.

5.3.2.6 Sludge transfer tank and pumps

Wasted sludge (post magnetite recovery) will be stored in a sludge storage tank prior to transfer to a sludge thickener system. The sludge dry solids concentration will be less than 1%, so thickening will be provided prior to further treatment on site using existing dewatering and digestion facilities. CoMag™ sludge storage will be provided to permit continuous operation of the CoMag™ plant and batch operation of the sludge thickener.

5.3.2.7 Monitoring

Phosphorus, total iron and TSS monitors are recommended for the Ozone pumping station to monitor CoMag™ performance. These instruments won't be used for control. It is not recommended to provide an interlock which shuts down ferric sulphate dosing in the case of a high iron concentration, as this can worsen process performance due to a loss of sludge blanket.

5.3.3 RISKS

5.3.3.1 Total phosphorus assumption

The provision of a tertiary solids removal process assumes that a 0.2 mg/l total phosphorus will be imposed on effluent discharged to Deerhurst / Haw Bridge. This permit requirement has not been confirmed. With no confirmation, there is a risk the design is incorrect, and a different phosphorus removal technology could be required if the permit tightened. Total phosphorus permits less than 0.2 mg/l are being applied across the United Kingdom (UK). CoMag™ can achieve lower than 0.2 mg/l total phosphorus concentrations, but this design is to achieve 0.2 mg/l.

5.3.3.2 Environmental impact

The CoMag™ system requires ferric sulphate, polymer, magnetite, potable water and final effluent to operate, each with their own power demands. It is expected that there will be 3 deliveries of chemical to site per month to comply with 28-day storage requirement. This could be reduced by providing larger storage capacity. Ferric sulphate and polymer require their own independent dosing systems with high operational demands (carbon) and raw material consumption. Ferric sulphate dosing systems have been standardised and can be easily manufactured and commissioned. These are therefore the solution of choice for rapid deployment to comply with new phosphorus permits. However, ferric sulphate supplies are diminishing, and due to the number of phosphorus permit imposed on United Kingdom (UK) wastewater treatment works, the demand is increasing.

Polymer is derived from crude oil, and given the existing political disruption in Eastern Europe, the supply chain may be at risk. The provision of these chemicals also has its own carbon footprint and energy demands, and the overall carbon footprint of the project may push the more appropriate solution towards filtration technologies or biological phosphorus removal.

CoMag™ was included as a recommendation from Gate 1. Budget capital expenditure (CAPEX) costs for Mecana units are approximately £4million, but removes the need for a polymer dosing plant, potable water supply, magnetite. Filtration technologies for tertiary phosphorus removal are generally less operationally intensive than a CoMag™ plant, where the balance of chemicals is crucial to its performance.

5.3.3.3 Magnetite Losses

There is an ethical debate as to whether a loss of magnetite to the environment either by discharge from the clarifier, or sludge sent to landfill is acceptable. A 53 kg/day top up equates to over 19.3 tonnes (T) of magnetite per year. Whilst some of this is to top up due to settlement within the systems (sections of the clarifier or transfer pipework), it is reasonable to expect tonnes of magnetite to be lost to the environment each year. At Finham WwTW, whilst low phosphorus concentrations are achieved, magnetite traps have had to be installed on the outlet to river due to losses.

5.3.3.4 Magnetite Consumption

Chemical consumption and hence OPEX may change once jar testing and commissioning has been completed. An evaluation test report will be produced by Evoqua with recommendations on performance suitability. At Finham WwTW, there was a 93% increase in actual magnetite consumption compared to the consumption identified prior to operation, meaning magnetite costs could be £8,940 greater per year.

If the same increase is applied for Netheridge, magnetite consumption increases to 102 kg/day. At this requirement, and automated addition with silo, such as that installed at Finham WwTW would be required, and would add additional cost and operational complexity to the project. This won't be realised until jar tests are undertaken, and the requirement fully confirmed at detailed design.

5.3.3.5 Maintenance

No side manway access to the CoMag™ reaction tanks is provided. Evoqua have not identified a requirement to enter the tank for maintenance. The mixers in each tank are top entry mixers that can be accessed from the platform and walkway on top of the tanks.

5.3.3.6 Operational demands

Whilst the CoMag™ process is designed to be fully automated with a daily magnetite top up and sampling the only fundamental requirements, it is a complex process to operate and relies on multiple pieces of equipment to perform. Troubleshooting therefore can be quite a lengthy process unless the operators have a lot of experience.

5.3.3.7 Redundancy

The provision of one stream provides a single point of failure for the whole treatment process prior to discharge, putting compliance with the new permit at high risk.

5.3.3.8 Integration with other tertiary treatment equipment

The CoMag™ uses a standalone programmable logic controller (PLC) which will require integration with the PLC for the rest of the works, which can add complications and time to commissioning programmes.

5.3.3.9 Manual Handling

2no. 25 kg bags of magnetite (3no. a day once a week) will need to be lifted manually onto a trolley and lifted out into the basket of the winch.

Magnetite is not classified at hazardous to humans or the environment. Dust masks are recommended when adding magnetite to T3. In addition to standard personal protective equipment (PPE) requirements at wastewater treatment works.

5.3.3.10 Potable water availability

The polymer make-up system requires 13 m^3/ day of potable water. It has been reported that the existing demand is close to the capacity of the supply, and sometimes the demand has deprived neighbouring houses of their potable water supply. During detailed design, a review of potable water supply should be undertaken, and serious consideration made for an improved potable water supply. There may be an opportunity to use Netheridge SRO effluent as process water, provided it is of sufficient quality.

5.3.3.11 Suitability for stop/start operation

The CoMag™ can handle a rate of change of 5%/minute in flow. Any greater rate of change risks disturbance to the sludge blanket in the clarifier and a loss of solids and hence phosphorus. At Finham WwTW, when the CoMag™ was shut down for a few days and brought back online, the sludge that had not been removed from the process had turned septic and led to sludge handling issues/ solids carry over during start up.

The CoMag™ unit should be kept online with a sweetening flow even when there is no demand for water from STT, so it is able to respond when the transfer pumping station is called to run. This will be a necessary operational cost to ensure that the treatment will operate effectively when called to run.

5.3.3.12 Magnetite settlement

Due to the extreme settlement properties of magnetite, if not kept in suspension it will settle, and this should be considered in the design of pipework (velocity) to prevent blockages and a high maintenance demand.

5.3.3.13 Ferric sulphate handling hazards

Ferric sulphate is harmful if swallowed, causes skin irritation and serious eye damage and can be corrosive to metals. Individuals with pre-existing liver diseases may have increased susceptibility to the toxicity of exposure.

Ferric sulphate may also cause irritation to the upper respiratory tract, mucous membranes and lung tissues if inhaled. Skin and eye contact could lead to burns. Consideration to prevent direct contact should be included in detailed design.

A safety shower and eye wash station will be included. To avoid contact with skin, eyes and clothing, chemical resistant suits, full face shields and splash resistant goggles must be worn when handling ferric sulphate.

5.3.3.14 Polymer handling hazards

Powdered polymer has been assumed for this project. Powdered polymers are generally classed as non-hazardous, although can cause irritation to eyes. Aqueous solutions of polymer, or polymer powders that have become wet, render surfaces extremely slippery.

5.3.4 OPPORTUNITIES

that form part of the assumed permit for discharge to Deerhurst/ Haw Bridge.

5.3.5 ALTERNATIVE TREATMENT PROCESSES

5.3.5.1 Filtration Technologies

At high level, filtration technologies such as Eliquo Hydrok Mecana units or Bluewater Bio Filter clear units were not considered due to the high volume of backwash water.

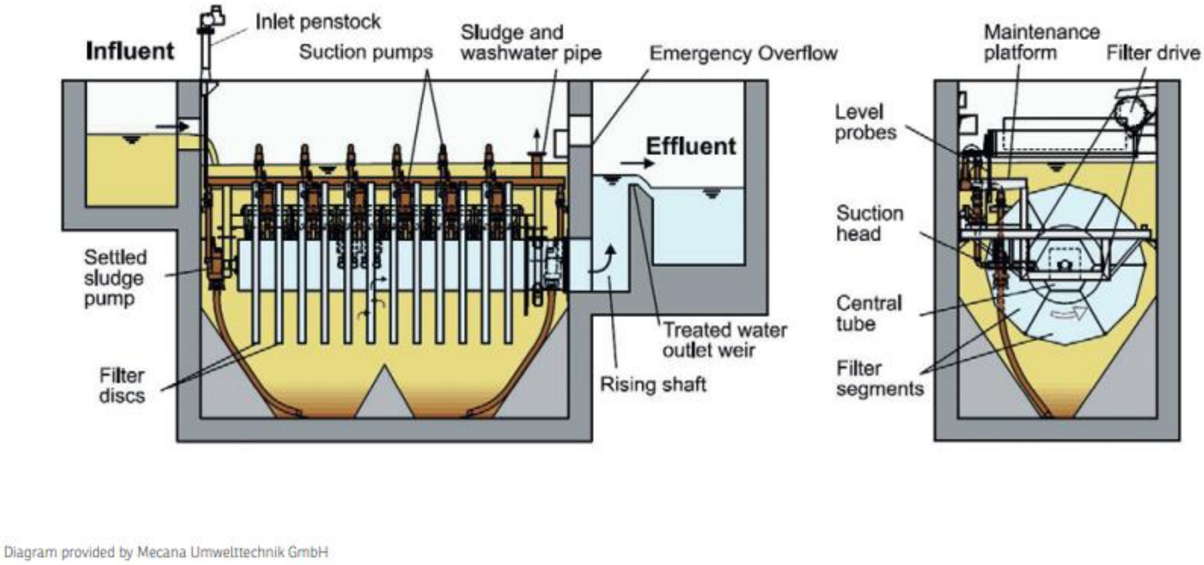
Mecana

Mecana units operate as a series of pile cloth units to remove total suspended solids, phosphorus and some micro-pollutant ‘priority substances’, as clean water flows through the filter with solids retained on the cloth. As the resistance of the filters increases due to fouling, the level in the tank will rise to initiate a backwash, generally every two hours. The pile cloth filter media comprise a series of fine fibres microns in diameter woven into a filter cloth, creating a large surface area.

With regards to chemical consumption, the Mecana units will require a coagulant dose upstream of the filters only. There is no polymer, magnetite or potable water requirement as part of normal operation.

Mecana units can achieve total phosphorus concentrations less than 0.1 mg/l. A diagram of how they operate is shown in Figure 5-9.

Figure 5-9 - Mecana Unit (Mecana Umwelttechnik GmbH)



FilterClear

FilterClear units have been installed on STW assets for flows up to 160 l/s, and can achieve total phosphorus concentrations of 0.1 mg/l.

The filter clear unit is a down flow filtration technology containing four layers of different media; anthracite, silica, alumina and magnetite. Solids are removed progressively through the depth of the filter bed as the pore size of the media decreases. Throughput ranges of up to 500 l/s are permitted per vessel at a filtration rate of 25 metres per hour (m/hr) and above. The units are a package plant assembled off site for easy installation. The configuration of a FilterClear vessel is shown in Figure 5-10.

Figure 5-10 - FilterClear high rate multi media filtration system (Bluewater Bio)

The filtration technologies could be more suitable for intermittent flow and could be turned off when there is no demand from STT which would save on annual OPEX costs.

However, the impact of returning the back wash water (typically 10% of FFT which could be up to 198 m^3/hr) to the head of the works will require analysis to ensure the works is not hydraulically overloaded, and there is no significant increase in OPEX costs.

5.3.5.2 Biological Phosphorus Removal

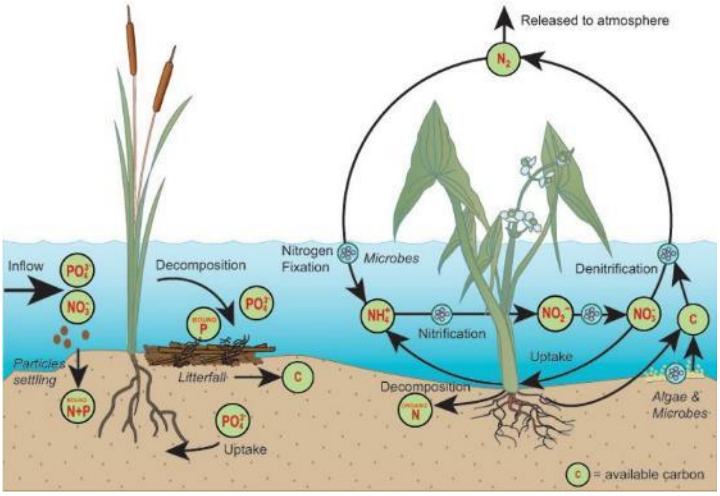
As described in Section 5.1.3, biological phosphorus removal may be an attractive alternative to remove the high demand for chemicals and reduce the environmental impact.

If flow through the tertiary treatment process is to be continuous, then a Nereda® process (Section 5.1.3) could remove the MBBR and CoMag™ process to achieve combined ammonia and phosphorus removal in a small footprint.

5.3.5.3 Wetlands Technology

5.3.5.4 Wetlands are a low carbon wastewater treatment technology that can be used to remove phosphorus from wastewater through a combination of physical, chemical and biological processes. The means of phosphorus removal depends on its form in the influent to the wetland. Some settles to the base of the wetland, encouraged by the reduction in flow caused by roots in the water. Dissolved forms are used by plants and microorganisms and can accumulate in sediments by adsorption due to reaction with iron, aluminium, calcium and magnesium.

Figure 5-11 - Simplified illustration of the nitrogen and phosphorus cycles in a Wetland (Integration and Application Network, University of Maryland)



Uptake by plants converts the phosphorus into organic compounds for growth. However, the majority of the phosphorus is assimilated in the plant and released back into the water when the plant decomposes. Plant uptakes vary between 30 and 150 kilograms per hectare per year (kg/ha/yr).

The wetlands have a limited amount of phosphorus that can be stored. Adsorption is reversible, and each substrate has a particular capacity until it cannot adsorb any more. To continually remove phosphorus, new soils need to be created in the wetland made from plant stems, root debris, leaves and undecomposable parts of dead algae, bacteria and invertebrates.

The phosphorus removal performance of the wetland is highly dependent on loading rate and retention time and is influenced by several factors such as oxygen presence, nutrient loading, season and temperature.

One study proved a median phosphorus removal rate of 1.2 grams per square metre (gm^{-2}). For 35 MLD, a reduction of 1.0 mg/l would require an area of 29,167 square metres (m^2) (170m x 170m). Average total phosphorus concentrations in the final effluent are 4 mg/l, so a reduction of 3 mg/l to achieve a 1mg/l permit would require and area of 87,500 m^2 (295m x 295m) if the removal rate is scalable linearly. The median total phosphorus removal efficiency was 46%, with a 95% confidence interval of 37-55%.

Another study resulted in 75% total phosphorus removal in the first year of operation but decreased to 40% in the second year as the wetland becomes saturated with phosphorus.

There are some unresolved concerns regarding quality fluctuations, seasonable variations and the lowest achievable total phosphorus concentrations.

Despite the significant land take there are environmental benefits, and water companies are currently being asked by Ofwat to include nature-based solutions in their business plans for asset management plan eight (AMP8).

5.3.6 CARBON IMPACT

The CoMag^{TM} process has been chosen as a continuation of the technology proposed at Gate 1 though there are lower carbon alternatives. The high chemical demand compared to the alternative options presented in section 5.3.5 leading to 4 chemical deliveries per month is a contributing factor, as is the carbon demand from the ancillary treatment equipment. However, tertiary phosphorus removal processes that use filtration have been discounted from a treatment practicality perspective due to the high volumes of backwash water, subject to a review of the impact at the head of the works. The carbon impact of the CoMag^{TM} process has been reduced by using powdered polymer to reduce chemical transportation, and variable speed pumps to optimise flow and dose.

The feed to the CoMag^{TM} plant is by gravity from the MBBR outlet, removing the requirement for pumping flow between the units. Filtration processes such as filter clear would require a pumped feed.

A wetlands process would provide the lowest carbon. It is a passive system that uses natural processes therefore the carbon impact is very low, and the provision of plants helps to remove carbon dioxide from the atmosphere and increases biodiversity. However, such plants require a large footprint and do not have the ability to further optimise treatment performance if required.

The provision of a biological nutrient removal process to combine ammonia and phosphorus removal into an intensive process could also provide a lower carbon impact, especially if the existing ASP is upgraded instead of building new.

5.3.7 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

- Confirmation of total phosphorus permit requirements for each discharge location.
- A review of the impact of backwash returns from filtration processes on hydraulics at the head of the works
- Explore the progression of the proposed changes to the existing permit to include a phosphorus consent and identify what implications it will have on tertiary phosphorus removal for this SRO scheme.

5.4 ORGANICS REMOVAL

An interstage pumping station (3no. transfer pumps, duty/assist/standby) receives clarified effluent from the CoMag^{TM} clarifier and pumps this (200 l/s to 550 l/s) to 4no. ozone contactors operating in parallel.

5.4.1 OZONE

Treatment with ozone (a strong oxidising agent) is provided to oxidise pesticides and herbicides, namely chlorothalonil and octyl phenols into biologically degradable substances. The data in Table 4-2 shows removal of chlorothalonil is not required although the inclusion of this on any future permit requires further discussion with regulatory bodies. A removal of up to 50% of octyl phenols is required. Laboratory scale experiments have shown up to 60% removal of chlorothalonil and almost complete removal of octyl phenols with ozone however it is not a selective process, and the actual removal performance will require verification with pilot trials. Ozone has been developed as a robust treatment process to contribute towards the achievement of a high-quality effluent.

Ozone combined with a downstream biologically active filtration process will contribute to organic carbon removal, leading to an improved GAC process performance downstream (Section 5.6.1).

The ozone treatment system comprises a liquid oxygen storage facility, ozone generators, ozone contactors and a cooling water loop around the generators.

The ozone generators and injection system are provided in containers for ease of installation on site.

Ozone can be switched off when there is no demand from STT, but it may affect the downstream microbiology in the BAF process. This may lead to process instability and compliance problems when the demand from STT returns. Therefore, it is recommended that ozone remains online throughout the year. This will impact OPEX and carbon footprint.

5.4.1.1 Oxygen source

Ozone can be generated from the air or from pure oxygen. Liquid oxygen is the preferred source because it is low in contaminants and water vapour. The liquid oxygen will be delivered to site in tankers and stored in pressurised tanks. Liquid oxygen used for ozone generation will have lower capital costs and is simpler to operate and maintain compared to using air.

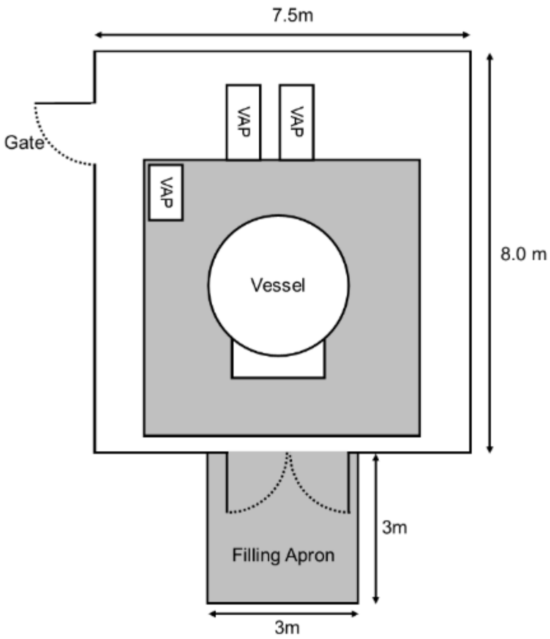
The liquid oxygen for supply to the generators must meet the requirements in Table 5-7.

Table 5-7 – Liquid oxygen feed requirements

Feed Gas Requirement	Value
Liquid oxygen content	99.5 – 99.9%
Nitrogen content	>700 parts per million volume (ppmv)
Water content	<2.6 ppmv
Hydrocarbon content	<60 ppmv
Solids	Particle free
Pressure inlet generator	2 – 6 bar (g)
Pressure outlet generator	0.9 bar (g)
Design Pressure	1.2 bar (g)
Temperature	0 – 40 degrees Celsius (°C)

The bulk liquid oxygen and supply equipment would be installed and rented from the supplier as part of a minimum 3-year supply agreement. The system includes telemetry for remote stock monitoring and scheduling. A typical plinth construction for liquid oxygen storage is shown in Figure 5-12.

Figure 5-12 - Typical plinth construction for liquid oxygen storage (BOC)



5.4.1.2 Ozone Generators

Due to its instability, ozone must be generated on site at the point of use. The ozone generators produce ozone directly in the water. An advantage of this method is no impurities are introduced from the feed gas. Ozone is produced by an electrolysis cell continuously in the water flow. The electrolysis cell consists of an anode/cathode and polymer solid electrode membrane which operates as a non-dissolvable electrolyte and separator in the cell. The electrolysis process leads to the production of ultra-pure ozone in the water. Released heat from ozone generation will be removed by cooling water. Xylem's SMOevo^{PLUS} ozone generator is shown in Figure 5-13.

Figure 5-13 - WEDECO Ozone Generator Type SMOevo^{PLUS} 860 (Xylem)

WEDECO Ozone Generator Type SMOevo^{PLUS} 860

1	Power Supply Unit
2	Operator Panel
3	Safety Pressure Switches
4	Safety Valve
5	Pressure Reducer
6	Gas Inlet Flange
7	Gas Inlet Filter
8	Gas Control Valve
9	Check Valve
10	Cooling Water Flow Meter
11	Cooling Water Inlet Flange
12	Ozone/Gas Outlet Flange
13	Ozone generator
14	Ozone Concentration Monitor optionally
15	Gas flow Meter
16	Air Conditioned Cabinet Door

Table 5-8 – Ozone generator design specification

Parameter	Value
Feed gas supply	Liquid Oxygen
Feed gas pressure	3 – 6 Bar
Ozone concentration	179 grams per normal metre cubed (g/Nm^3) at 12.0 weight percent (wt.%)
Ozone production range	1 – 100% (20-100% gas controlled)
Outlet gas pressure	0.9 bar (g)
Cooling water temperature inlet	15°C
Cooling water pressure	1 - 3.5 Bar

Table 5-9 – Ozone Generator Performance

Parameter	Value
Ozone production	7,500 grams per hour (g/hr)
Liquid oxygen demand	62.5 kilograms per hour (kg/hr)
Liquid oxygen demand	43.6 normal metres cubed per hour (Nm^3/hr)
O_2/O_3 Gas flow rates	41.9 Nm^3/hr (0°C. 1013 hectopascal (hPa))
Cooling water demand (generator)	11.9 m^3/hr
Cooling water demand (PSU) max.	0.4 m^3/hr
Cooling water (total)	12.3 m^3/hr
Specific energy consumption	10.1 kW h/kg
Total energy consumption	76.1 kilowatt (kW)

5.4.1.3 Ozone introduction system

Ozone will be introduced into the effluent via venturi injection. The side stream flow is taken from the effluent via a motive water booster pump followed by venturi injector. Water flow through the injector produces a partial vacuum which is utilised to draw ozone into the water stream and mix the two phases vigorously. The water jet leaves the venturi under turbulent conditions and disperses the gas through fine bubbles, increasing the contact surface area between the gas and water phases. The ozonated water is introduced back into the main water pipe and to the contact tanks. An ozone injection system supplied by Xylem is shown in Figure 5-14.

Figure 5-14 - WEDECO skid mounted ozone pump / injection system (Xylem)

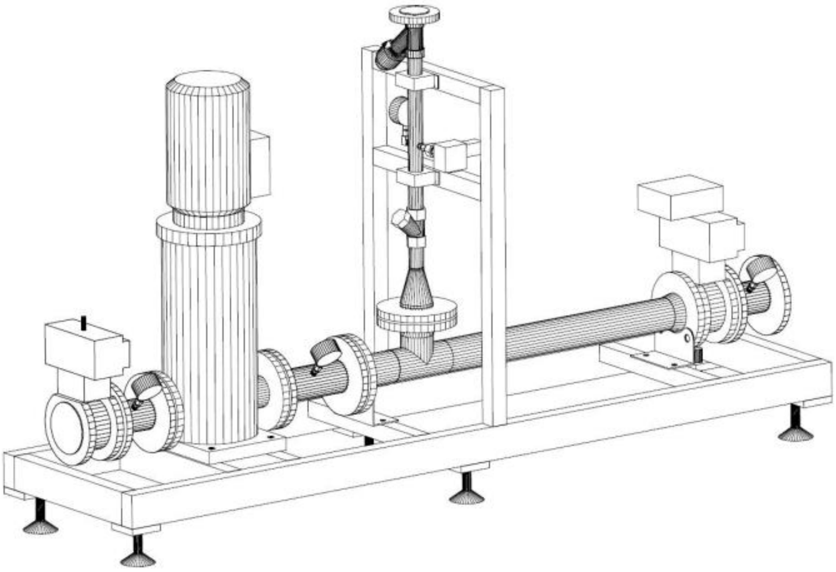


Table 5-10 – Ozone injector technical data

Item	Value
Type	Unpressurised system
Quantity	1
Injector type	DN 150
O_2/O_3 gas flow rate/unit	41.9 Nm^3/hr
Water flow/unit	90 m^3/hr
Water pressure at pump inlet	0 bar (g)
Pressure pump (max)	11.5 m
Discharge pressure	0.5 bar (g)

The advantage of a side stream injection system over a fine bubble diffuser is that the mechanical components are kept outside of the contact tank for ease of maintenance.

5.4.1.4 Ozone destruction system

The off-gases from the contact tank must be treated to destroy any remaining ozone as it is extremely irritating and toxic. A thermo catalytic ozone destruction system removes non-dissolved gas and converts residual ozone to oxygen using catalytic material in a reaction chamber unit to achieve a concentration lower than 0.1 parts per million (ppm) (Figure 5-15). A blower collects the gas stream from the reaction system by vacuum through the destruction system, controlled by a pressure sensor. The gas stream is heated to 8-10 °C above the inlet temperature. The chimney is equipped with a condensate trap to avoid condensate backflow to the blower.

Figure 5-15 - Catalytic ozone destruction system (Xylem)

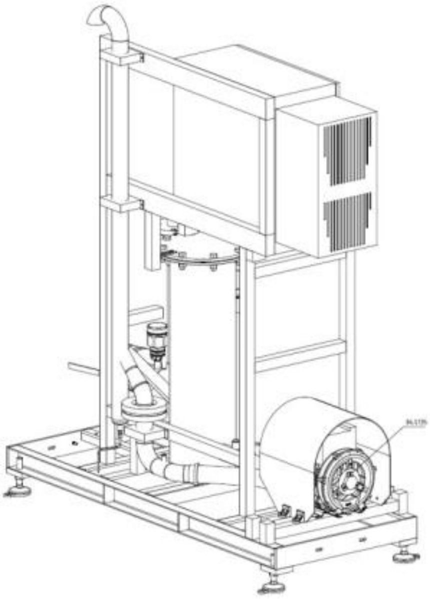


Table 5-11 – Ozone destruction system technical data

Item	Value
Quantity	1 + 1 (Standby)
Gas flow rate	42 Nm^3/hr
Max gas flow	100 Nm^3/hr
Installed heating capacity	0.7 kW
Installed power blower	0.6 kW
Consumed blower	1.0 kW
Dimensions (depth x width x height)	0.65 m x 1.60 m x 1.95 m

5.4.1.5 Dose Control

The dosage is controlled by the flow rate through the ozone treatment plant. The ozone dose is initially set up during commissioning and can be adjusted via the HMI.

5.4.1.6 Cooling Water System

Much of the energy used in ozone generation is lost as heat and a cooling water system is required to avoid overheating and decomposition of the ozone. A closed loop system is preferred, and the cooling water must be of high quality.

If during pilot plant testing it is identified that the final effluent transferred to the River Severn is of sufficient quality, it could be used as the cooling water supply.

5.4.1.7 Ozone contactors

Ozone contactors are required to provide the necessary contact time for organics oxidation and elimination. 4no. contact tanks of 30 m^3 (3.11m diameter with a top water level (TWL) of 4m) will be provided to achieve a contact time of 5 minutes.

5.4.1.8 Ozone monitoring

Ozone monitoring of the surrounding area is required for health and safety purposes to alarm against hazardous releases.

5.4.2 RISKS

5.4.2.1 Perfluoroalkyl and polyfluoroalkyl substances

There is a risk that ozone will break down longer Perfluoroalkyl and polyfluoroalkyl substances (PFAS) chains into shorter chains, which are more difficult to remove and could pass straight through the GAC adsorbers. This is particularly important with regards to the requirement to remove perfluorooctane sulfonic acid (PFOS). The fate of PFAS chains would need to be determined during a pilot trial.

5.4.2.2 Disinfection by-products

Ozone addition to water forms disinfection by products (DBPs), including bromate in the presence of bromide. Bromate can cause adverse health effects and can be damaging to the receiving environment and could have a significant impact from a Water Safety Planning perspective. Therefore, a biologically active filtration (BAF) process has been included downstream to mitigate the emission of DBPs.

One mitigation to reduce bromate formation is to reduce the pH to below 7 using a chemical such as sulphuric acid, but until a pilot plant is conducted, the extent of the development of these DBPs is unknown.

Once any pilot plant trials have been completed, and the concentration of DBPs in the final effluent identified, Water Safety Planning risks should be reviewed as part of Gate 3 investigations.

5.4.2.3 Corrosion

Material selection must be considered for the use of ozone. 304/316 Stainless steel is compatible, but galvanised steel and cast iron are not. Other grades of stainless steel will break down after prolonged exposure.

5.4.2.4 Unknown effectiveness

The micropollutants that require removal by ozone are chlorothalonil and octyl phenols. These chemicals have been highlighted under the ‘introduction of a new substance’ rule, so it is assumed they will require removal to limit of detection levels. Until the wastewater is tested under pilot plant conditions, the removal performance is unknown and therefore a full proposal cannot be provided.

An outline cost to perform one test through a 5 litres (L) reactor would be £1,271 (excl. value added tax (VAT)), or £2,120 (excl. VAT) for one test through a 400 L reactor. This is a cost provided by a research facility in France.

5.4.2.5 Validity of the received proposal

The proposal assumes a reaction system that is unpressurised. The data provided is preliminary and will be subject to revisions as the project develops.

5.4.2.6 Liquid oxygen hazards

Pure oxygen is very reactive and at high pressure can react violently with oil and grease. Other materials may catch fire spontaneously. Textiles, rubber, metals and metals will burn vigorously in oxygen. A leaking valve or pipe in a confined space can quickly increase the oxygen concentrations to dangerous levels.

The main danger to people in an oxygen enriched atmosphere is clothing and hair can spontaneously catch fire. Sources of ignition around the liquid oxygen storage area must be prohibited, and all equipment appropriately rated.

5.4.2.7 Ozone hazards

Ozone is a toxic gas with a distinctive odour and is a normal constituent of the earth's atmosphere. Exposure to ozone will first be noticed at the respiratory tract, the lungs and, at higher concentrations, the lungs. Ozone can cause irritation and damage to the small airways of the lungs.

The main concern of concentrations usually found in the workplace are irritation to the upper airways indicated by coughing and a feeling of tightness in the chest. Uncontrolled exposure to high levels of ozone could lead to lung damage.

Ozone is a powerful oxidising agent and can react explosively with oil and grease. Low concentrations can have a significant effect on metals and plastics.

A safety device is required to monitor the ozone concentration in the surrounding air to alarm in case of a leak and automatically turn the system off in emergency cases. The current workplace exposure limit is 0.2 ppm in air averaged over a 15-minute period.

5.4.2.8 Liquid oxygen supply

The hydrocarbon content in liquid oxygen should be within the range of 40 to 80 ppm to prevent damage to ozone generation equipment. Newer systems are more tolerant than older systems, but this still poses a risk. The ozone generators require a hydrocarbon content in the liquid oxygen of less than 60 ppm as methane. BOC (suppliers of liquid oxygen) are not confident they can commit to the required limits of hydrocarbon concentration.

BOC have three liquid oxygen plants in the UK, located in Sheffield, Port Talbot and Thame. This needs to be considered with regards to delivery and logistics in case one of the plants is unable to supply the liquid oxygen.

5.4.3 OPPORTUNITIES

5.4.3.1 Permitting

Apart from PFOS, the micropollutants listed in Table 4-2 are pesticides and herbicides. The results of environmental screening exercises by external consultants indicate that these pesticides and herbicides in the Netheridge effluent present the risk of introduction of new substances to the river. This is because these chemicals were not detected during sampling of the river at the proposed discharged location between 2020 and 2021. The requirement to remove these micropollutants may change after dilution and dispersion modelling has been undertaken.

If the limit of detection of the analysis technique used is greater than the limit of detection of other analytical techniques, then there could be an opportunity to resample and confirm whether these are new substances, or undetected by the test.

5.4.4 ALTERNATIVE TREATMENT PROCESSES

Alternative available techniques for chlorothalonil and octyl phenol removals are presented in Table 5-12.

Table 5-12 – Alternative removal technologies for chlorothalonil and octyl phenols

Chemical	Alternative removal technologies
Chlorothalonil	Activated carbon Advanced oxidation Reverse Osmosis
Octyl phenols ((4-(1,1’,3,3’-tetramethylbutyl)phenols))	Photocatalytic processes UV (low removal efficiency) Nanotechnology.

Ozone was selected as a treatment process that has been proven on an industrial scale to oxidise organics and can therefore be reliably costed. GAC is an alternative process that can remove these organics and will be included further downstream for PFOS removal for this discharge option. However, ozone is recommended at this stage to provide a degree of robustness to remove organic matter prior to the GAC and prevent it becoming a site for biomass development - obstructing adsorption sites.

The other alternatives listed above have been sourced from research papers and universities, and have not yet been proven on this scale, so have been discounted. Reverse osmosis was not considered due to the extensive pre-treatment, reject water volumes that would require treatment or disposal and high energy input. An alternative could be to culture the bacteria in the ASP to decompose some of the pesticides. This has been successfully applied at laboratory scale.

5.4.4.1 Advanox

Advanox is an advanced oxidation process (AOP) developed by Van Remmen UV Technology (Figure 5-16), used primarily for direct treatment of groundwater in the production of drinking water. The Advanox process breaks down micropollutants by combining UV-C light (ultraviolet light with a wavelength between 100 and 280 nanometres (nm)) with hydrogen peroxide.

A small amount of hydrogen peroxide is added to the feed wastewater before exposure to the UV-C reactor. Here UV light splits the hydrogen peroxide into hydroxyl radicals, powerful oxidants which oxidise the micropollutants in milliseconds. The reaction converts the micropollutants into less harmful and more biodegradable end products, similar to the ozonation process. At full mineralisation these are mainly water and carbon dioxide gas.

Figure 5-16 - Advanox reactor (Van Remmen)

The results from pilot plants found the Advanox process can remove micropollutants up to 99.5%, produces no bromate, N-Nitrosodimethylamine (NDMA), halogenated organic compounds (AOX) or other toxic by-products and has a high degree of disinfection. Pilot plant studies also indicated it can work on dirty wastewater which could be advantageous at Netheridge

One case study on micropollutant removal added a cocktail of 40 micropollutants to a number of waste streams that appear in the preparation of drinking water. UV-C light was applied with low pressure lamps and hydrogen peroxide added. Removal efficiencies of over 90% were proven with a power consumption of 0.12 kilowatt hours per cubic metre (kWh/m^3). A conversion of over 80% was achieved using 0.06 kWh/m^3.

A similar case study investigated the removal of pharmaceuticals from tertiary wastewater and led to at least an 80% removal efficiency of 35 – 40 pharmaceuticals when the levels of UV-C and hydrogen peroxide (H_2O_2) were optimised. This pilot plant operated continuously to better replicate the tertiary wastewater treatment process.

To provide a full-scale version of the tertiary wastewater treatment process for 80% removal to treat 800 m^3/hr would cost 1.5 million euros (£1.27 million). The anticipated electricity consumption was 0.36 kWh/m^3 treated.

5.4.4.2 Wetlands Technology

Wetlands technology as described in section 5.2.4 has also proven to remove pesticides at pilot scale. One pilot plant scale showed a removal of 71 to 99% of 2,4 dichlorophenol, but at feed concentrations of 20 mg/l, meaning effluent concentrations of to 5.8 to 0.2 mg/l. Whilst this is effective removal, the EQS requirement is 0.02 µg/l. So, whilst wetlands are an attractive low carbon technology, the low concentration requirements may not be met.

Dissolved organic carbon concentrations impact the pesticide removal efficiency.

5.4.5 CARBON IMPACT

Filtration processes such as reverse osmosis and ultrafiltration have been discounted because of the need to handle large volumes of concentrated waste streams. Both reverse osmosis and ozone processes are energy intensive.

Whilst wetland processes provide a much lower carbon alternative, pilot plants reviewed have shown that the low EQS requirements cannot be achieved.

5.4.6 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

- Discuss the test methods used to detect the presence of the pesticides and herbicides listed in Table 4-2 in the River Severn at Deerhurst/ Haw Bridge.
- Pilot plant trials to confirm removal performance and the development of any by products or PFOS daughter compounds.
- Once any pilot plant trials have been completed, the concentration of organics in the final effluent should be analysed and reviewed with regards to Water Safety Planning at Gate 3, particularly for discharge into areas for water reuse. Natural organics in the feed water to water treatment works could lead to the development of disinfection by-products during reactions with chlorine.

5.5 ORGANIC CARBON REMOVAL

As much of the calcitrant organics that have been made biologically degradable by ozonation must be removed prior to the GAC plant to prevent development of a biomass in the GAC process which would occupy adsorption sites leading to a decrease in treatment performance. High levels of DOC and TOC can compromise the lifetime of the GAC plant by 25% according to industry experts. The BAF unit can contribute to the removal of readily biodegradable nonbrominated by products from the ozone unit.

The amount of organic carbon that will require removal prior to the GAC is unknown and should be determined by a pilot plant trial including all proposed treatment processes.

5.5.1 BIOLOGICALLY ACTIVE FILTRATION

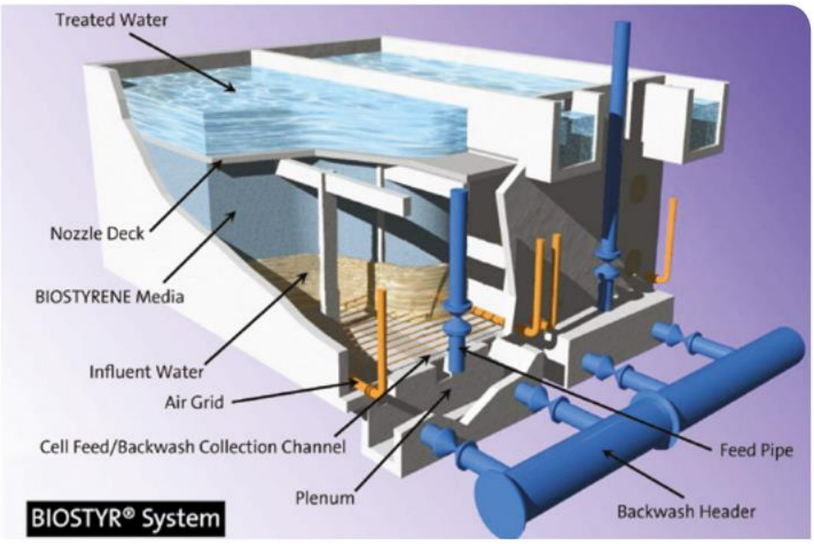
Wastewater that has undergone ozonation will be pumped and evenly distributed between 4 no. biological active filters (BAFs).

The BAF unit is a robust process with a relatively small footprint. The process can provide treatment for a range of flows and loads. The BAF unit generally gives a very high-quality effluent which can be discharged directly onto the GAC plant. BAF units are well established as tertiary treatment processes in the wastewater industry.

Whilst trickling filters provide an attractive low energy alternative, the solids concentration discharged straight onto the GAC filters would be too large and would block the adsorbers.

The BAF has a depth of tightly packed relatively small sized fixed media to provide a high surface area for the biomass to grow. The filter bed is submerged with settled wastewater containing partially oxidised organics from the ozone treatment stage flow upwards through the filter. Feeding from the base causes self-compacting of the filter bed. The microorganisms within the filter media destroy and consume the organics and oxidation by-products, whilst the tightly packed media provides filtering of suspended solids, eliminating the need for a downstream clarification process. The media is retained by a nozzled deck at the top of the unit.

Figure 5-17 - Veolia's BIOSTYR® BAF System



This BAF system includes 2no. air blowers to provide oxygen through a coarse diffuser system in case there is little dissolved oxygen remaining from the ozone introduction stage (as part of a conventional BAF design). The process shown in Figure 5-17 uses buoyant polystyrene beads, the size of the beads depends on the process application. Backwash water is stored above the cell, so no separate clear well is needed. Backwashing to remove filtered suspended solids and excess biomass is performed through a series of valve operations controlled by the PLC.

Effluent will flow via gravity to the GAC pump station.

Table 5-13 – BAF Process design parameters

Design parameter	Value	Comment
Ammonia feed concentration	0 mg/l	Assumed to be minimal post MBBR
BOD feed concentration	5 mg/l	Assumes minimal BOD post ozone
Average Flow	1458 m^3/hr	
Maximum Flow	1,905 m^3/hr	
SOTR	347 kg O_2/day	
Air required	1,342 m^3/hr	Maximum
Number of Blowers	2	Duty/Standby
Blower capacity	75 kW	
Retention time	30 minutes	At FFT.
Total Volume required	990 m^3	Retention time used to size the reactors
Number of units	4	Assumption
Volume per unit	248 m^3	
Top Water Level	5 m	Assumption
Surface area per reactor	50 m^2	
Backwashes per day per reactor	1	
Backwash duration	15 minutes	Assumption
Backwash rate	2 m/hr	Assumption
Total backwash volume per day	99 m^3	
Backwash water flow	28 l/s	
Backwash solids concentration	500 mg/l	Assumption
Backwash solids returned to head of the works	50 kg/day	

The BAF units will be constructed out of concrete rather than delivered to site as a package plant.

As this is a biological treatment process, it will need to remain online when there is no demand from STT and will therefore incur additional OPEX and carbon costs.

5.5.2 RISKS

5.5.2.1 Performance

The organic loading from the ozone process will not be fully understood until the effluent is passed through a pilot plant. Therefore, the full design process design cannot be undertaking until the carbon loading is known. This could impact on CAPEX and OPEX costs, and blower arrangement (more than one assist blower may be required to meet turn down requirements for times of low loading). Sizing based on retention time has been used in the absence of carbon loading data.

5.5.2.2 Backwash water

The diverted effluent to the tertiary treatment process should be greater than 35 MLD to account for the losses due to BAF backwash volumes. Whilst only 99 m^3/day of backwash water is required (0.3% of 35 MLD), when combined with other losses could account for 1,540 m^3/day (4.4% of 35 MLD).

5.5.2.3 Health and Safety

The requirement to construct out of concrete introduces health and safety risks that would not be present with offsite design for manufacture and assembly (DFMA) solutions. Concrete also has a large associated carbon footprint.

5.5.2.4 Process

Despite the ability to achieve a high effluent quality, loss of solids (biomass) to the downstream GAC process will cause adsorption problems and affect removal performance. No solids removal process is included downstream, but this may be required subject to pilot plant tests.

5.5.3 OPPORTUNITIES

5.5.3.1 Recycled polystyrene

The use of recycled polystyrene should be investigated to reduce the carbon footprint and material utilisation of virgin polystyrene, if polystyrene is selected as the media.

5.5.3.2 Combination of biological treatment and adsorption

There is an opportunity to combine biological treatment to remove carbon and adsorption, removing the BAF stage of the process. This is discussed in more detail in section 5.5.4.

5.5.4 ALTERNATIVE TECHNOLOGIES

5.5.4.1 Low energy biological treatment

High rate biofilters were considered as an attractive low energy and low maintenance option, but solids loading onto the GAC must be kept to a minimum and would require an intermediate solids removal process. Furthermore, the timescale for seeding the media tends to be longer.

5.5.4.2 Biological activated carbon

GAC is a material that can support the development of bacteria for the purpose of metabolising biodegradable organic matter. Where GAC has been implemented at drinking water facilities, it was noted that the bacteria in the filter are responsible for a fraction of the organics removal. Pre-ozonation was found to significantly improve the biological activity on GAC.

The bacteria can remove the required portion of DOC to keep adsorption sites free.

Extracellular polymers secreted by the bacteria along with the unevenness of the carbon granules allow the bacteria to remain attached during backwashing. The organic molecules from the ozone process can be trapped in the GAC and used as a nutrient for the biomass. The biological oxidation process within the GAC filters can also be used for ammonia removal.

This combination allows a biological treatment process to be coupled with a physiochemical process and could remove the need for the BAF unit entirely.

The combination of ozonation and GAC in series in water treatment has proven to be effective at pilot plant scale in removing endocrine disrupting chemicals (EDCs) to safe levels. However, if the GAC is biologically active, then EDCs with a high solubility or the chemicals formed during oxidation of EDCs by the ozonation process may be removed further and increase the service time of the GAC plant.

5.5.5 CARBON IMPACT

The low carbon alternative of using high-rate filters was discounted because of the potential for media sloughing onto the GAC which will block the media.

Biological activated carbon would remove the requirement for aeration, typically an energy inefficient process, because the bacteria consume the remaining oxygen from the ozonation process. However, will require activated carbon handling and disposal costs as described in section 5.6. The spent carbon may require incineration if contaminated with PFAS, which has a carbon impact in the form of transportation and incineration.

The system shown in Figure 5-17 utilises a head of water above the media to perform the backwash, from which the backwash water will flow via gravity to a common dirty wash water tank. This removes the requirement for backwash pumping.

5.5.6 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

- Pilot plant trials in series with ozonation to confirm DOC and TOC concentrations in the effluent fed to the GAC process, or the effectiveness of biological activated carbon to remove the requirement of a BAF process.

Undertake detailed analysis into the impact of returning backwash water to the head of the works.

5.6 PFOS REMOVAL

5.6.1 GRANULAR ACTIVATED CARBON

The primary purpose of the granular activated carbon (GAC) is to provide PFOS removal. PFOS has been identified as an impediment towards achieving the WFD target river status. The multi barrier approach of the treatment train ensures the predominant process within the GAC is adsorption.

The EQS of 0.00065 µg/l has been used because it is listed on gov.uk as the relevant EQS for Freshwaters priority hazardous substances, priority substances and other pollutants environmental quality standards (EQS) [updated 21 February 2022]. This corroborates the 2015 WFD Directions. The EA 2019 report: Perfluorooctane sulfonate (PFOS) and related substances: sources, pathways, and environmental data. The October 2019 report discusses other values relevant to setting EQS, but it does not make binding recommendations, and as such those listed in the 2015 WFD Directions remain valid. The target of LoD concentrations has been used to ensure the EQS can be met.

At Netheridge, flow from the BAFs will be pumped and distributed to 11no. downflow filters (with allowance for one to be taken offline for backwashing). The advantage of a downflow filter is it lessens the chance of accumulating particulate material at the bottom of the bed, where it would be difficult to remove by backwashing.

GAC is commonly used for removing organic constituents and residual disinfectants in water treatment. The two principal mechanisms by which activated carbon removes contaminants from water are adsorption and catalytic reduction. Organics are removed by adsorption and oxidising disinfectants are removed by catalytic reduction. Absorption efficiency is promoted by the highly porous structure of GAC; the typical surface area for activated carbon is approximately 1,000 m^2/gm. Absorption is affected by other factors such as molecular weight of the organic material; volatile organics may be removed to 1mg/l levels provided the influent is less than 500mg/l. Activated carbon adsorbs organic material because the attractive forces between the carbon surface (non-polar) and the contaminant (non-polar) are stronger than the forces keeping the contaminant dissolved in water (polar). The kinetics of absorption are comparatively slow and an empty bed contact time of at least 20 minutes is common, however times up to 120 minutes have been required.

Removing organics can be onerous and is always site-specific so testing with different types of carbon is usually required. Combining technologies such as ozone with carbon filtration enhances the removal of organics significantly. GAC also has the capacity for the removal or reduction of heavy metals such as lead, mercury, chromate etc in the influent water, however the efficiency of removal is affected other factors such as water chemistry and flow rates and carbon type.

GAC beds are in effect deep bed filters and accumulate particulate matter which in time increases the headloss and affects the absorption capacity. Cleaning involves backwashing with an upward flow to expand the bed by up to 50%. As well as removing particulates in the influent water the backwashing removes carbon ‘fines’ generated by the abrasion of the carbon granules as they move in the filter bed. Backwashing does not regenerate the capacity of the carbon; it simply removes accumulated debris and reclassifies the filtration bed. Control of micro-organisms in carbon can be problematic, hot water, steam and ozone can be used together with regular backwashing. In this instance a trace dose of sodium hypochlorite is proposed for the backwash. Generally, GAC media should be replaced frequently. For an organics removal application, the bed life could be 6 to 12 months depending on the flow, media loading and carbon type.

The volume of the GAC system has been based on a contact time of 20 minutes and an activated carbon depth of 2m. The empty bed contact time will require confirmation by bench scale testing to ensure the target removal efficiency is achieved. It is proposed to install modular steel units for ease of construction (off site) and installation.

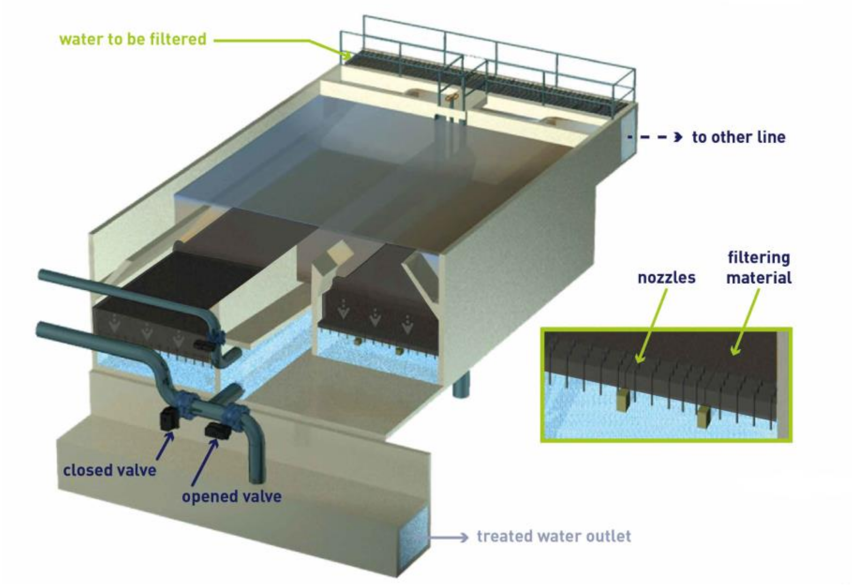
Table 5-14 – GAC Design parameters

Design Parameter	Value	Comment
Filter construction	Civil construction	
Number of units	11 (10 + 1)	10 in service allowing for one offline for backwashing
Maximum flow	1,980 m^3/hr	
Maximum loading	10 m/hr	
GAC adsorber loading rate	5.45 m/hr	
GAC adsorber loading rate (1 unit out of service)	6.00 m/hr	
Empty Bed Contact Time (EBCT)	20 minutes	Typical EBCT for PFAS removal in drinking water.
Total Carbon Volume required	660 m^3	
Carbon density	0.5 kilograms per litre (kg/l)	Assumption
Carbon mass	330 T	
Carbon bed depth	2.0 m	
Carbon expansion	45%	Assumption
Total depth	2.9 m	
Total area	330 m^2	
Area per filter	33 m^2	
Backwash rate	25 m/h	
Backwash duration	15 minutes	
Water per backwash	206 m^3	
Backwash flow	229 l/s	
Backwash cycle	6 days	Assumption
Contactors washed per day	2	

Design Parameter	Value	Comment
Total volume of backwash water	413 m^3/day	
Backwash solids concentration	100 mg/l	
Backwash solids load	51 kg/day	

An example of an open GAC filter at a water treatment works is shown in Figure 5-18.

Figure 5-18 - Suez's Carbazur® GH open GAC filter



The effluent from the GAC will flow via gravity to the transfer pumping station.

It is assumed that backwashing will occur every 6 days to remove debris and reclassify the media. Backwashing will be initiated either manually or as a result of high media head loss (monitored by differential pressure monitors) or elapsed time. Clean back wash water fed from the clean back wash water tank adjacent to the transfer pumping station will be provided at a rate of 87 l/s for 15 minutes. Strainers fitted into the top of each vessel will prevent the loss of carbon into the final effluent.

5.6.2 BIOMASS CONTROL

The upstream organic carbon removal process (BAF) is designed to remove any available carbon that would encourage bacterial growth, but to further control biomass development in the GAC vessels, sodium hypochlorite could be dosed into the backwash water if required. This will be dosed from an intermediate bulk container (IBC) suitably stored to protect against degradation caused by UV using a WES dosing cube or similar (Figure 5-19). Control using chlorine monitors is not required.

Figure 5-19 - Dosing cube and IBC storage installation (WES)

Whilst the sodium hypochlorite may aid in preventing the development of biomass within the GAC units, it can also break down the GAC into fines which could be lost in the final effluent and discharged to river, worsening final effluent quality.

5.6.3 RISKS

5.6.3.1 PFAS compounds

PFAS are a large, complex, and ever-expanding group of manufactured chemicals used for non-stick cookware, waterproof clothing, stain repellent carpets and enhancing the effectiveness of firefighting foams.

The PFAS molecules are made of a chain of carbon and fluorine atoms with the carbon-oxofluoride being one of the strongest. These chemicals do not degrade in the environment, and PFAS are often referred to as forever chemicals. As they migrate into the soil, water, and air, they can ultimately find their way into the food chain.

PFASs present a number of health risks including:

- Impacting foetal development.
- Increasing the risk of pre-eclampsia in pregnant women.
- Decreased vaccine responses.
- Development of kidney and testicular cancer.
- Changes in liver enzymes.

Further research is required to understand the extent of the health risks.

There are over 4,500 different types of PFAS compounds, and this list continues to lengthen as more are discovered. The most common are PFOA and PFOS, which have been banned for use in manufacturing in some countries but persist as legacy chemicals in the environment.

For Netheridge, only PFOS has been listed as a PFAS compound that requires removal. There is likely to be more PFOA than PFOS in the effluent given the source of the wastewater and transitional processes in the wastewater treatment. In some tertiary treatment plants, some of the bacteria can exchange the sulphate from PFOS and convert it to its carbonate form (PFOA). According to industry experts, PFOA is currently the only PFAS compound the EA have listed with regards to legislation, but that is under review. The Drinking Water Inspectorate has produced specific guidance with regards to PFOA and PFOS in drinking water. Experts have also commented that perfluorohexane sulfonic acid (PFHxS) is about to come under the spotlight and has been used in far greater quantities over the last 20 years than PFOS, mainly in aqueous film forming foam (AFFF) where it forms a multitude of precursors. This will impact the capacity of the adsorbent process and reduce the throughput.

The required PFOS concentration of 0.0002 µg/l is derived by using the limit of detection assuming worst case in the absence of confirmation of the actual target. The long-term mean defined from Atkins' data set is 0.00065 µg/l . This is lower than the requirement listed in the DWI's guidance on the Water Supply (Water Quality) Regulations 2016 specific to PFOS and PFOA concentrations in drinking water. The guidance values in the guidance document should be considered during discussions with regulatory bodies.

5.6.3.2 Removal performance, design and cost

Detailed GAC design will require a pilot trial to establish removal performance and establish design parameters. From this, a more reliable design can be derived and appropriately costed. This may differ from the current proposed cost. Some designs which require almost complete removal of micropollutants have needed a series of reactors, the first batch acting as a sacrificial step containing reactivated carbon to protect the virgin magnetite in the subsequent series of vessels. EBCT to be confirmed as part bench scale tests.

5.6.3.3 Supply of carbon

There are differing views, amongst industry experts, on the availability of carbon and whether the existing infrastructure in the UK is suitable to accommodate a treatment works in this scale. There is a risk, particularly if GAC is installed at other SRO schemes, that sufficient regeneration capacity is not available. There is no UK market for regeneration of GAC media used in a wastewater environment currently.

5.6.3.4 Waste

The PFOS contaminated carbon (also likely to be contaminated with other PFAS compounds) will require high temperature destruction at 1200°C to destroy the PFAS. The thermal destruction systems will require vast amount of energy to maintain the high temperatures, which will add to the overall carbon footprint of the treatment. If temperatures are not hot enough, hydrochloric acid is formed and there is no guarantee all PFOS will be removed. Industry experts have commented on the limited availability of PFAS contaminated waste disposal facilities in the UK and Europe. Waste disposal facilities in the United States of America (USA) (where PFAS removal is becoming more prominent) are beginning to reject waste contaminated with PFAS due to its legacy if not completely destroyed. Some companies have up to 100 tonnes of waste on site that they cannot currently dispose of.

For comparison, in the United States where PFAS removal and subsequent incineration processes are reasonably well established, the cost to incinerate PFAS contaminated carbon is £275 per m^3. This equates to £182,000 each year in disposal costs. This cost can be as high as £386 per m^3, increasing incineration costs to £255,000 each time the media is replaced.

5.6.3.5 Monitoring

Currently there is no instrumentation that can provide online or continuous monitoring of PFOS concentrations in the wastewater. Indeed, likely target levels are limited by the limit of detection in laboratory-based analysis. Therefore, a breach of permit or failure of removal performance would not be realised until the wastewater is sampled for permitting.

5.6.3.6 Backwash returns

The calculated backwash flow rate per vessel is 229 l/s. Under certain conditions, this is greater than the effluent flow available at Netheridge, so a clean backwash storage tank of 206 m^3 is included to ensure buffering capacity. The GAC backwash requires the largest volume and instantaneous flow of backwash water but operates only twice a day for 15 minutes. Therefore, flow can still be transferred to the point of discharge whilst slowly replenishing the clean backwash water tank.

These losses due to back wash water (406 m^3/day) will need to be included in the total flow diverted to the tertiary treatment plant to ensure 35 MLD is transferred.

GAC backwash water could contain PFOS and other micropollutants when returned to the head of the works. This could lead to an increased concentration and load of PFOS and micropollutants feeding the GAC plant if not completely removed from the process, and the ability of the GAC plant to achieve low PFOS concentrations could be reduced.

5.6.3.7 Sodium hypochlorite

Sodium hypochlorite may be corrosive to metal, causes sever skin burns and eye damage, is very toxic to aquatic life, toxic to aquatic organisms with long lasting effects and contact with acids liberates toxic gases. Sodium hypochlorite should be stored in well ventilated areas and protected against exposure from sunlight.

PPE to protect contact with the eyes, skin and clothing should be worn if there is a need to handle sodium hypochlorite.

A safety shower and eye wash station are included as part of the design.

5.6.4 OPPORTUNITIES

5.6.4.1 Protection against future permits

The provision of GAC could provide future proofing against any tightening of micropollutant permits, particularly with regards to PFAS compounds other than PFOS.

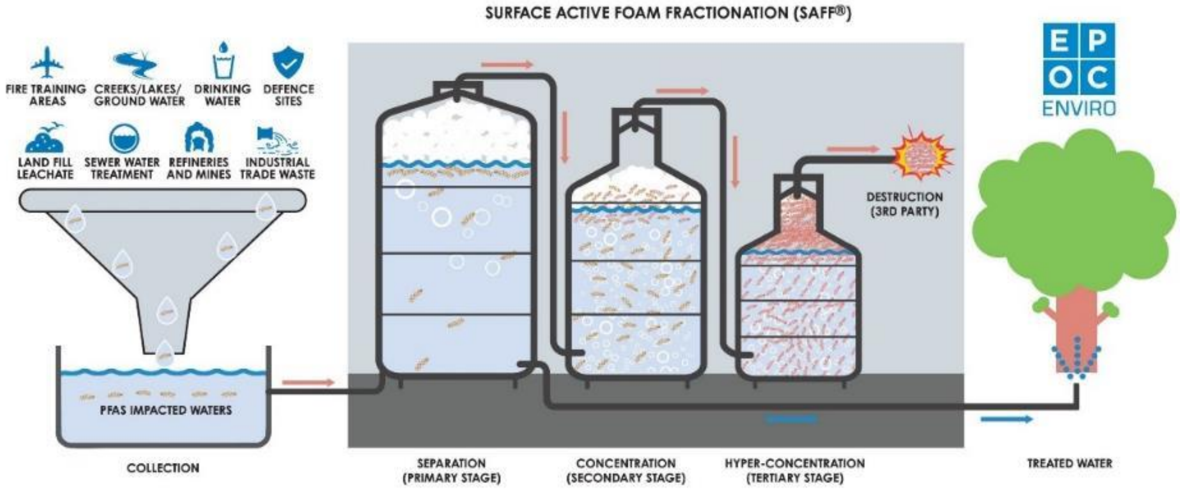
5.6.5 ALTERNATIVE TREATMENTS

A non-media-based solution may be better suited for this application, given the uncertainty of the carbon supply and distribution structure in the UK. Alternative PFOS removal treatment processes have been considered in this section, however, are not yet established at sufficient scale, so GAC has been selected and costed as a solution that will remove PFOS at this scale.

5.6.5.1 Surface active foam fractionation

The surface active foam fractionation (SAFF) process is not employed for flows approaching 35 MLD yet, although a 20 MLD plant has been specified and costed for a site in Europe. This is a modular process that passes the effluent through a series of vessels where bubbles of air are passed through and rise to the surface. As they rise through the vessels, it leads to a concentrated fraction at the top of the column, which passes through to the next vessel (Figure 5-20). The waste is concentrated to two million times less than the flow throughput, which would generate a waste stream of just 18 l/day at Netheridge.

Figure 5-20 - Surface active foam fractionation diagram (EPOC Enviro)



At the pilot plants where SAFF has been installed, the waste streams have been passed onto to research companies to see if it can be destroyed. One company uses a catalytic technology to turn the contaminants into their hydrocarbon counterparts (a dehalogenation process). The contaminated water is pumped into the reactor and in contact with the catalyst the carbon-fluorine (C-F) or carbon-chlorine (C-Cl) bonds are replaced by carbon-hydrogen (C-H) bonds, plus fluorine (F) and chlorine (Cl) anions. No regeneration of the solid catalyst is required, and the technology works better on shorter chain PFAS which are typically more difficult to remove. The catalyst is still in the developmental stage but has received funding from the UK government to support its implementation into industry.

SAFF has been used in the food and oil refinery industry for many years, and at airports to treat firefighting foams that contain PFAS compounds and can remove PFOS to non-detect levels.

With regards to cost, a SAFF process was installed at a plant in Switzerland to treat 4.4 MLD and cost £3.5 million. Assuming the technology is scalable, this would increase to £28 million, although the actual cost may be less due to economies of scale and the generally cheaper material cost in the UK. However, the high CAPEX cost would be offset by the cost of handling carbon and the disposal.

5.6.5.2 Powdered Activated Carbon

Powdered activated carbon (PAC) has been proven to be more suitable to removing shorter chain PFAS compounds (if they are developed from the ozone process).

Powdered activated carbon can be used in combination with ballasted flocculation and sedimentation to achieve phosphorous and organics removal as well as an adsorbent process to remove micropollutants.

Veolia's ballasted flocculation process, Actiflo®, has been widely used for tertiary treatment and water reuse for flow volumes between 0.1 and 60 MLD. The process can be combined with PAC to provide a physical adsorption stage for the removal of PFAS and other calcitrant micropollutants as shown in Figure 5-21.

Figure 5-21 - Schematic of Veolia's Actiflo® Carb

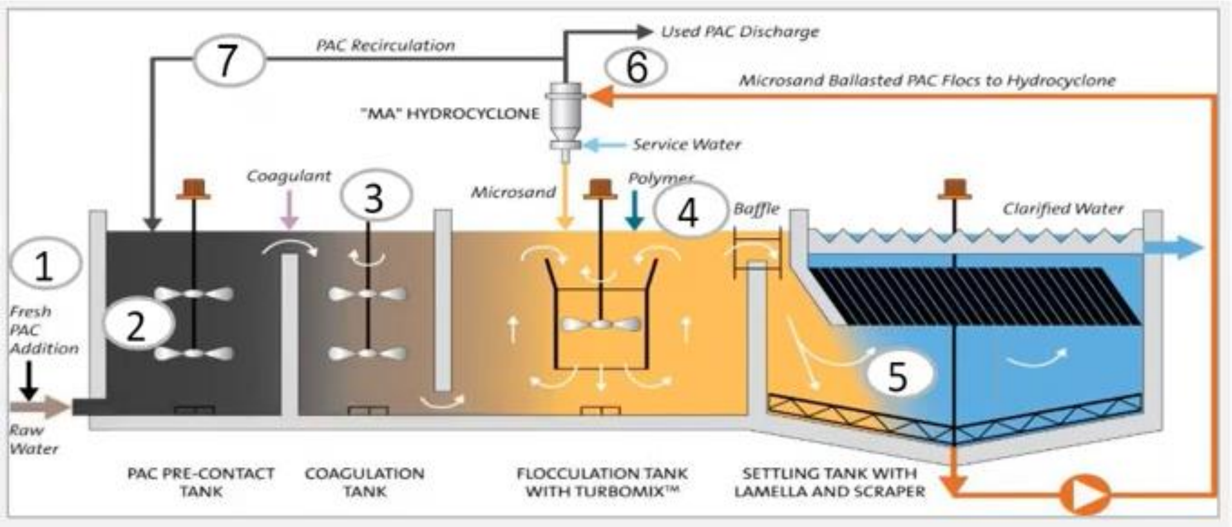
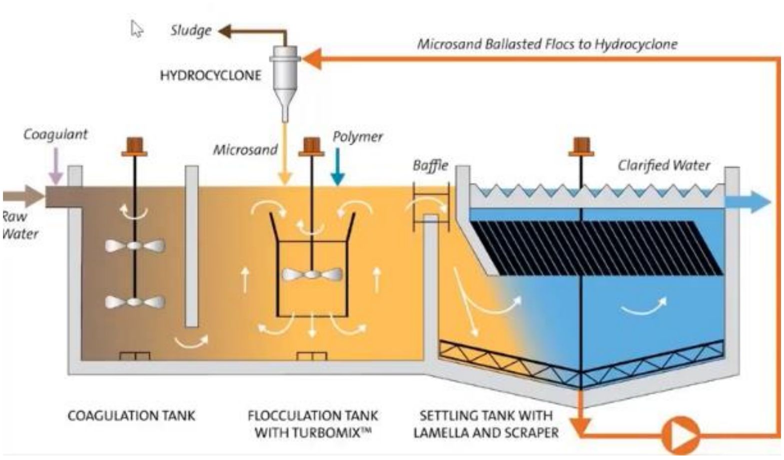


Figure 5-21 Key:

1 - Fresh PAC
2 - PAC Contact Tank
3 - Coagulation
4 - Flocculation
5 - Lamella Clarifier
6 - Cyclone
7 - PAC Sludge Return

To ensure the removal of PFAS to almost non-detect levels, a conventional ballasted flocculation unit (Figure 5-22) can be installed upstream of a ballasted flocculation with PAC addition in the second (Figure 5-21). The first unit provides chemical coagulation for the removal of TSS and flocculable compounds, as some of the PFOS is contained within the organic material, and the second a physical adsorption process to act as a polishing stage.

Figure 5-22 – Schematic of Veolia's Actiflo® ballasted flocculation unit



The first unit can absorb fluctuations in flow and loads, has operational flexibility with regards to start ups and shutdowns, can operate at up flow velocities in the settling tank of 60 to 150 m/hr (compare to 0.5 to 1.5 m/h of a conventional clarifier) and is fully automated with remote operation.

In the second unit, the hydraulic residence time in the coagulation tank of the carbon dose is typically 5 to 10 minutes, and the clarifier has a rise rate of 30-40 m/hr. Whilst the high-rise rate is advantageous, if the lamella clarifier is not operated correctly there is a risk of PAC carry over into the discharged effluent. The concentration of PAC is typically between 1000 and 3000 mg/l carbon.

Ozone can also be added into the PAC contact tank to further improve adsorption capacity and remove the hydrophilic compounds that cannot be removed by carbon adsorption alone. Studies proved the addition of ozone did not lead to the development of toxic bromate because the ozone (oxidant) is directly mixed with the PAC (reducing agent) so there is no time to produce any ozone by products.

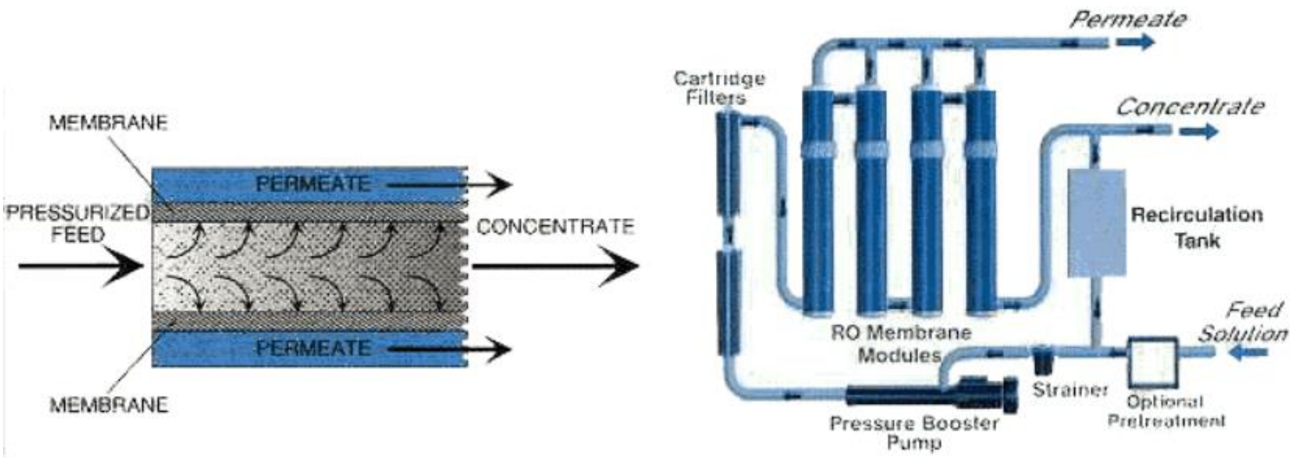
This option could combine the proposed CoMag™, ozone, BAF and GAC processes into a simpler two stage process. The CoMag™, Ozone, BAF and GAC processes have been considered as a multibarrier approach for the purpose of a feasibility study, but the combination of coagulation, flocculation and adsorption in one process could provide an attractive alternative, subject to removal performance.

As is applicable with the GAC, the spent powdered carbon will require replacement and exposure to high temperatures to destroy the PFAS. The majority of the PAC slurry is recycled back into the process, with approximately 5% removed as waste, depending on the mass of fresh PAC added at the beginning of the process. Typical sand losses are 1 g/m^3, therefore a daily top up of 35 kg/day would be required.

Membrane Filtration

Membrane filtration processes such as reverse osmosis (RO) could be used instead of ion exchange for metals removal. Membrane filters are defined by their pore size (microfiltration (0.1 micrometre (µm)) / ultrafiltration (0.01 µm)). The membranes can be arranged as pressure vessel systems or submerged systems. Pressure vessel modules may be more suitable for this arrangement because they can operate within a large pressure range to handle variation in influent quality and process upset. Broken fibres are easier to maintain in pressure vessel systems than submerged systems.

Figure 5-23 - Reverse osmosis cross flow filtration and basic arrangement (The Merit Partnership)



Direct filtration is not recommended, any filtration process would need to be downstream of the GAC plant to prevent excessive fouling. The proposed configuration could be rearranged to accommodate a series of membrane filtration units that decrease in pore size (for example microfiltration followed by reverse osmosis).

An important consideration for membrane technology is the handling of the concentrated waste stream. Backwashing can be performed with a flush of clean water or a flush with chemicals (chemically enhanced backwash), and waste volumes are typically 5% to 20% of incoming flow (up to 7,000 m^3/day). This backwash volume will be concentrated with micropollutants, and if recycled to the head of the works, will lead to a reservoir of micropollutants within the wastewater treatment plant. Any PFOS within the waste stream would still require destruction for it to be completely removed. If the waste volume is as large as 20%, then feed through the process would need to be 20% greater (44 MLD), which would add additional CAPEX and OPEX costs to the scheme. A report by Concawe on the use of RO for PFAS removal defined an energy requirement of 0.4 kWh/m^3 for the RO plant. At 44 MLD, this equates to 17,600 kWh per day. At £0.17/kWh, this would equate to pumping costs of just under £3,000/day or £1.09 mill per year. The RO effluent may also require remineralisation prior to discharge.

Membrane technology has been discounted at Netheridge primarily because of the requirement to handle the waste stream, but membranes located closer to the point of use (the Thames catchment) could be an attractive alternative if the concentrate waste stream can be safely handled.

The concentrated waste stream could be treated by the catalyst described in section 5.6.5 for onsite PFAS destruction, however the development of the technology is still in its infancy and is not ready to be employed at an industrial scale at this time.

5.6.6 CARBON IMPACT

The use of GAC for PFOS removal as opposed to reverse osmosis removes the need for high energy pump requirements to overcome the osmotic pressure within the membrane. Higher water pressure increases the contaminant removal of the filters as well as producing less wastewater. Open vessels instead of pressure vessels have been selected to remove the requirement for interstage pumping from the BAF units to the GAC plant. The GAC plant can be fed by gravity from the high-level outlet of the BAF units. The selection of a 2m bed depth instead of 2.5m to 3m will lead to some

The SAFF process for removing PFOS does require aeration which makes the process more energy demanding, but the highly concentrated waste stream could offset the carbon impact associated with transportation of the carbon media and incineration.

GAC has been selected because of its proven ability in the water industry to remove PFOS rather than the innovative SAFF process that is still in its infancy.

A carbon depth of 2 m has been selected based on industrial experience. Whilst using a carbon depth of 2.5 m to 3 m (compliant with STW design manuals) will reduce the number of vessels to provide the same EBCT of 20 minutes, it will reduce the head between the upstream BAF unit and could facilitate the requirement for an interstage pumping station. The use of 2m, an established bed depth in industry, helps to promote a gravity fed system.

5.6.7 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

Confirmation of the requirement to remove PFOS to non-detect levels.

Pilot plant trials to confirm the effectiveness of PFOS removal by GAC and how it performs downstream of the proposed treatment technologies.

Detailed analysis into the impact of returning backwash water to the head of the works.

5.7 SLUDGE TREATMENT

A full investigation into the existing sludge thickening facilities has not been undertaken, but the capacity has been discussed with Netheridge’s operations team, who concluded that there is very little spare capacity.

Therefore, it is proposed to provide a new sludge thickening plant to handle sludges derived from the tertiary treatment plant. The capacity of the existing SAS handling process will require detailed assessment at Gate 3 to determine if it can handle additional sludge from the primary ferrous sulphate dosing, or whether additional capacity will need to be provided.

5.7.1 SLUDGE THICKENERS

1,032 m^3 of wasted sludge from the CoMag^{TM} plant at <1% dry solids will be fed to 3no. thickeners from a storage tank located close to the CoMag^{TM} plant. Backwash water from the BAF and GAC processes will be returned to the head of the works rather than sludge thickening. The sludge will be dosed with polymer and thickened to 5% dry solids by the thickeners. The flocculated sludge will flow through a porous filter belt, with thickened sludge collected at the end. The thickened sludge will be stored in a thickened sludge storage tank before transfer to the existing sludge handling system for treatment with another indigenous sludge. Filtrate will flow by gravity to the dirty backwash water tank and returned gradually to the head of the works.

Figure 5-24 - Typical gravity belt thickener operation (BDP industries)

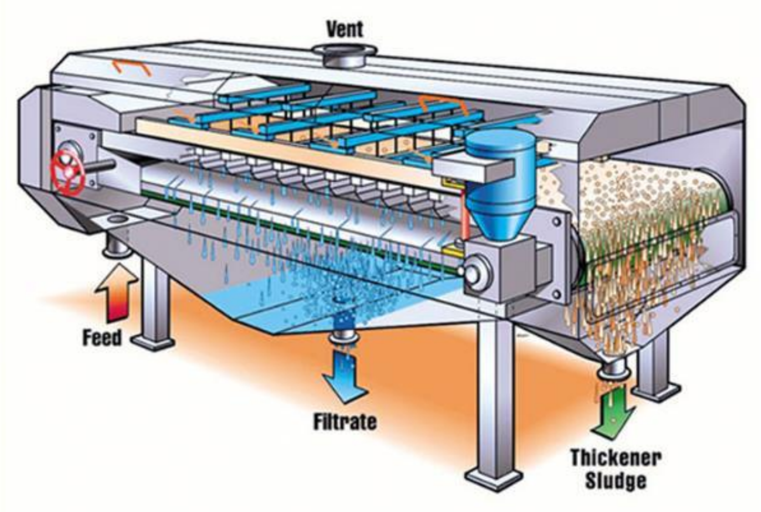


Table 5-15 – Sludge treatment high level design parameters

Parameter	Value	Comment
Sludge Source	CoMag™ plant	
Sludge Volume	1,032 m^3/day	
Feed Sludge concentration	0.80 % dry solids	
Sludge dry solids mass	8,256 kg/day	
Thickened sludge concentration	5% dry solids	
Unit type	Alfa Laval AS-H	
Number of thickener units	3	
Hours of operation per day	8 - 12	Assumption
Belt thickener wash water requirement	4.5 m^3/hr per thickener	
Total wash water volume	162 m^3/day	
Polymer dose	10 kg/ Tonnes dry solids (TDS)	Active ingredient. Assumption

Parameter	Value	Comment
Polymer consumption	83 kg/day	Active ingredient
Potable water requirement	28 m^3/day	Assuming a 0.3% polymer solution is used

It is proposed that tertiary treatment plant effluent will be used for wash water in the thickener unit. This is to prevent starvation of the existing wash water network.

5.7.2 POLYMER DOSING

Polymer required for the thickening plant will be provided by a dedicated polymer dosing kiosk, separate to the CoMag™ polymer dosing kiosk. Polymer will be dosed at 10 kg/TDS at a 0.3% solution strength (83 kg/day). A bulk bag make up system may be preferred for this demand. A potable water supply will be required for the makeup, and a final effluent supply for the carrier water. The final effluent supply will be taken from the tertiary treatment effluent rather than tying into the existing system which struggles to meet the demand for the main site.

5.7.3 RISKS

5.7.3.1 Filtrate returns

A full analysis of the impact of filtrate return to the head of the works has not been undertaken. This should be undertaken and may lead to a requirement for a filtrate storage tank to protect against hydraulic overloading of the PSTs or premature storm spills. The intention is to operate the thickening plant over a prolonged period to minimise loading issues. A large, unthickened sludge storage volume is included to this end.

5.7.3.2 Polymer make up

As with the CoMag™ polymer make up, a large volume of potable water will be required to make the polymer solution (27 m^3/day). The precise availability of potable water on site is unknown, but the operations team have highlighted issues with the existing demand on site and the supply to a cluster of houses close to the site boundary. An upgrade to the existing potable water network may be required.

5.7.3.3 Polymer handling hazards

Powdered polymer has been assumed for this project. Powdered polymers are generally classed as non-hazardous, although can cause irritation to eyes. Aqueous solutions of polymer, or polymer powders that have become wet, render surfaces extremely slippery.

5.7.3.4 Sludge increases

Given the uncertainty of how much sludge will be generated, these processes are operated in series, it is recommended that the system undergoes a trial to confirm sludge increase.

Further assessment of the impact of changes to the stirred specific volume index (SSVI) or volatile suspended solids (VSS) on the ASP should also be made.

5.7.3.5 Struvite

Additional phosphorus loads in sludge can lead to the precipitation of struvite crystals to form which block pumps and pipework, leading to additional maintenance demands and shutdown periods. Struvite can be removed using specialised cleaning agents to descale pumps and pipework but may require a pre-treatment stage to prevent formation.

5.7.3.6 Thickened sludge pumping

Pumping thickened sludge from the new thickeners to the existing sludge handling area will require long lengths of pipework which could have a high energy demand and high maintenance requirements. The position of the thickener plant could be changed to increase the length of pumping unthickened sludge, reducing the pumping distance of thickened sludge. Or the existing thickened sludge plant could be increased, and unthickened sludge pumped from the SRO treatment process.

5.7.3.7 Final effluent water supply

The belt thickener unit will require wash water to keep the belts clean. A total of 162 m^3/day could be required. Final effluent water may also be required for the polymer carrier water, which could be as much as 650 m^3/day (15 l/s if the process is operational for 12 hours).

An independent wash water supply is considered to meet the demand.

5.7.4 OPPORTUNITIES

5.7.4.1 Reduction in sludge volume

The volume of sludge produced is based on the CoMag™ process receiving a high concentration of suspended solids from the MBBR process as a worst-case scenario. If the MBBR performs better than expected with regards to solids carry over or is deemed surplus to requirement because the existing ASPs can be upgraded, this will significantly reduce the sludge production - potentially by up to 10 times. This reduction in sludge volume could lead to the utilisation of the existing sludge handling facilities rather than constructing new.

5.7.5 CARBON IMPACT

Powdered polymer has been selected to reduce the carbon impact associated with chemical deliveries.

Gravity Belt thickeners have been provided to thicken the sludge and reduce the volume pumped to the existing sludge handling facilities for treatment. These are a low energy process compared to centrifuges.

5.7.6 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken

Undertake a detailed review into the capacity of the existing SAS handling process to determine if it can receive additional sludge generated from the primary ferrous sulphate dosing, or whether additional capacity will need to be provided.

Undertake a detailed analysis of the effects of returning filtrate to the head of the works.

To remove the requirement to pump thickened sludge long distances, review the capacity of the existing sludge thickening process to determine whether unthickened sludge from the tertiary process can be accommodated. If it cannot, consider increasing the existing plant capacity.

5.8 ALKALINITY

The availability of alkalinity across the site was reviewed and the calculated residual in the final effluent was 47 mg/l. This can be consumed during tertiary treatment processes and with the addition of ferric compounds. Any alkalinity credit as a result of denitrification was discounted. Additional alkalinity was not deemed to be required at this stage of the design but would require review during pilot plants.

5.9 TREATMENT SUMMARY

The technologies proposed are not selective in terms of what they will remove. Table 5-16 summarises the primary treatment technology and where partial removal can be achieved.

Little or no removal
Partial Removal
Effective Removal

Table 5-16 – Process Unit Determinant Removal

Determinant	Ferrous Sulphate Dosing	MBBR	CoMag™	Ozone	BAF	GAC
Ammonia						
Total Phosphorus						
2-4, dichlorophenol						
Chlorothalonil						
Nonylphenols (4-nonylphenol technical mix)						
Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol)						
Perfluorooctane sulfonic acid and its derivatives						

The extent of the partial removal will need to be determined by pilot plant trials, as the effluent from each process unit will affect the downstream process unit, changing the removal efficiencies stated in literature. This is often driven by lower removal efficiencies as the applied concentration reduces. In some cases, for example achieving PFOS removal using ozone, removal is achieved by adjusting the pH or adding an iron-oxide catalyst, so would change the design of the process provided.

Table 5-17 – Option 1 and 2 Removal Performance

Determinant	Primary Removal Technology	Required Removal (Avg/Max) (%)	Maximum Achievable Removal	Comment
Ammonia	MBBR	0% / 15%	92%	
Total Phosphorus	Primary Ferrous Sulphate Dosing CoMag™	70% / 70% 83% / 90%	75% 95%	Design parameters will require confirmation with pilot plants
2-4, dichlorophenol	Ozone	9% / 60%	40% - 98%	Based on data from water treatment processes at laboratory scale
Chlorothalonil	Ozone	0% / 0%	60%	Based on data from water treatment processes at laboratory scale
Nonylphenols (4-nonylphenol technical mix)	Ozone	93% / 96%	100%	Based on data from water treatment processes at laboratory scale
Octylphenols (4-(1,1',3,3'-tetramethylbutyl)phenol)	Ozone	9% / 50%	83%	Based on data from water treatment processes at laboratory scale
Perfluorooctane sulfonic acid and its derivatives	GAC	70% / 80%	99%	Based on data from water treatment processes at laboratory scale

High removal efficiencies of the micropollutants can be achieved according to evidence seen at pilot plant scale, but the design parameters for Netheridge must be defined by its own pilot plant trials and could be subject to change compared to the sizing provided in this report. As stated above, removal efficiencies may decrease at lower feed concentrations.

5.10 RESILIENCE

Table 5-18 summarises the consequences of process units being unavailable at various stages of the treatment process. The required resilience is to be outlined in more detail at Gate 3 depending on the requirements of STT.

Table 5-18 – Option 1 and Option 2 Treatment Train Resilience

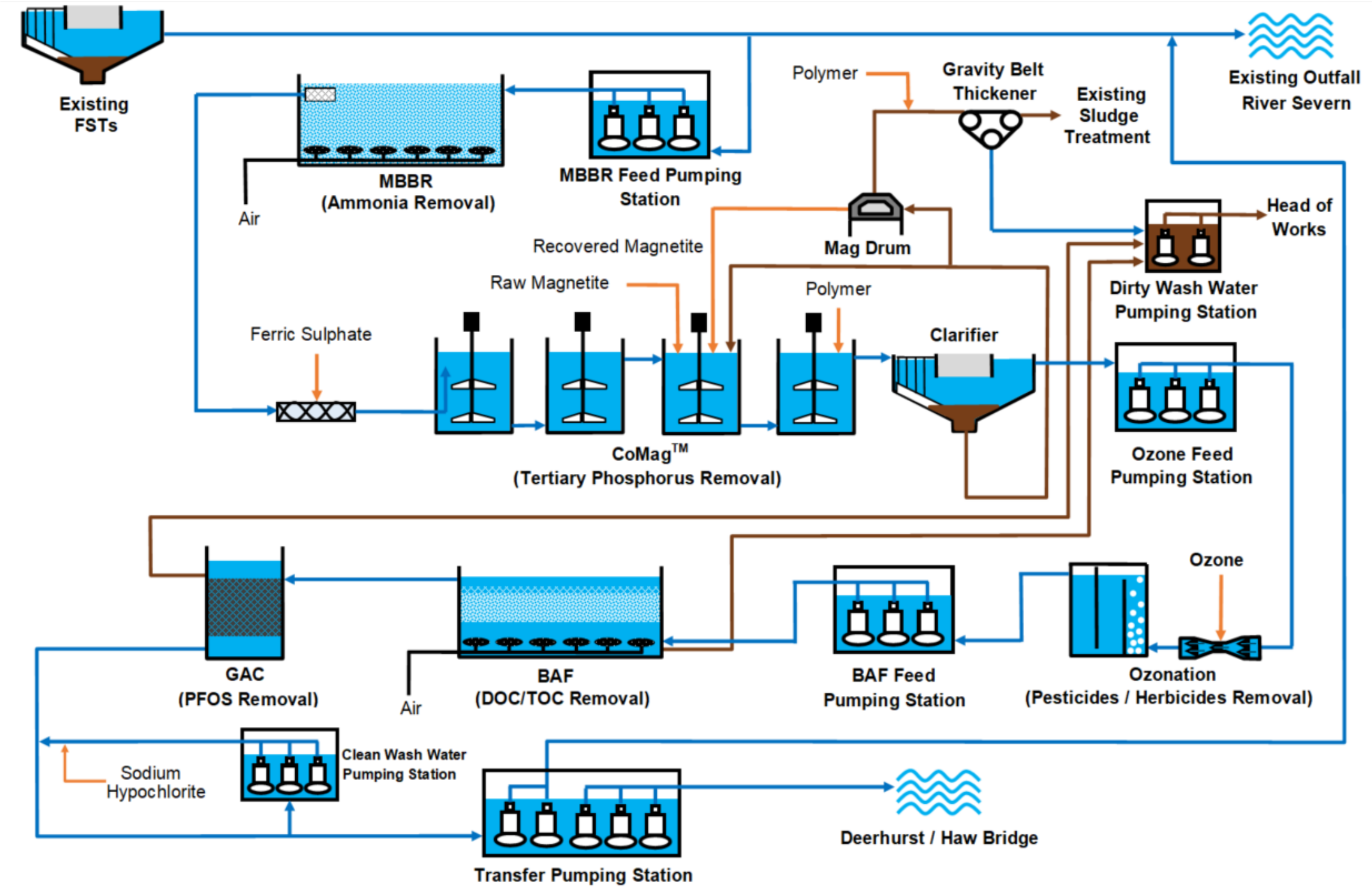
Technology	Purpose	Number of Units	Consequence if one unit is offline
Ferrous Sulphate Dosing	Primary Phosphorus Removal	2 (Duty Standby Dosing Units)	■ Standby dosing system available. ■ 35 MLD throughput unaffected.
MBBR	Tertiary Ammonia Removal	■ 5no. MBBR units ■ Duty/Assist/ Standby Blowers	■ Residence time at FFT decreased from 2 hours to 1 hr 37 min with one MBBR unit offline. ■ FFT reduced from 47.5 MLD to 38.4 MLD. ■ Standby Blower will start in the event of a duty blower failure ■ Ammonia Loading increases from 0.05 kg/m³/d to 0.06 kg/m³/d with one MBBR unit offline. ■ BOD Loading Increases from to 0.12 kg/m³/d to 0.14 kg/m³/d with one MBBR unit offline.
CoMag™	Tertiary Phosphorus Removal	■ 1no. CoMag™ Treatment stream. ■ 1no. Duty Clarifier ■ Duty standby chemical dosing pumps ■ Duty standby sludge pumps ■ Duty sludge transfer tank	■ CoMag™ Tank or mixer failure can impact on final effluent quality. ■ No change in FFT throughput but will impact on final effluent quality. ■ Standby pump will start in the event of a duty chemical pump failure. ■ Standby pump will start in the event of a duty sludge pump failure. ■ Failure of the magnetite recovery system will inhibit sludge to waste, process will remain online. ■ Failure of the clarifier scraper will affect magnetite recovery and process performance ■ High level in the sludge to waste tank will inhibit the sludge to waste process and effect process performance.
Ozone Feed Pumping Station	Pump CoMag™ treated flow to the Ozone units	■ Duty/Assist/ Standby Pumps	■ • Standby pump will start in the event of duty pump failure
Ozonation	Pesticides/ Herbicides Removal	■ Ozone Storage Tanks ■ Duty/Duty Ozone Generator ■ Duty Ozone injection System	■ Failure of an Ozone generator will affect pesticide/herbicide removal by halving the amount of ozone that can be generated. ■ Contact time at average flow decreases to 3 mins 45 s.

Technology	Purpose	Number of Units	Consequence if one unit is offline
		■ Duty/ Standby Catalytic Ozone Destruction System ■ Duty Cooling Water Supply ■ 4no. Ozone Contact Tank	■ Contact time at FFT decreases to 2 mins 45 s. ■ Average flow decreases from 35 MLD to 26.2 MLD to maintain 5 mins contact time. ■ Failure of the Ozone injection system will affect pesticide/herbicide removal. ■ Failure of the duty ozone destruction system will initiate the start of the standby ozone destruction unit. ■ Failure of the cooling water supply will lead to a failure of removing excess heat and risk overheating of the generators. The process will be inhibited in this event.
Ozone Feed Pumping Station	Pump Ozone treated water to the BAF units	■ Duty/Assist/ Standby Pumps	■ • Standby pump will start in the event of duty pump failure.
GAC	PFOS Removal	■ 11 no. GAC Units. 10 online at any time. ■ Duty Sodium hypochlorite dosing cube	■ EBCT will decrease from 27 mins to 24 mins at average flow with one vessel out of service ■ EBCT will decrease from 20 mins to 18 mins at FFT with one vessel out of service. ■ FFT decreases from 47.5 MLD to 42.8 MLD. ■ Process impact unknown but could affect final effluent quality. ■ Failure of the sodium hypochlorite dosing pump will increase risk of biological activity in the GAC plant.
Clean Backwash water tank	Provision of backwash water to the GAC Plant	■ Duty/Assist/ Standby Pumps	■ Standby pump will start in the event of duty pump failure. ■ Failure of the clean wash water tank will inhibit backwashing of the GAC plant.
Dirty Backwash water tank	Transfer of GAC backwash water, BAF sludge and sludge thickener liquors to the head of the works	■ Duty/Standby Pumps	■ Standby pump will start in the event of duty pump failure.

Technology	Purpose	Number of Units	Consequence if one unit is offline
Sludge Thickener	3no. Gravity belt thickeners to thicken CoMag™ sludge to 5%. Polymer addition to aid flocculation and improve thickening.	■ Duty/Assist/ Standby Sludge Thickener Feed Pumps ■ Duty/ Standby Thickened sludge transfer pumps ■ 3no. Sludge Thickener Units ■ Duty/ Standby Polymer Dosing Pumps ■ 1no. Thickened Sludge Holding Tank and mixer	■ Standby pump will start in the event of duty thickener feed pump failure. ■ Standby pump will start in the event of duty thickened sludge pump failure. ■ Standby sludge thickener will start in the event of a duty thickener failure. ■ Standby polymer dosing pumps will start in the event of duty polymer pump failure. ■ Failure of the thickened sludge tank mixer will not cause system shutdown, but risks rat-holing.

5.11 PROCESS SCHEMATIC

Figure 5-25 - Option 1 and Option 2 Process Schematic



6 TREATMENT APPROACH 2 - RIVER SEVERN EAST CHANNEL (DISCHARGE OPTION 3)

For discharges into the East Channel, no screening exercise or dispersion and dilution modelling has been undertaken to determine permit requirements. Therefore, it is assumed the same sanitary permit for options 1 and 2 will be applicable for option 3, with an additional requirement for metals and additional micropollutants removal (Table 4-3 and Table 4-4).

6.1 TREATMENT REQUIREMENT

It is assumed that the ammonia and total phosphorus permit requirements for option 1 and option 2 will apply to option 3. The same MBBR and CoMag™ processes will therefore be provided. Additionally, the ozone, BAF and GAC processes intended to remove pesticides and PFOS have been included for option 3. The same caveat applies in that the effluent will need to undergo pilot plant trials to confirm removal efficiency of the additional micropollutants listed in Table 4-4. It is also proposed to include the same sludge thickening system and backwash facilities.

Additional treatment is required for option 3 to remove metals. The CoMag™ and GAC process may remove some of the metals listed in Table 4-4, but until the effluent is subjected to a pilot plant, the removal efficiency of these is unknown. Therefore, an ion exchange process is proposed downstream of the GAC process to provide a polishing stage of metals removal prior to transfer.

6.2 ION EXCHANGE

Effluent from the GAC process will flow via gravity into the ion exchange feed pumping station. Duty/Assist/Standby pumps will feed effluent to the top of the ion exchange vessels. The key design parameters for the ion exchange system are shown in Table 6-1.

Table 6-1 – Ion Exchange Design Details

Parameter	Value	Comment
Flow	1,980 m^3/hr	
Metal Concentration	100 µg/l	Assumption based on data received.
Resin Capacity	50 g/l resin	Assumption
Bed flow rate	15 BV/hr	
Resin Volume	132 m^3	
Run length	3.8 years	If online continuously.
Number of lead vessels	10	
Lead vessel diameter	3 m	
Lead vessel bed depth	2 m	

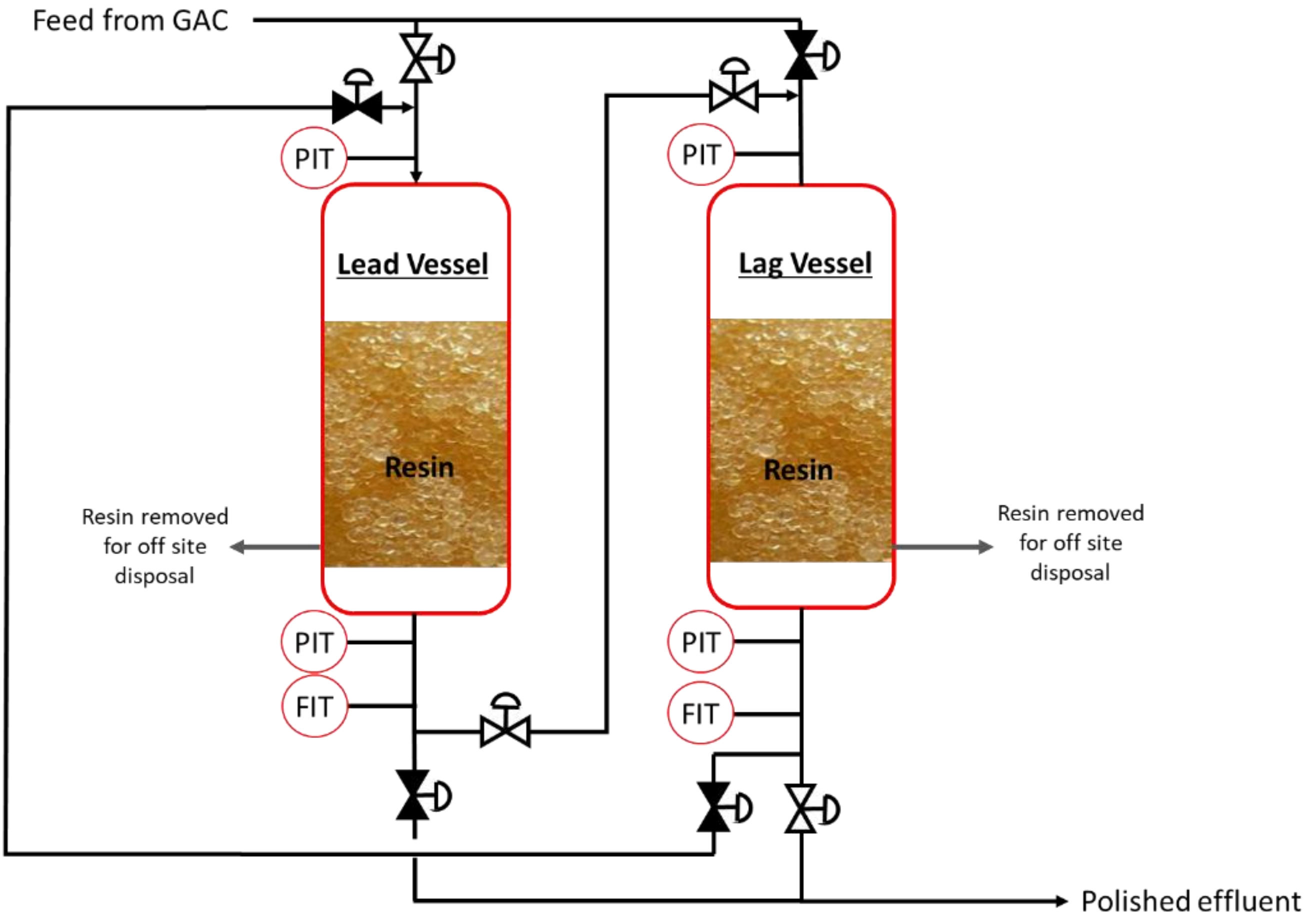
Parameter	Value	Comment
Lead vessel bed volume	14 m^3	
Number of lag vessels	10	
Lag vessel diameter	3 m	
Lag vessel bed depth	2 m	
Lag vessel bed volume	14 m^3	
Total Number of vessels	20	
Total resin volume	283 m^3	

The proposed chelating resins are a class of ion-exchange resins used to bind cations using chelating agents attached to a styrene polymer matrix. These resins have the same bead form and polymer matrix as typical ion exchange resins; however, they form strong complexes with a target ion, have excellent selectivity for metals and work in high total dissolved solids environments. Depending in the chelating agent used, it may be possible to regenerate the resin by disrupting the complex formation using strong acids and caustic, however with the high selectivity and capacity of the resins, it may be more practical as a single use resin.

The proposed vessel layout is as a typical lead/lag configuration, with 10 pairs of two vessels in series, allowing the system to be sampled at the midpoint and when the lead bed hits the change out criteria at the midpoint, the lag vessel becomes the lead vessel. The lead vessel is emptied and filled with new resin and becomes the lag. This configuration allows the resin to be used more efficiently and reduced the risk of exceeding permitted limits.

Figure 6-1 - Ion Exchange Vessels (Evoqua)

Figure 6-2 - Lead/ Lag Ion Exchange Arrangement



The ion exchange process can be bypassed when there is no demand from STT. Treated water from the ion exchange plant will flow by gravity to the transfer pumping station for discharge to the East Channel.

6.2.1 RISKS

6.2.1.1 Unknown treatment performance

The effectiveness of ion exchange to achieve the assumed water quality parameters is unknown and would need to be investigated in pilot plants intended to produce detailed design parameters. The CAPEX and OPEX costs are therefore liable to change.

6.2.1.2 Competition for adsorption sites

Water chemistry is important, other cations will compete for capacity on the resin bead and the pH is crucial in determining resin capacity. To reach very low residual levels in a water, metals may need to be ionized at lower pH. A complete understanding of the water chemistry is necessary, not only the cations and anions, but pH, TOC, TSS, TDS, temperature, oil and grease. Whilst this data is available for current Netheridge final effluent, the concentrations will change through the proposed tertiary treatment processes.

6.2.1.3 Suspended solids fouling

Ion exchange resins can act as TSS filters; however, when the bed becomes fouled with solids, the resin bed will need to be either replaced or regenerated. Therefore, effective TSS removal before the ion exchange process is required if the process is to be economic. TSS loading onto the ion exchange unit should be low given the series of process units upstream but will need to be carefully monitored.

6.2.1.4 Resin replacement

A resin cost of £15/l is assumed based on industrial experience. This equates to a total cost of £4.24mill to completely replace the resin.

If the same waste disposal cost of £275/m^3 used for PFAS contaminated waste (section 5.6.3) is applied to the spent resin, this equates to £77,825 for disposal after replacement. If the higher cost of £386/m^3 is applied, the disposal costs could increase to £109,000. This is based on the cost in the USA to incinerate PFAS contaminated material. The costs in the UK where the PFAS removal industry is less established may be different.

6.2.1.5 First flush

The ion exchange process can be turned off when there is no demand from STT to preserve the adsorption capacity of the resin. However, once brought back online, the first flush may be non-compliant with the proposed permit conditions. Therefore, this should be returned to the head of the works or recycled around the tertiary treatment plant prior to discharge.

6.2.2 OPPORTUNITIES

6.2.2.1 Upstream metals removal

Performing a pilot plant trial on CoMag™ and GAC may prove that the required effluent quality can be achieved without the need for ion exchange. GAC is a proven technology for the removal of metals from water, as is coagulation and flocculation. Ion exchange has been included in this proposal as a polishing stage for robustness.

6.2.3 ALTERNATIVE TECHNOLOGIES

6.2.3.1 Membrane filtration

Membrane filtration can be used for the removal of metals from wastewater but has been discounted for Netheridge due to the requirement to handle a concentrate waste stream of up to 20% of flow, as described in section 5.6.5.

6.2.4 CARBON IMPACT

Unlike membrane filtration processes, the ion exchange process does not require energy intensive pumping to overcome membrane pressures, and there is no requirement to pump away concentrated waste streams because of the provision of single use resin.

The results from pilot plant trials may prove that the upstream CoMag™ and GAC processes remove some of these metals. This would be identified at pilot plant trials. Proof of removal could remove the need for Ion Exchange and the associated carbon impact.

6.2.5 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

Confirmation of the water quality requirements to permit discharge into the East Channel of the River Severn from Netheridge WwTW.

Confirmation of the requirement to remove metals to non-detect levels.

Pilot plant trials to confirm the effectiveness of metals removal by ion exchange and how it performs downstream of the proposed treatment technologies.

6.3 TREATMENT SUMMARY

In addition to the removal or determinants by process technologies included in the treatment train for option 1 and option 2 shown in Table 5-16, Table 6-2 summarises the primary treatment technology and where partial removal can be achieved of the additional determinants for discharge into the River Severn East Channel.

Little or no removal
Partial Removal
Effective Removal

Table 6-2 – Process Unit Determinant Removal

Determinant	**Ferrous Sulphate Dosing**	**MBBR**	**CoMag™**	**Ozone**	**BAF**	**GAC**	**Ion Exchange**
Ammonia							
Total Phosphorus							
2-4, dichlorophenol							
Chlorothalonil							
Nonylphenols (4-nonylphenol technical mix)							
Octylphenols (4-(1,1’,3,3’-tetramethylbutyl)phenol)							

Determinant	Ferrous Sulphate Dosing	MBBR	CoMag™	Ozone	BAF	GAC	Ion Exchange
Perfluorooctane sulfonic acid and its derivatives							
Chromium (III) dissolved							
Glyphosate							
Mecoprop							
Permethrin							
Triclosan							
Cypermethrin							
Dichloromethane							
Hexabromocyclododecane (HBCDD)							
Lead dissolved							
Mercury dissolved							
Nickel dissolved							
Pentachlorophenol							
Terbutryn							
Tributyltin compounds (as tributyltin cation)							
Boron total							
Chloride							
Dibutyl phthalate							

Determinant	Ferrous Sulphate Dosing	MBBR	CoMag™	Ozone	BAF	GAC	Ion Exchange
Diethyl phthalate							
Diflubenzuron							
EDTA							
Fluoride							
Mancozeb							
Maneb							
Sulphate							
Tributyl phosphate (µg/l)							
Triphenyltin (TPT) compounds (as tryphenyltin cation) (µg/l TPT)							

The extent of the partial removal will need to be determined by pilot plant trials, as the effluent from each process unit will affect the downstream process unit, changing the removal efficiencies stated in literature. This is often driven by lower removal efficiencies as the applied concentration reduces. In some cases, for example achieving PFOS removal using ozone, removal is achieved by adjusting the pH or adding an iron-oxide catalyst, so would change the design of the process provided.

In addition to the list of determinants and removal efficiencies shown in Table 5-16, the removal efficiencies of the additional micropollutants for option 3 by the primary removal technology are shown in Table 6-3. The removal percentages have been taken from literature.

Table 6-3 – Option 3 Removal Performance

Determinant	Primary Removal Technology	Required Removal (Avg/Max) (%)	Maximum Achievable Removal	Comment
Chromium (III) dissolved	Ion exchange	36% / 85%	95%	Based on pilot plant investigations under optimum conditions

Determinant	Primary Removal Technology	Required Removal (Avg/Max) (%)	Maximum Achievable Removal	Comment
Glyphosate	Ozone	78% / 72%	>94%	Based on pilot plant investigations under optimum conditions
Mecoprop	Ozone	79% / 80%	>80%	Based on treatment of agricultural waters.
Permethrin	GAC	50% / 40%	99.9%	Based on pilot plant investigations under optimum conditions (pH).
Triclosan	Ozone	50% / 75%	99.7%	At 5 mg/l and 10-minute contact time. Removal efficiency decreases to 70% at 5 min contact time. Up to 95% removal through GAC.
Cypermethrin	Ozone	58% / 44%	75%	Based on pilot plant scale under optimum conditions
Dichloromethane	GAC	50% / N/A	95%	Based on pilot plant scale under optimum conditions
Hexabromocyclododecane (HBCDD)	GAC	82% / 90%	>80%	Based on pilot plant scale under optimum conditions
Lead dissolved	Ion Exchange	N/A / 14%	92%	Under optimum conditions (pH, bed height and mesh)
Mercury dissolved	Ion Exchange	N/A / 23%	>80%	Based on optimum conditions.
Nickel dissolved	Ion Exchange	N/A / 36%	97%	High feed concentration and under optimum conditions. pH will affect removal.
Pentachlorophenol	GAC	13% / 33%	>80%	High feed concentration and under optimum conditions. pH will affect removal.
Terbutryn	GAC	33% / 71%	>80%	Expected efficiency based on the substance's properties and treatment technology mechanism.
Tributyltin compounds (as tributyltin cation)	GAC	4% / 0%	>80%	Expected efficiency based on the substance's

Determinant	Primary Removal Technology	Required Removal (Avg/Max) (%)	Maximum Achievable Removal	Comment
				properties and treatment technology mechanism.
Boron total	Ion Exchange	54% / N/A	91%	Pilot plants typically with a high feed concentration This particular case achieved 0.80 mg/l, target is <0.1mg/l. Actual efficiency likely to be lower.
Chloride	Ion Exchange	49% / N/A		
Dibutyl phthalate	GAC	33% / 80%	>80%	Based on pilot plant scale under optimum conditions
Diethyl phthalate	GAC	0% / 0%	>80%	Based on pilot plant scale under optimum conditions
Diflubenzuron	GAC	0% / 0%	No removal required	N/A
EDTA	Ozone	40% / 70%	Removal information not available	Literature states EDTA can be removed using ozone, but little information available regarding the efficiency.
Fluoride	Ion exchange	1% / 0%	95%	Under optimum conditions and pH. WHO level feasibly.
Mancozeb	Ozone	52% / 86%	Removal information not available	Literature states Mancozeb can be removed using ozone, but little information available regarding the efficiency.
Maneb	Ozone	29% / 80%	>99.9%	Feed concentration of 240 mg/l
Sulphate	Ion Exchange	43% / N/A	93%	Pilot plant with feed of 1000 mg/l sulphate to 70 mg/l. EQS is 57.4 mg/l.
Tributyl phosphate	GAC	75% / 97%	>88%	Based on pilot plant tests under optimum conditions. 1.5 µg/l concentration in a micropollutant mixture.

Determinant	Primary Removal Technology	Required Removal (Avg/Max) (%)	Maximum Achievable Removal	Comment
Triphenyltin (TPT) compounds (as tryphenyltin cation)	GAC	N/A / 90%	99.8%	Based on pilot plant tests under optimum conditions. Feed concentration 100 mg/l TPT.

High removal efficiencies of the micropollutants can be achieved according to evidence seen at pilot plant scale, but many of the stock solutions used in the literature reviewed contained a higher concentration of the micropollutant of concern than the concentration present in the final effluent at Netheridge. As stated above, there is a risk that removal efficiencies may decrease at lower feed concentrations (µg/l concentrations in most cases at Netheridge), therefore the success of reducing the micropollutants to EQS concentrations is yet to be proven. The multi-barrier approach provides a solution designed to improve the removal efficiencies to low concentrations, but success will be determined by pilot plant trials. The results of pilot plant trials may change the design parameters included in this report.

6.4 RESILIENCE

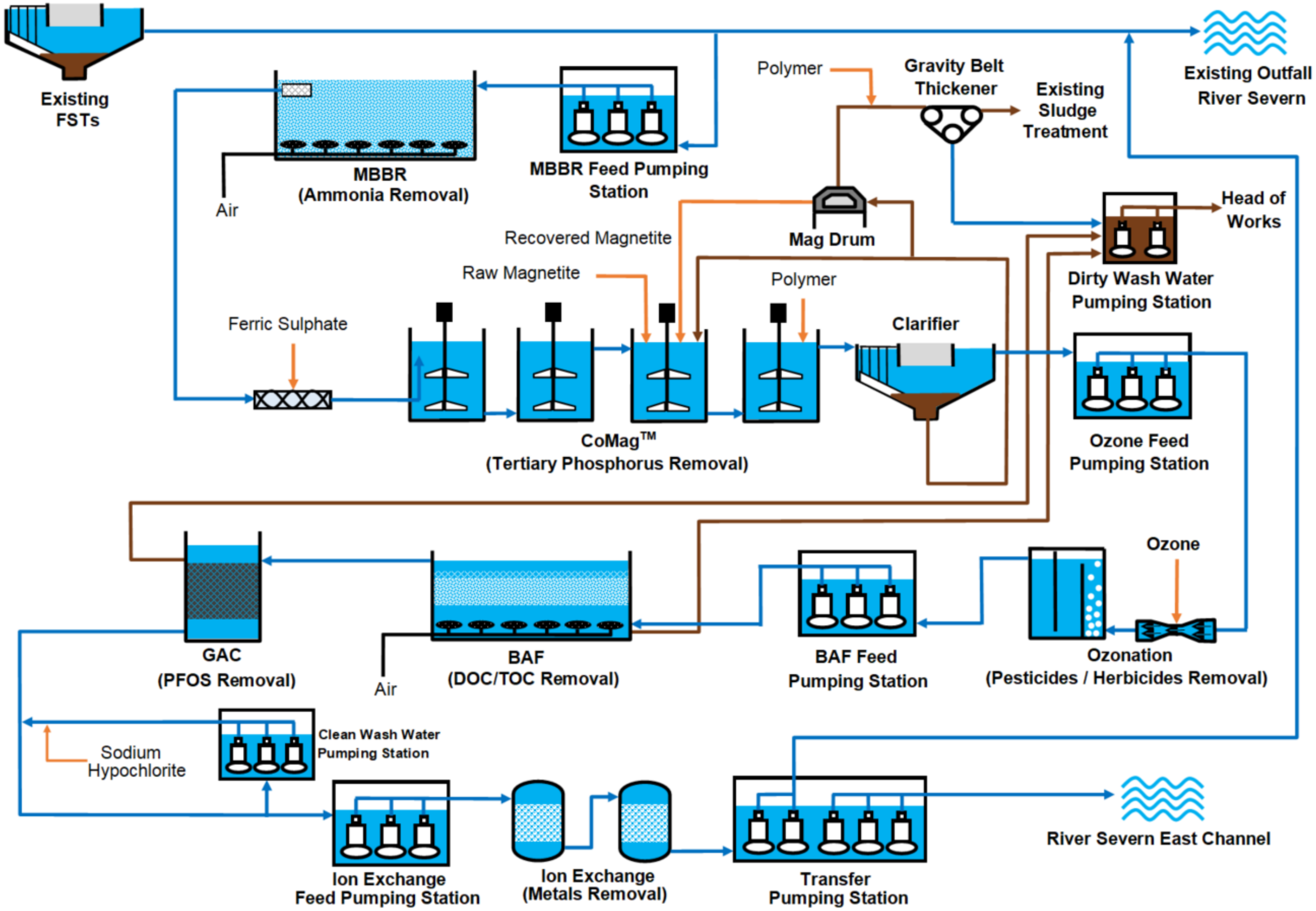
In addition to the resilience of process units shown in Table 5-18, the following is applicable for the River Severn East Channel treatment train. The required resilience is to be outlined in more detail at Gate 3 depending on the requirements of STT.

Table 6-4 – River Severn East Channel Treatment Train Resilience

Technology	Purpose	Number of Units	Consequence if one unit is offline
Ion Exchange Feed Pumping Station	Provision of water to the Ion Exchange Plant	■ Duty/Assist/ Standby Pumps	■ Standby pump will start in the event of duty pump failure.
Ion Exchange	Metals Removal	■ 20no. Vessels (10no. Lead/lag vessels in series)	■ Throughput will increase from 10.3 BV/hr to 11.5 BV/hr at average flow, and from 14 BV/hr to 15.5 BV/hr at FFT with one ion exchange stream offline. FFT decreases from 47.5 MLD to 45.8 MLD.

6.5 PROCESS SCHEMATIC

Figure 6-3 - Option 3 Process Schematic



7 TREATMENT APPROACH 3 – GLOUCESTER AND SHARPNESS CANAL (DISCHARGE OPTION 4)

For discharges into the Gloucester and Sharpness Canal, no dispersion or dilution modelling has been undertaken. The drinking water inspectorate (DWI) have been engaged to discuss discharging into the Gloucester and sharpness canal, but any additional requirements to permit discharge into the drinking water protected area have not been confirmed.

Therefore, in the absence of modelling and confirm permit requirements, it is assumed the same sanitary permit for options 1, 2 and 3 will be applicable for option 4, with a similar requirement for multiple metals and micropollutants removal as option 3 (Table 4-3 and Table 4-4). This is to provide a robust treatment process that can achieve an effluent suitable for discharge into a drinking water protected area. The actual chemicals and their permit values have not been confirmed. Additionally, because the canal is a drinking water protected area, it is assumed there is a requirement for disinfection prior to discharge although this has not been confirmed.

7.1 TREATMENT REQUIREMENT

It is assumed the same ammonia and total phosphorus permit requirements for option 1, 2 and 3 apply to option 4 and so the same MBBR and CoMag™ process will be included.

Additionally, the ozone, BAF and GAC processes that remove pesticides and PFOS will also be applicable for option 4, as will the ion exchange process from option 3. The same sludge thickening system and backwash facilities will also be included.

To provide the disinfection, an inline UV reactor will be provided, downstream of the ion exchange units.

7.2 UV DISINFECTION

Effluent from the ion exchange unit will flow via gravity and be subjected to electromagnetic radiation at 100 nm to 400 nm (ultraviolet light) to disinfect any microorganisms prior to discharge into the canal. UV radiation has been proven to be an effective bactericide and virucide for wastewater without causing the development of by-products that could be toxic. The ultraviolet light penetrates the cell wall of the microorganism and causes the cell death or failure to reproduce by adsorption in the nucleic sites. The effectiveness of the UV depends on system hydraulics, turbidity (which can shield the microorganisms from the UV light), microorganism characteristics and chemical characteristics of the water. Closed vessels are typically used for drinking water applications and are proposed here rather than open channels.

In the absence of water quality data, the assumptions shown in Table 7-1 were made to design the UV treatment:

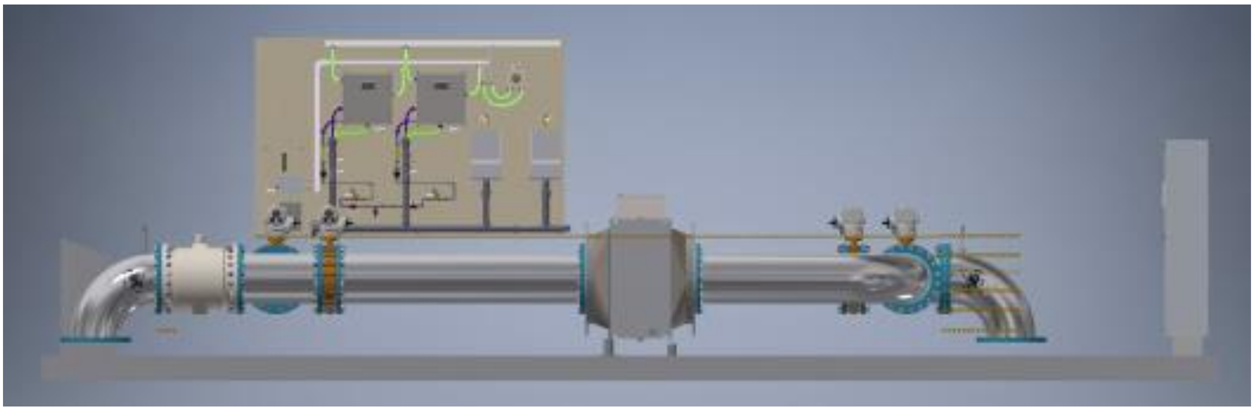
Table 7-1 – UV disinfection specification

Parameter	Value	Comment
Expected Ultraviolet transmittance (UVT)	90%	Assumes worst case in absence of confirmation. The effluent at this point would have been subjected to multiple tertiary treatment processes so should be very clean.
Total Suspended Solids	<5 mg/l	Assumption due to the tertiary treatment processes upstream of the UV.
Discharge Limit	Not confirmed	UV reactor specified to meet drinking water standards
Design Dose	30 millijoules per square centimetre (mJ/cm^2) wall dose	Assumption

A skid mounted UV assembly (factory built and delivered to site) complete with an electromagnetic flow meter is proposed, which will also comprise sampling sinks and back boards. The closed UV reactor, with 48no. 1000-Watt lamps will be contained within a kiosk. A stainless-steel chamber houses the lamps in a configuration perpendicular to the flow. Quartz sleeves isolate the lamps from direct water contact and help to maintain a uniform lamp output. Mechanical wiping of the quartz sleeves helps to maintain performance. The controller processes real time inputs such as flow rate, treatment objectives, UV transmission and operational parameters for data feedback.

Figure 7-1 - Modular UV Chamber (Trojan)

Figure 7-2 - Side view of UV reactor arrangement (Lintott)



Effluent exposed to UV will flow via gravity to the transfer pumping station for discharge.

7.2.1 RISKS

7.2.1.1 Incorrect treatment design

The pathogen destruction requirement is unknown, and the wall dose of 30 mJ/cm^2 has been assumed based on typical doses for water treatment applications. The requirement will need to be determined by testing the final effluent from pilot plant studies, once the water has been subjected to upstream processes. The proposed ozone process may also contribute to some disinfection.

No modelling has been undertaken for the Gloucester and Sharpness Canal. The treatment process proposed is designed to be robust and facilitate a reasonable expectation of CAPEX and OPEX for discharge into the canal. Discussion with regulatory bodies and confirmation of the impact that Netheridge effluent will have on the canal drinking water protected area of the canal, particularly with regards to water safety planning, could lead to a change in design and costs.

7.2.1.2 Drinking water protected area discharge requirements

There is a risk that all equipment used in the tertiary treatment process must comply with drinking water standards to permit discharge into the canal. This could alter the CAPEX costs provided and add construction delays if equipment becomes bespoke.

The concentration of natural organics in the final effluent should be analysed, and the impact this has on water safety planning for water treatment works that will abstract this water reviewed. Natural organics in the feed water to water treatment plants (in this case Purton WTW) can react with disinfectants like chlorine and produce disinfection by-products such as trihalomethanes and haloacetic acids (both carcinogenic).

7.2.2 OPPORTUNITIES

7.2.2.1 Membrane filtration

Nanofiltration or reverse osmosis processes can remove chemical constituents and micropollutants to a high degree as well as provide disinfection. This could provide an attractive solution to combine ion exchange and UV (and potentially GAC) to achieve the same targets, however the concentrate stream will require handling, and may need a microfiltration process upstream to prevent fouling.

7.2.3 ALTERNATIVE TECHNOLOGIES

7.2.3.1 Chlorination

Disinfection by chlorination has been discounted as the effluent will be discharged directly into a water course.

7.2.4 CARBON IMPACT

UV disinfection was provided because chlorine dosing into an effluent destined for discharge to a water course was considered unacceptable. The UV lamps are low-pressure high output lamps to improve electrical efficiency. The lamps have a dimming range of 30% to 100% power to save on operational costs and carbon. Efficiencies have also been made in the lamp design to provide the maximum dose in the smallest footprint (40 to 50% lower than UV-oxidation systems) and the lowest headloss to reduce the requirement for pumping. The control system can process multiple inputs such as flow rate, treatment objectives, UV transmission and operational parameters to compare log reduction of contaminants and compare against real treatment requirements. This then automatically controls the number of lamps required and lamp power settings to optimise energy consumption. High efficiency UV systems intended for potable water are available and could be further investigated if the applied effluent quality is considered to be appropriate.

7.2.5 NEXT STEPS

For Gate 3, it is proposed the following investigations are undertaken:

Confirmation of the water quality requirements to permit discharge into the Gloucester and Sharpness Canal from Netheridge WwTW.

7.3 PURTON WATER TREATMENT WORKS

Option 4 is indirect reuse, and the treatment process has been designed to achieve water of a high quality. The treatment process at Purton water treatment works (WTW), operated by Bristol Water located 16 km downstream along the canal, abstracts 165 MLD of water for treatment and distribution to Bristol. The water treatment works has not been assessed and may already account for some of the chemical species targeted by the planned treatment at Netheridge, effectively meaning the water is treated twice.

The original treatment process at Purton WTW comprised screening, pH control, clarification, rapid gravity filtration, super-chlorination and de-chlorination. Pre and post ozone systems were installed in 1993 with granular activated carbon filtration to improve taste and odour. In 2012 there were further upgrades with the inclusion of ultraviolet treatment, but now only marginal chlorination of the final water is practised. Bulk chlorine dosing systems are provided for Zebra mussel control at the intake and a marginal final dose.

Since analysis and modelling for the Gloucester and Sharpness canal has not been undertaken, the impact of Netheridge final effluent on water quality and treatment at the WTW cannot be assessed. There is a risk that the effluent from Netheridge is being treated to a high quality twice. Once at Netheridge WWTW and then at Purton WTW which adds to operational costs.

Option 4 to supply Purton WTW (Bristol Water) is identified as an opportunity and if this option is selected at Gate 3, then the impact on existing water safety planning risks will be assessed using the ACWG template.

7.4 TREATMENT SUMMARY

The micropollutant removal requirements are assumed to be the same as Option 3, therefore refer to Table 6-2 and Table 6-3 for removal efficiencies. The design UV dose of 30 mJ/cm^2 has been assumed and is achievable using the proposed unit.

7.5 RESILIENCE

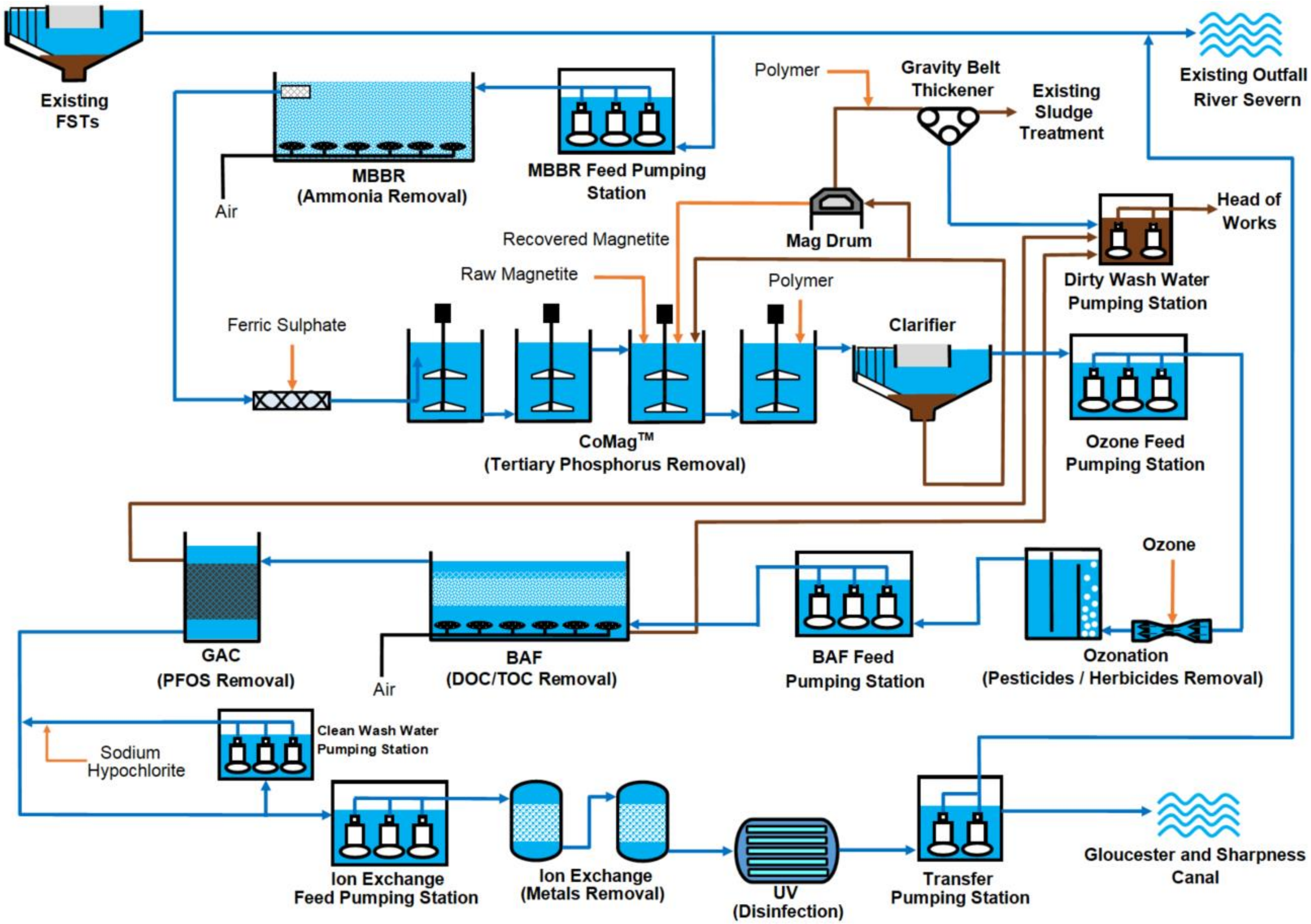
In addition to the resilience of process units shown in Table 6-4, the reliance shown in Table 7-2 is applicable for Option 4. The required resilience is to be outlined in more detail at Gate 3 depending on the requirements of the end user.

Table 7-2 – Gloucester and Sharpness Canal Treatment Train Resilience

Technology	Purpose	Number of Units	Consequence if one unit is offline
UV	UV Disinfection	▪ 1no. UV reactor	▪ Failure of the UV reactor will lead to water that has not been disinfected discharged into the receiving water course.

7.6 PROCESS SCHEMATIC

Figure 7-3 - Option 4 Process Schematic



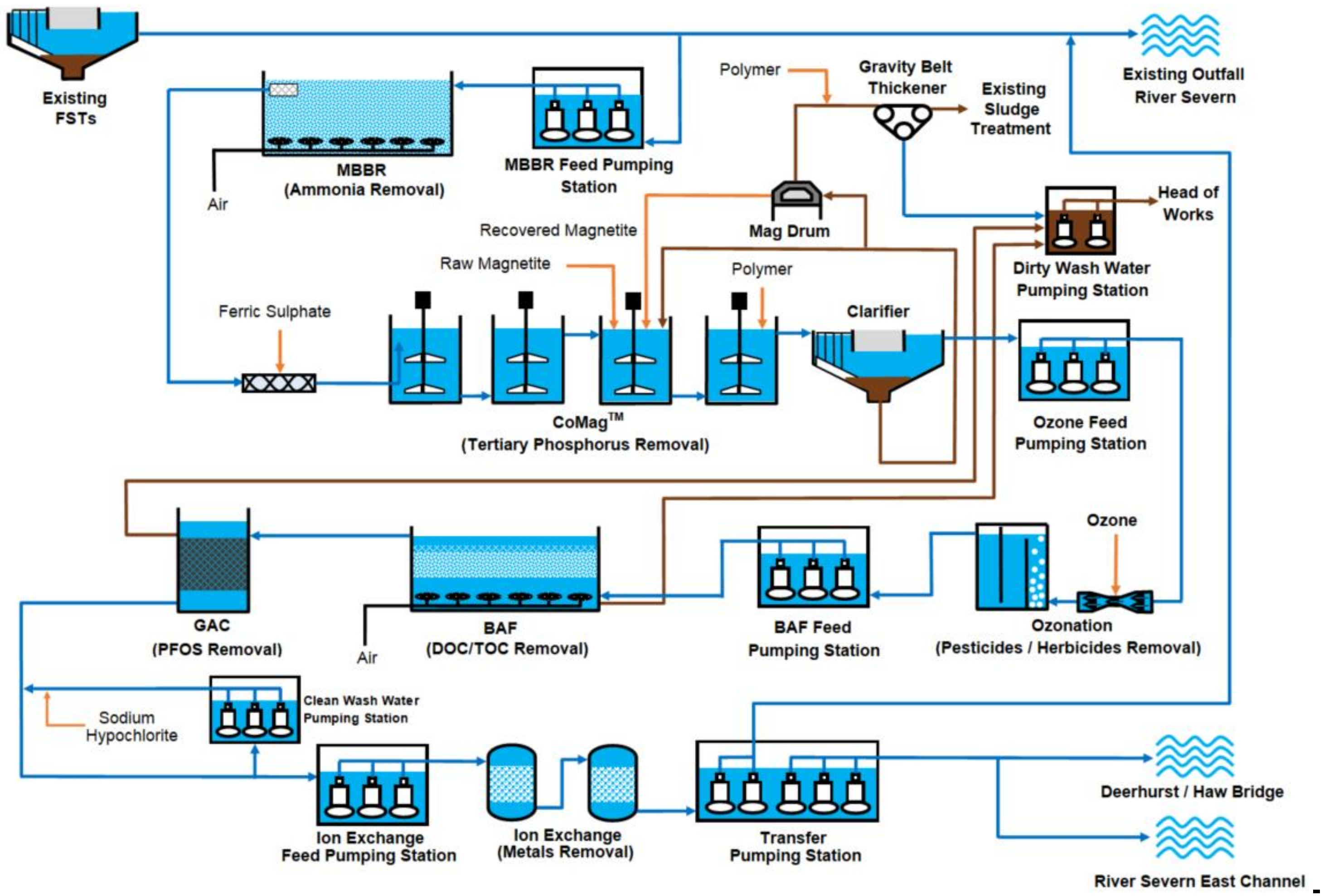
8 TREATMENT APPROACH 4 – SOUTHWEST REGION BRANCH PIPELINE (OPTION 5)

Option 5 comprises additional pipeline for diversion of flow from the main STT SRO pipeline for discharge to the East Channel of the River Severn downstream of the intake for Gloucester Docks.

The treatment process will be identical to option 3.

8.1 PROCESS SCHEMATIC

Figure 8-1 - Option 5 Process Schematic



9 CONTROL PHILOSOPHY

At this stage, the Control Philosophy is a high-level outline only based on the requirements that must be met by the project.

9.1 OVERVIEW

The Netheridge SRO treatment includes the addition of ferric dosing ahead of the existing ASPs; the addition of an STS final-effluent treatment stream that intercepts final-effluent ahead of the River Severn discharge; and a transfer pumpstation to the selected discharge location.

The operational philosophy outlined in Section 4.3 provides the constraints within which the treatment processes must operate. The STS final-effluent treatment is designed to operate within a range of 200 l/s to 550 l/s, although short durations of lower flow would not present a process issue.

In periods where transfer of treated final effluent is not required, the treated final effluent would be returned to the Netheridge outfall line.

The operational philosophy proposed is, for the Netheridge SRO treatment stream to operate to a predetermined diurnal flow profile, the control philosophy aligns with this operating regime. However, as detailed within Section 3.2, the fifteen-minute average flow recorded by the site MCERTS meter occasionally falls below the minimum Netheridge SRO treatment flowrate (of 200 l/s). Consequently, sufficient final effluent will need to be stored at the head of the Netheridge SRO treatment stream to smooth out the periods of below minimum final effluent inflow and permit operation on a fixed diurnal profile.

During periods where transfer of treated final effluent is not required, it is anticipated that the diurnal profile would be adjusted to treat only the minimum required to maintain the processes, i.e., 17.3 MLD.

The STS final-effluent treatment stream will comprise a sequence of treatment units, associated blowers, supporting chemical dosing units, sludge management equipment and primary and inter-stage pumping.

At a high-level, the control philosophy proposed for the pre-ASP ferrous dosing unit is for this to be integrated within the main works control system. The dosing rate will be controlled based on the ASP inflow to maintain the post-ASP final effluent quality to that required by the Netheridge SRO treatment stream. A standalone ferric dosing controller is assumed, supplied as a complete dosing-plant package. This controller would require interfacing to the existing site PLC/ supervisory control and data acquisition system (SCADA) network to integrate control within the main treatment process' control system.

At a high-level, the control philosophy for the STS final-effluent treatment stream is for an independent control system, separate to the main works control system. This approach would isolate the STS final-effluent treatment control from the main works control allowing the main works to operate independently were the STS final-effluent treatment stream to be offline. The approach will additionally isolate the development of the STS final-effluent treatment design from ongoing or future modifications of the main works control system that could otherwise impact upon its design.

Control interlocks or process data would be required for process interfaces between the main works and Netheridge SRO treatment stream. These would pass between the Netheridge SRO treatment control system and main works control system, via wired interfaces (either digital or electric as appropriate to the functional requirement).

Operator interaction with the pre-ASP ferrous dosing equipment would be through local panel HMI or via the existing site SCADA system which monitors the main treatment process.

Operator interaction with the STS final-effluent treatment stream would be through a central motor control centre (MCC) HMI. Interfacing the STS final-effluent treatment control network to the site SCADA would be an option to allow central monitoring and supervisory control by staff in a central control room. Alternately, a standalone SCADA system could be provided for STS final-effluent treatment monitoring and supervisory control by staff in a central control room or other location on site.

The control philosophy proposed would be for a master controller (PLC) to manage the end-to-end Netheridge SRO treatment process and supervise packaged plant which have their own control systems. Netheridge SRO treatment units that would be expected to have their own control systems are, the CoMag^{TM} unit, Ozone unit, BAF unit (including backwash), Sludge Thickener unit, all chemical dosing units, and (for Options 3 and 4) the UV unit. Netheridge SRO treatment units that are expected to be controlled directly by a master controller are, the MBBR unit, GAC unit, Sludge Management, Backwash (GAC units), and (for Options 3 and 4) the Ion Exchange unit. The air-blowers required by the MBBR, and BAF processes are expected to be self-contained machines with integral drives and controls to support blower operation.

The direct control enacted by the master controller would cover, treatment flow control, level protection (dry flow and overtopping), MBBR, GAC backwash control, sludge storage management, return of waste streams to the main works, and transfer of treated final effluent to the River Severn (either at Netheridge or at the STS discharge location dependent upon the STT transfer being called for). For Options 3 and 4, direct control would cover placement of each Ion Exchange vessel into a lead-lag flow orientation and monitoring the vessel's state of exhaustion. The master controller would provide supervisory commands to the package plant and monitor the status of each.

At Gate 2, the Netheridge SRO treatment concept design includes, under Options 1 and 2, 35 directly controlled pumps, 57 directly controlled valves, 8 independently controlled package plants, 5 self-contained blower machines. Option 3 would have an addition of 3 directly controlled pumps, and 80 directly controlled valves over those for Option 1 & 2. Option 3 would have an addition of 3 directly controlled pumps, 80 directly controlled valves, and 1 independently controlled package plant over those for Option 1 & 2.

The concept design is based on these elements being centrally controlled via a single intelligent MCC housing the master controller (PLC) and directly controlled motor drives. This would utilise control equipment currently available to the market and follow STW standards, principally intelligent MCCs, PLC, networked motor drives and actuated valves and HMI. Future project Gate stages should review the concept of a single central MCC verses separate MCCs by process area or stage, as further design information becomes available; the segmentation of the Netheridge SRO treatment control system into distinct process areas with separate MCCs, controllers or input/output (IO) interfaces is not excluded by Gate 2.

All direct controlled drives would be networked to the master controller aligning with STW standard interface philosophy. All direct controlled valves would be networked to the master controller aligning with STW standard interface philosophy. Package plant would be networked to the master controller, where the package's controller has such functionality and the volume of control data warrants this, otherwise discrete hardwired signals would be interfaced to the master controller's IO for basic package monitoring and supervisory control. Process instrumentation associated with directly controlled units would be interfaced to the master controller via hardwired IO. Package plant instrumentation would be connected to the package control panel.

Duty-assist-standby equipment would be segregated across network-nodes to maintain their independent operation. A duty/hot-standby master controller is not proposed at Gate 2 as the overall availability requirement for the Netheridge SRO treatment stream is not, presently, seen to warrant such. Future project Gate stages should review the treatment availability target required by the STS scheme and consider high-availability design approaches where necessary. Detailed Control description

At Gate 2 a cost analysis of the proposed Netheridge SRO treatment and transfer infrastructure has been undertaken. As the basis of this analysis the following control requirements have been used.

The Netheridge SRO treatment and transfer, for Options 1 and 2, comprises the following process units:

- (Main Works) Pre-ASP Coagulant Dosing
- (Netheridge SRO treatment primary stream) Netheridge SRO treatment (MBBR) feed pumpstation 1,
- (Netheridge SRO treatment primary stream) MBBR treatment,
- (Netheridge SRO treatment primary stream) CoMagTM treatment (and Magnetite ballast recovery),
- (CoMagTM sub-system) Coagulant Dosing,
- (CoMagTM sub-system) Polymer Dosing,
- (Netheridge SRO treatment primary stream) Inter-stage (Ozone) feed pumpstation 2,
- (Netheridge SRO treatment primary stream) Ozone Disinfection treatment (and Ozone production / destruction),
- (Netheridge SRO treatment primary stream) Inter-stage (BAF) feed pumpstation 3,
- (Netheridge SRO treatment primary stream) BAF treatment,
- (Netheridge SRO treatment primary stream) GAC treatment,
- (Netheridge SRO treatment primary stream) Treated Final Effluent Transfer pumpstation,
- (BAF & GAC sub-system) Backwashing process,
- (GAC Backwash sub-system) Hypochlorite Disinfection,
- (Netheridge SRO treatment sub-system) Sludge Management (storage, thickening & main works return), and
- (Netheridge SRO treatment sub-system) Liquors Return pumpstation (dirty wash-water, sludge-liquors).

For Option 3 the above units would be extended to include (between GAC treatment and Treated FE Transfer Pumpstation) the following process units:

- (Netheridge SRO treatment primary stream) Inter-stage (Ion Exchange) feed pumpstation,
- (Netheridge SRO treatment primary stream) Ion Exchange treatment.
- For Option 4 the above units would be extended to include (between GAC treatment and Treated FE Transfer Pumpstation) the following process units:
- (Netheridge SRO treatment primary stream) Inter-stage (Ion Exchange) feed pumpstation 4,
- (Netheridge SRO treatment primary stream) Ion Exchange treatment, and
- (Netheridge SRO treatment primary stream) UV Disinfection.

9.2 MAIN WORKS PRE-ASP FERROUS SULPHATE DOSING

The ferrous sulphate storage and dosing would conform to the STW standard. The control philosophy is for a self-contained package plant with independent controller that manages the ferrous sulphate storage and dosing into the RAS stream. The dosing rate is likely to be based upon the inflow to the ASPs and a process measurement is assumed to be available on the existing main works control system. No downstream quality monitor is identified to trim or alarm the dosing rate, this should be confirmed at future Gate stages as the process design matures. The pre-ASP Ferric Dosing controller would be integrated into the existing main works control system for monitoring and supervisory control. Remote monitoring would be through the existing main works telemetry outstation(s).

9.3 NETHERIDGE SRO TREATMENT (MBBR) FEED PUMPSTATION

The final effluent flow to the Netheridge SRO treatment stream (MBBR units) will be controlled to a pre-set diurnal profile. The diurnal profile would be configured at commissioning based upon a future dynamic model of the Netheridge SRO treatment stream and the final effluent flow from the works.

It is proposed that differing diurnal profiles would be used when the STT transfer is being dispatched or not dispatched and based upon the target transfer volume (i.e., sweetening flow). During periods of no STT transfer or sweetening flow, when treated final effluent is returned to the River Severn at Netheridge, it is assumed a flat diurnal profile of the minimum treatment flow (200 l/s) would be set.

At Gate 2, analysis of the Netheridge WwTW MCERTS flow data has identified there will be a requirement for substantial buffering storage of final effluent to provide sufficient volumes to the Netheridge SRO treatment process. Future Gate stages should review the availability of final effluent, inclusive of worst-case dry periods, and determine the necessary volume of buffer storage. Alternate operational philosophies may be considered once a more detailed modelling of inflows and Netheridge SRO treatment response is understood.

The feed pumpstation would operate continuously whilst there is final effluent available within the wet-well. The pumps would be ramped up-down to maintain the flow to the Netheridge SRO treatment stream at the rate defined by the diurnal profile. The proposed pumpstation would have three pumps operating as duty-assist-standby. Downstream level within the treatment (at hydraulic low points) would be monitored and pumped flow reduced or stopped to prevent overtopping within the treatment stream. The primary process measurements would be pumped flow, pumped volume, wet-well level and downstream high-level thresholds.

9.4 MBBR UNIT

The MBBR unit proposed has five separate tanks. The flow will be equally split across all five tanks and no automated flow control is expected. Were one or more MBBR tanks to be taken offline (by hand) then the maximum Netheridge SRO treatment flow from the feed pumpstation #1 would need reducing, it is assumed this would be a manual action via the Netheridge SRO treatment HMI screen / SCADA. A ‘tanks in service’ selection can be included at the HMI.

The effluent flow through the MBBRs is aerated to support the treatment biosphere. Each MBBR would have an independent air-diffusion system connected to a common air manifold. The rate of air flow to each MBBR would be at or above a commissioned minimum (to keep the MBBR media dispersed); above this minimum air flow, the air flow would be controlled to maintain the MBBR dissolved oxygen level at a pre-set threshold. Each MBBR would have automated valves to control the flow of air.

The common air manifold would be maintained at a pre-set pressure, sufficient to drive airflow through the MBBRs, by three Air Blower machines operating as duty-assist-standby. Manifold pressure measurement would provide the primary control measurement for blower speed control. Each blower is assumed to be a self-contained machine with independent controller and instrumentation for safe operation of the blower unit. The Netheridge SRO treatment master controller would modulate the blower output to maintain the air manifold pressure and modulate the MBBR air control valve to maintain the MBBR dissolved oxygen level and media dispersal. Control will include mitigation of the potential system shocks associated with starting and stopping the assist blower.

9.5 COMAG™ UNIT

The CoMag™ treatment unit proposed comprises four reactor tanks in series followed by a clarifier and sludge recycling and magnetite recovery circuits. The technology vendor provides a controller to manage the reactor mixer, sludge recycle rate and waste sludge draw-off with magnetite recovery. The package excludes an MCC, and all package drives are covered by the proposed Netheridge SRO treatment MCC. The package includes the TSS primary process measurement for control.

The CoMag™ treatment includes upstream ferric sulphate dosing to an in-line static mixer and subsequent polymer dosing in reactor 4. These would be supported by separate chemical dosing packages that would conform to the STW standard. Dosing control would be either based upon Netheridge SRO treatment flow (by the master controller) or by a CoMag™ control algorithm (by the CoMag™ controller). This should be confirmed with the technology provider at future Gate stages; both are allowed for under the Gate 2 concept design.

The concept design proposed assumes the master controller will monitor the CoMag™ package through a network connection with the package controller, supplemented by hardwired IO if required.

9.6 (OZONE) FEED PUMPSTATION 2

The final effluent flow to the Netheridge SRO treatment Ozone Disinfection unit would be controlled to follow the Netheridge SRO treatment flow. The Gate 2 concept design assumes this would be through maintenance of the pumpstation wet-well level at a target threshold rather than a direct copy of the pumpstation 1 flow (diurnal profile). The wet-well would thus provide an element of inter-stage balancing to cater for step changes in treatment flow.

The feed pumpstation would operate continuously whilst there is final effluent available within the wet-well. The proposed pumpstation would have three pumps, operating as duty-assist-standby. Downstream level within the treatment (at hydraulic low points) would be monitored and pumped flow reduced or stopped to prevent overtopping within the treatment stream. The primary process measurements would be pumped flow, pumped volume, wet-well level and downstream high-level thresholds.

9.7 OZONE DISINFECTION UNIT

The Ozone Disinfection unit proposed is comprised of four contact tanks where ozonated water is introduced to the effluent stream. The unit will comprise ozone generator plant, ozone injection plant, contact tanks and ozone destruction (of surplus off-gas) plant. The combined package would be controlled by a package controller that manages the production, injection, and destruction processes. The concept design assumes the Netheridge SRO treatment master controller will monitor the Ozone package through a network connection with the package controller, supplemented by hardwired IO if required.

The ozone dose rate would be based on the final effluent flow to the contact tank as measured by a common instrument associated with Feed Pumpstation 2. This would be the primary process measurement for control. Ozone monitoring within the vicinity of the treatment unit would be required, at Gate 2 this is assumed to be linked to the Ozone package controller for alarm annunciation locally and remotely, via the Netheridge SRO treatment master controller.

9.8 (BAF) FEED PUMPSTATION 3

The final effluent flow to the Netheridge SRO treatment BAF unit (and following GAC unit) would be controlled to follow the Netheridge SRO treatment flow. The Gate 2 concept design assumes this would be through maintenance of the pumpstation wet-well level at a target threshold rather than a direct copy of the pumpstation 1 flow (diurnal profile). The wet-well would thus provide an element of inter-stage balancing to cater for step changes in treatment flow.

The feed pumpstation would operate continuously whilst there is final effluent available within the wet-well. The proposed pumpstation would have three pumps operating as duty-assist-standby. Downstream level within the treatment (at hydraulic low points) would be monitored and pumped flow reduced or stopped to prevent overtopping within the treatment stream. The primary process measurements would be pumped flow, pumped volume, wet-well level and downstream high-level thresholds.

9.9 BAF

The BAF unit proposed would have four separate tanks. The flow will be equally split across all four tanks. Were one or more BAF tanks to be taken offline (by hand) then the maximum Netheridge SRO treatment flow from the feed pumpstations 1, 2 and 3 would need limiting to the remaining BAF treatment capacity, it is assumed this would be a manual action via the Netheridge SRO treatment HMI screen / SCADA. Dependent on the chosen supplier, the system control may be proprietary and included as part of the package.

The effluent flow through the BAFs is aerated to support the treatment biosphere. Each BAF would have an independent air-diffusion system connected to a common air manifold. The rate of air flow to each BAF would be at or above a commissioned minimum (to keep the BAF media dispersed); above this minimum air flow, the air flow would be controlled to maintain the BAF dissolved oxygen level at a pre-set threshold. Each BAF would have automated valve to control the flow of air.

The common air manifold would be maintained at a pre-set pressure, sufficient to drive airflow through the BAFs, by two Air Blower machines operating as duty standby. Manifold pressure measurement would provide the primary control measurement for blower speed control. Each blower is assumed to be a self-contained machine with independent controller and instrumentation for safe operation of the blower unit. The Netheridge SRO treatment master controller would modulate the blower output to maintain the air manifold pressure and modulate the BAF air control valve to maintain the BAF dissolved oxygen level and media dispersal.

Periodically, the BAF tank will require backwashing. The initiation of a backwash would be by elapsed time from last backwash or by operator request, the concept design does not expect initiation of backwashing by other process measures. The backwash system is common between the BAF unit and GAC unit, with a single BAF or GAC tank being backwashed at a time. The backwash control is detailed below as a treatment sub-system.

9.10 GAC

The GAC unit proposed would have seven separate tanks. The flow will be equally split across all seven tanks and no automated flow control is expected. Were one or more GAC tanks to be taken offline (by hand) then the maximum Netheridge SRO treatment flow from the feed pumpstations 1, 2 and 3 would need limiting to the remaining GAC treatment capacity; it is assumed this would be a manual action via the Netheridge SRO treatment HMI screen.

The effluent flow through the GAC is gravity fed and there is no aeration or dosing. Periodically, the GAC tank will require backwashing. The initiation of a backwash would be by elapsed time from last backwash, by operator request, or on high differential pressure across a GAC tank. The backwash system is common between the BAF unit and GAC unit, with a single BAF or GAC tank being backwashed at a time. The backwash control is detailed below as a treatment sub-system.

9.11 (OPTIONS 3 & 4 ONLY) (ION EXCHANGE) FEED PUMPSTATION 4

The final effluent flow to the Netheridge SRO treatment Ion Exchange unit (and following UV Disinfection unit under Option 4) would be controlled to follow the Netheridge SRO treatment flow. The Gate 2 concept design assumes this would be through maintenance of the pumpstation wet-well level at a target threshold rather than a direct copy of the pumpstation 1 flow (diurnal profile). The wet-well would thus provide an element of inter-stage balancing to cater for step changes in treatment flow.

The feed pumpstation would operate continuously whilst there is final effluent available within the wet-well. The proposed pumpstation would have three pumps operating as duty-assist-standby. Downstream level within the treatment (at hydraulic low points) would be monitored and pumped flow reduced or stopped to prevent overtopping within the treatment stream. The primary process measurements would be pumped flow, pumped volume, wet-well level and downstream high-level thresholds.

9.12 (OPTIONS 3 & 4 ONLY) ION EXCHANGE UNIT

The Ion Exchange unit proposed would have twenty resin filled vessels that operate in two blocks of ten lead vessels that receive equal effluent flow followed by ten lag vessels that receive equal flow from the lead vessels. The Gate 2 concept design assumes each vessel is run to media (resin) exhaustion before it will be manually swapped out. The concept design has allowed for automatic control of the vessels to manage which vessels are operating as lead vessels and which are operating as lag vessels. This will be through the automation of actuated valves on each vessel. The differential pressure across each vessel would be the primary control parameter used to determine when vessel's lead / lag operation and identify when the media needs replacing. A secondary measure of total volume flow through each vessel would also be recorded for operator assessment of the remaining media lifespan.

9.13 (OPTIONS 4 ONLY) UV DISINFECTION UNIT

The UV Disinfection unit proposed is a packaged unit with its own controller and instrumentation for operation of the disinfection unit. The concept design proposes the Netheridge SRO treatment master controller will monitor the UV Disinfection package through a network connection to the package controller, supplemented by hardwired IO if required.

9.14 (TREATED FE) TRANSFER PUMPSTATION

The treated final effluent flow from the Netheridge SRO treatment stream (Option 1 & 2 – GAC outfall, Option 3 – Ion Exchange outfall, Option 4 – UV Disinfection outfall) will discharge to the Treated Final Effluent Transfer pumpstation for either transfer to the selected STT discharge location or returned to the Netheridge River Severn outfall. The discharge route will depend on whether the STT transfer is dispatched or not.

The Gate 2 concept design is based upon the high-level STT operating criteria stated by STW (refer to section 1.2). At Gate 2 it is assumed that the request for transfer would be manually initiated by Netheridge site operators based upon a request from the STT operator, with due notice and recharging of the (drained) transfer pipeline. Future Gate stages should confirm the required control of the transfer flow and determine if any electronic / automated dispatch of the Netheridge SRO treatment stream / transfer is warranted / required.

The feed pumpstation would operate continuously whilst there is final effluent available within the wet-well. The proposed pumpstation would have three STS transfer pumps operating as duty-assist-standby for transfer to the discharge point and, dependent on option, an additional duty-standby pump set to return treated FE to the Netheridge WwTW outfall channel (downstream of the Netheridge SRO treatment offtake).

At Gate 2 the concept design has considered the operational approach to filling and draining the transfer pipeline between dispatch periods at high level. Automated control of drain down and charging of the transfer pipeline is currently not expected; a manual procedure with hand control of draining / filling is assumed. This should be considered further at future Gate stages to identify any control equipment (i.e., jockey pumps), or pipeline monitoring necessary to enable manual or automated control of filling/drain down.

The transfer pump operational philosophy is for the transfer flow, when dispatched, to follow the diurnal profile set for the Netheridge SRO treatment stream. As with the inter-stage pumpstations, this would be through maintenance of the pumpstation wet-well level at a target threshold rather than a direct copy of the pumpstation 1 flow (diurnal profile). The wet-well would thus provide an element of balancing to cater for step changes in treatment flow, however, the concept design has assumed no significant balancing volume to allow the transfer flow diurnal profile to differ from that of the Netheridge SRO treatment stream. The operational benefit of a larger balancing capacity should be considered at future Gate stages once the transfer constraints are matured.

The operational philosophy for the Netheridge SRO treatment stream is for the treatment flow, when not dispatched, to follow a constant flow profile. Either a dedicated duty-standby pump set (for Options 1 and 2) or the transfer duty-assist-standby pump set (for Options 3 and 4) would discharge treated FE back to the Netheridge WwTW outfall. For Options 3 and 4, actuated valves on the transfer pipeline and Netheridge outfall pipeline will be required to direct flows to the required destination. Future Gate stages should consider whether gravity discharge to the local outfall channel is possible. The assumed return pump control philosophy is for the transfer flow (when called) to follow the diurnal profile set for the Netheridge SRO treatment stream, as with the inter-stage pumpstations, this would be through maintenance of the pumpstation wet-well level at a target threshold rather than a direct copy of the pumpstation 1 flow (diurnal profile). The wet-well would thus provide an element of balancing to cater for step changes in treatment flow.

The primary process measurements would be pumped flow, pumped volume, wet-well level, and process interlock (permissive) from the discharge location as identified during future design stages.

9.15 BAF & GAC BACKWASH SUB-SYSTEM

The Gate 2 concept design proposes a combined backwash system for the BAF and GAC units. The source of wash-water would be the treated final effluent discharged by the Netheridge SRO treatment process; the concept design includes a clean wash-water tank replenished from the treated FE transfer wet-well at a constant rate. This would isolate the potential impact of backwashing on the transfer pump operation.

The concept design assumes the BAF and GAC backwash processes are common and follow the typical sequence of; isolation of the tank from effluent flow, opening the backwashing valves and passing wash-water through the tank for a fixed period or fixed volume; followed by returning the tank into service. The backwashing system would operate on a single BAF or GAC tank at a time, scheduling tanks according to their operational priority. Future Gate stages should review the dynamic model of the BAF and GAC backwashing cycles to confirm this mode of operation.

Backwash flow will be generated by a duty-assist-standby set of pumps that would operate to deliver the required backwash flow for the tank in cleaning. Operation of the pumps would be interlocked with the levels within the clean and dirty wash-water tanks to ensure availability of clean wash-water and capacity for storage of resulting dirty wash-water. The primary process measurements would be pumped flow, pumped volume, clean wash-water tank level and dirty wash-water tank level.

The GAC units will include hypochlorite dosing of the wash-water to inhibit biomass growth within the GAC tanks. The concept design proposes a separate chemical dosing package that would conform to the STW standard. Dosing control would be based upon wash-water flow to achieve a fixed concentration, a downstream residual chlorine analyser would protect against overdosing. The hypochlorite dosing unit would have a package controller that would be interfaced to the Netheridge SRO treatment master controller for monitoring.

9.16 SLUDGE TREATMENT SUB-SYSTEM

The project aim will be to operate the sludge treatment system, including thickener feed, thickened sludge and liquors pumping, in a stable manner making best use of the available storage and variable speed drives to this end.

9.17 SLUDGE THICKENING

The Gate 2 concept design proposes a single sludge treatment system for the management of waste sludges generated by the CoMagTM unit. The waste sludge would be discharged to a sludge holding tank before batch thickening in package sludge thickener units. The thickened sludge would be discharged to a further holding tank before transfer to the main work's sludge treatment process. Filtrate from the sludge thickening process would be passed to the dirty wash-water tank before being returned to the head of the main works.

The sludge treatment would be managed by the Netheridge SRO treatment master controller, with the sludge thickeners being a packaged unit with its own controller. The master controller would control the sludge thickener feed pumps, a duty-standby pair, call the sludge thickener package to run, and control the thickened sludge transfer pumps, a duty-standby pair, to pass thickened sludge to the holding tank.

The master controller would monitor the sludge storage tank level, the thickened sludge holding tank level and the dirty wash-water tank level to prevent overtopping. A high level in the sludge storage tank would stop the CoMag^{TM} waste sludge system. A high level in the thickened sludge holding tank, or the dirty wash-water tank would inhibit the sludge thickener and sludge feed pumps.

The sludge feed to the thickener would be dosed with polymer from a packaged polymer dosing plant that would conform to the STW standard. Dosing control would be based upon sludge flow and measured solids content.

The sludge storage and thickened sludge holding tanks would include pump mixers to prevent sludge settlement, these would operate on a run-dwell schedule whilst the tank levels were above a minimum setpoint.

The primary process measurements would be tank levels, pumped flow, sludge solids, pumped volume, thickener package status and polymer dosing package status.

9.18 THICKENED SLUDGE RETURN PUMPSTATION

The thickened sludge is collected within a holding tank before being returned to the main works for further treatment with other sludges. The tank would form balancing storage to disconnect the inflow of thickened sludge from its return to the main works.

The Gate 2 concept design proposes the sludge return pumpstation would operate when instructed by the main works (control system). The main works control system would provide a permissive interlock to the Netheridge SRO treatment controller to indicate return flow can be pumped. There would be a duty-standby pair of pumps operated to a constant flow. The primary process measurements would be pumped flow, pumped volume, tank level and interlock from the main works control system.

9.19 RETURN LIQUORS PUMPSTATION

The backwash and sludge thickening processes produce dirty wash-water and filtrate. These liquids are collected within a dirty water tank before being returned to the head of the main works for treatment. The tank would provide balancing storage to disconnect the inflow of liquors from their return to the main works.

The Gate 2 concept design proposes the liquors return pumpstation would operate continuously whilst there are liquors to return within the tank and the main works has capacity to take the additional flow. Future Gate stages should review this operational philosophy to determine if pumping could be scheduled during diurnal low flow periods, noting this may impact upon the proposed tank volume. The main works control system would provide a permissive interlock to the Netheridge SRO treatment controller to indicate return flow can be pumped. There would be a duty-standby pair of pumps operated to a constant flow. The primary process measurements would be pumped flow, pumped volume, tank level and interlock from the main works control system.

10 PUMPING DESIGN

10.1 INTERSTAGE PUMPING

The transfer of effluent between process units may be by gravity or pumped. Within Gate 2, assessment of the possibility of gravity flows between units was limited by the level of design development, with the result that several interstage pumping stations are assumed to be required in each process approach.

Flowrates for these interstage pumping stations follow the treatment design basis (between 200 – 550 l/s). This minimises the requirement for storage or buffering of flows at each interstage pump station. The Process schematic diagrams (Figure 5-25, Figure 6-3, Figure 7-3 and Figure 8-1) show the placement of the interstage pumping stations within the treatment train. The design of the interstage pumping stations is proposed to follow a typical wet-well, wastewater type pumping station, consisting of a below-ground wet well with submersible pumps and riser pipework, and an adjacent valve chamber.

A preliminary hydraulic design has been applied to all the necessary interstage pumping stations. Assumptions included the length of rising main, static lift, pipework diameters and the number and type of fittings. The key hydraulic parameters are indicated in Table 10-1 – Key hydraulic parameters. In more detailed design stages, consideration of the hydraulics for each individual pumping station would be made. Variable speed control of the pumps is expected to be required, to allow the instantaneous flowrate across the stages of the treatment to be similar and accommodate variations in available flow.

Table 10-1 – Key hydraulic parameters

Parameter	Value
Flow Range	200 – 550 l/s
Rising Main Length (assumed)	30 m
Rising Main Diameter	500 mm
Rising Main Velocity	1 – 2.8 metres per second (m/s)
Friction Losses	0.5 – 3.4 m
Geodetic Head (assumed)	8 m
Pump Arrangement	Duty / Assist / Standby
Pump Design Flow	275 l/s
Pump Motor Rating	37 – 55 kW

To achieve the range of flow rates, a duty/assist/standby pump arrangement is proposed. Pump selections were made for the purpose of determining the required diameter of the wet well and to inform the power demand.

10.2 BACKWASH, RETURNS AND SLUDGE PUMPING

Dependent on treatment option considered, the overall design may call for backwash pumping, returns pumping and sludge pumping.

Backwash pumping (either 'clean' water supplied for backwashing treatment units, or the resultant 'dirty' backwash water produced during this cleaning) is proposed to be based on submersible-type pumping stations either integral to or adjacent to the respective backwash tanks. These tanks are anticipated to be fed by gravity from the relevant processes. Similarly, returns pumping stations that return flow to the head of the works are also assumed to be submersible-type constructions. Preliminary hydraulic assessments relied on several estimations appropriate for the current level of design development (pipeline length, static lift, etc.). These assessments were used for preliminary pump selections to inform footprint sizes and power demands.

Details of the sludge type and characteristics are not available at this level of design development to inform the design of sludge pumping. STW's design standards suggest that either ram pumps or progressive cavity pumps would be used in most sludge pumping applications. Multiple stages of sludge pumping are provided for in the Netheridge SRO treatment: to the sludge thickeners, from the sludge thickeners to a storage tank, and from this storage tank to the main Netheridge sludge treatment system. For layout allocation and costing, assumptions on pump type, arrangement and size were made.

10.3 TRANSFER/CONVEYANCE PUMPING

10.3.1 PUMPING STATION DESIGN

The Transfer or Conveyance pumping station will be sited at Netheridge to pump treated flows to the discharge location corresponding to each option.

Some of the factors that impact the selection of a wet-well or dry-well type pumping station in this application are outlined in Table 10-2.

Table 10-2 – Pumping station comparison (dry/wet-well)

Factor	Wet-well pumping station	Dry-well pumping station
Physical Footprint	Relatively smaller	Relatively larger
CAPEX	Relatively smaller	Relatively larger
Operability & Maintenance	Pumps require lifts for inspection and maintenance. Consideration is needed for how pumps are to be stored for periods of inactivity.	Pumps are accessible directly during operation. Pumps can be drained for long-term periods of inactivity.
Health & Safety	Greater need for lifting operations, but handled at ground level	Potential for deep dry-well structure requiring ventilation to mitigate risk to operators
Pumping Efficiency	Only submersible pumps are applicable, precludes higher efficiency pumps	Potential to use higher efficiency pumps depending on quality of treated effluent

The size of the pumps (as dictated in this case by the discharge location) changes the balance of the factors given in Table 10-2, with the maintenance benefits of dry-well pumping stations outweighing the increase in station size and initial investment at a certain pump size.

Submersible-type pumps used for wastewater pumping have the capacity to pass large solid particles, whereas end-suction and split-case pumps can be more efficient but have smaller apertures which limit their application to clean fluids. In the Netheridge conveyance application, the effluent has been treated to a higher quality than typical final effluent. The particulate size and propensity of the effluent to form slime should be considered for their impact on the allowable pump types and hydraulics of the rising main respectively, particularly if the longer route options are taken forward. It is assumed that end-suction or split-case type pumps may be utilised, and so to realise the efficiency benefits of these pump types and for ease of maintenance, a dry-well pumping station is proposed for Options 1 and 2. For Options 3 and 4 relatively smaller pumps are required (similar in motor rating to the interstage pumps) and so for these options a wet-well pump station is proposed.

10.3.2 HYDRAULICS AND PUMPS

The flowrate basis for the conveyance pumping station must align with the upstream treatment stages and interstage pumping stations to avoid a flow imbalance. The flowrate design range for the conveyance pumping station is therefore 200 – 550 l/s, and this will follow the same diurnal profile as the treatment with some hysteresis.

Hydraulic design of the options for conveyance of treated effluent from Netheridge is described in the pipeline route appraisal report (Annex A2). As for the Interstage pumps, variable speed control is expected to be necessary to allow the conveyance flowrate to approximately match the treatment flowrate.

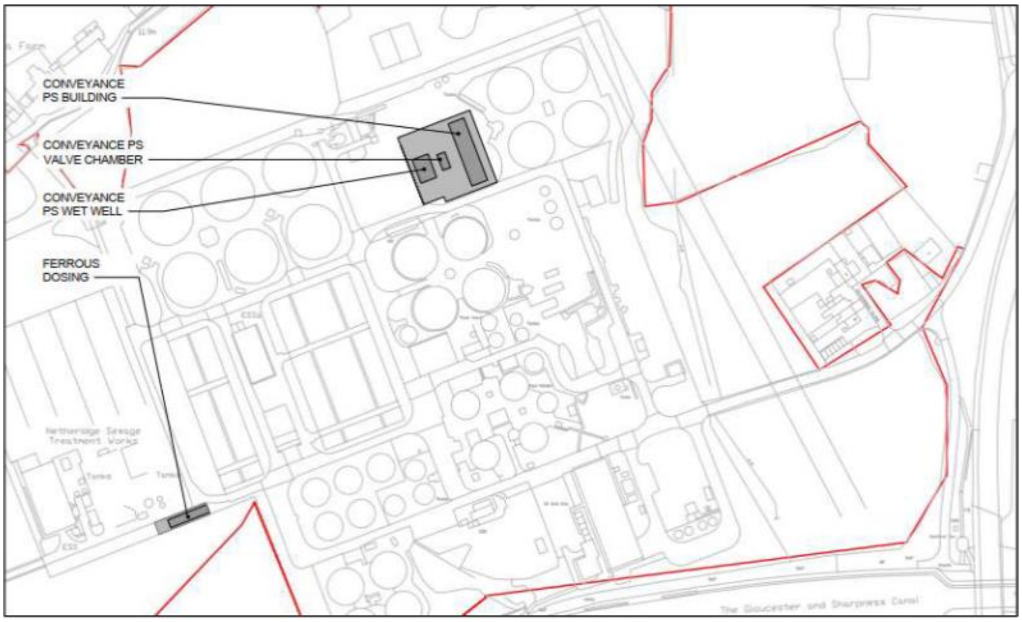
11 TREATMENT LAYOUT

11.1 GATE 1 LAYOUT OPTIONS APPRAISAL

The Gate 1 Concept Design Report outlined two different treatment process upgrades and two different concept design layouts:

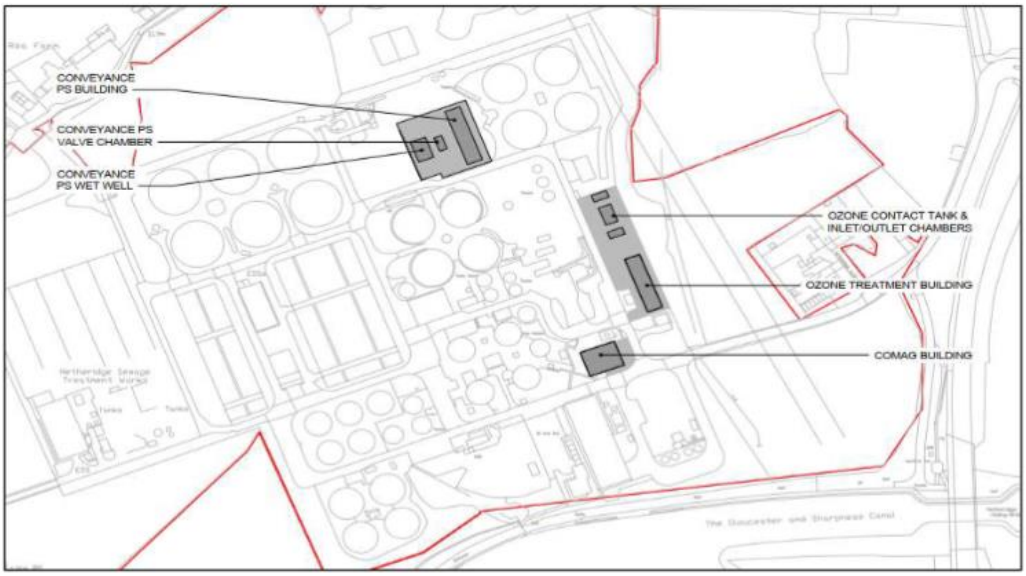
- Addition of Ferrous Dosing

Figure 11-1 - Gate 1 Ferrous Dosing Layout



- Addition of CoMag™ and Ozonation

Figure 11-2 - Gate 1 CoMag™ and Ozone Layout



As outlined in Section 4 of this report the treatment process requirements have altered significantly since Gate 1. The updated treatment process has resulted in the footprint of the scheme increasing substantially. In addition, a large portion of the land noted in the Gate 1 concept design report, and shown above, as available for the layout has been identified as unavailable during the Gate 2 review due to existing below ground infrastructure.

Therefore, the layout and footprint of the Gate 2 proposed treatment process upgrades are significantly different to those provided in the Gate 1 report.

11.2 SITE LAYOUT CONSTRAINTS

In order to determine the land available for construction of the new process footprint the existing site constraints were reviewed. There are several different constraints, outlined below, that reduce the areas that are feasible for the construction of the new treatment process.

11.2.1 LAND OWNERSHIP

STW's land ownership around the existing Netheridge WWTW extends beyond the existing fence line significantly, this is shown in red below. However, there are areas where the ownership boundary is tight to the existing as indicated by the orange arrows in Figure 11-3

. Extending the site layout into these areas would require an easement or for the land to be purchased.

Figure 11-3 - STW Land Ownership

[redacted]

11.2.2 RESIDENTIAL PROPERTY

There are areas of residential property in close proximity to the existing Netheridge site, as shown in pink on Figure 11-4, therefore consideration should be given to the proximity to these and the potential for odour generation though tertiary treatment should produce less odour than the existing processes.

Figure 11-4 - Location of Residential Property

[redacted]

11.2.3 EXISTING UTILITIES

Two areas of visibly available land used for the Gate 1 layout contain below ground infrastructure that places restrictions on their use, these are shown in Figure 11-5. The strip of land to the east contains the Storm Pumping Main and the Storm Culvert runs to the West of the Storm tanks.

A services search was undertaken for the area surrounding the existing Netheridge site There are high voltage 33 kilovolt (kV) overhead lines to the East of the site and an 11kV overhead line to the North and West. There is a gas line that comes into the site, previously supplying a now unused sludge process; the operators confirmed that this is no longer used on site however it is unknown if this pipeline has been decommissioned.

Figure 11-5 – Existing Utilities

11.2.4 GROUND CONDITIONS

Previous ground investigation information available from the site shows made ground underlain by alluvium. Additional ground investigation will be required to confirm foundation requirements however at this stage it is expected that all large structures will require piled foundations.

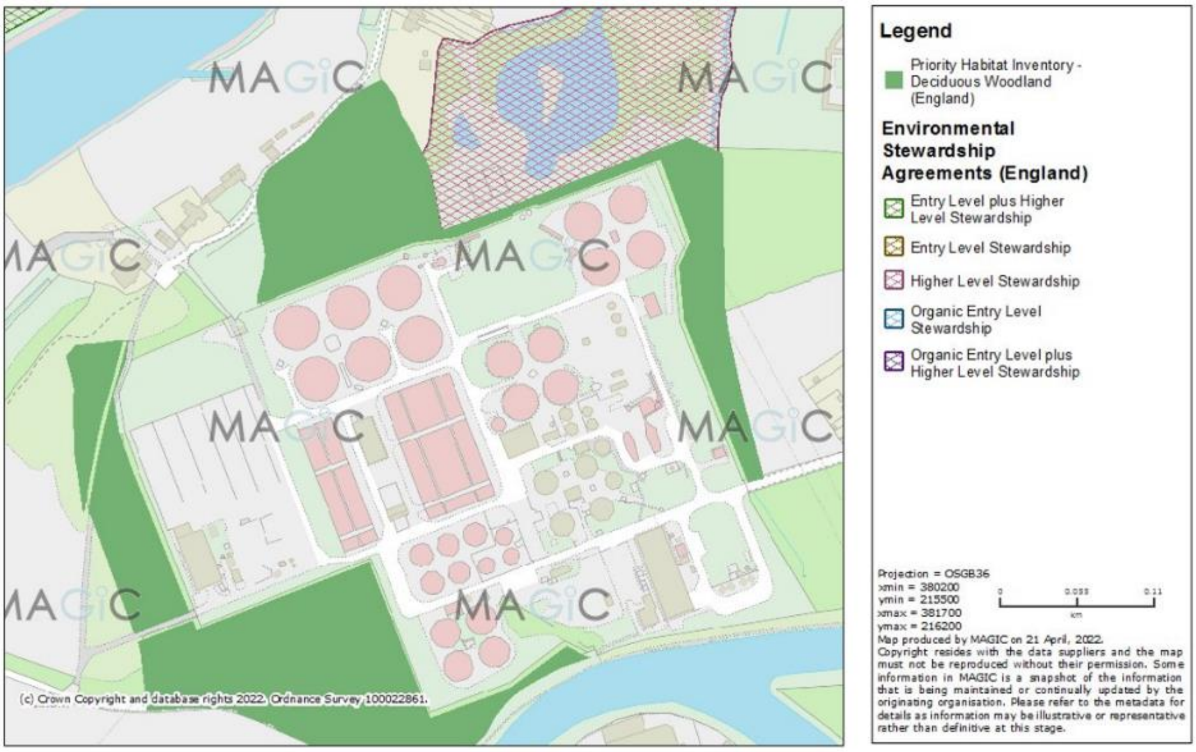
11.2.5 ENVIRONMENTAL

No formal environmental assessments have been undertaken at this stage; however, a desktop survey was completed using the DEFRA Magic Map.

As shown in Figure 11-6 STW have an Environmental Stewardship Agreement for the wetland area to the Northeast of the site. Therefore, any impact to this area should be minimised as far as possible. There are large areas of Priority Habitat Deciduous Woodland surrounding the site, any loss of this woodland may require offsetting.

During conversations with the Netheridge site operators, they noted that there are known to be badgers, great crested newts & bats residing in and around the site. No onsite habitat or species-specific surveys have been undertaken at this stage, but this clearly presents a risk to the project.

Figure 11-6 - Magic Map Output



11.3 CONCEPT DESIGN LAYOUT

The required footprint for the treatment process upgrade was based upon the sizing of the process units, the required space for pipelines and valving between units and the access required for maintenance and operation.

The layout was reviewed by all disciplines to ensure it provides a buildable solution. During the layout review consideration was given to:

- Access for operation and maintenance.
- Ease of operation.
- Proximity of units to reduce conduit and pipe lengths where possible.
- Minimising pumping therefore reducing OPEX costs.
- Minimising footprint.
- Minimising impact on valuable environmental assets.
- Minimising impact on existing operations during construction.

11.3.1 LAYOUT DEVELOPMENT

Consideration was given to several layout options, with the initial aim to keep the footprint within the existing Netheridge WWTW fence line, similar to the layouts provided in Gate 1. However, due to the below ground infrastructure outlined in Section 11.2.3 and the significant increase in the treatment footprint size, it was determined that it was not feasible to provide the process upgrade within the existing fence line.

Land available to the east of the site was considered but ruled out due to its restricted size and close proximity to several residential properties.

All layouts considered have three common elements:

- Weir chamber and lift pumping station (PS)

  The new process requires diversion of the existing WwTW final effluent. The construction of a below ground reinforced concrete weir chamber and lift pump station will divert effluent into the new process whilst allowing any excess flows to overflow and continue to the existing outlet.

  The concept layout places this structure over the existing effluent pipeline. This will enable the existing pipeline to remain in operation during construction until the new process is commissioned and ready to receive flows.

- Existing discharge Connection

  As outlined in this report it is not intended for the transfer pumps to run 365 days of the year, however the treatment process will run continuously. Therefore, an option to discharge into the existing outlet culvert is required. It is anticipated that a connection will be provided off the transfer pipeline that will connect into the existing outlet.

- Pipeline Corridor

  A new pipeline corridor is required to convey the effluent from the lift pump station to the new treatment process. This has been located alongside the Northern edge of the existing site. The width of this corridor should be kept as small as possible during construction to reduce the impact on the woodlands, Environmental Stewardship Agreement area and private residents. During detailed design a full utilities survey of the existing infrastructure on site should be undertaken to assess if it's feasible to run this pipeline corridor along the existing roadways on site. This would potentially reduce the environmental impact and land take of the scheme; however extensive existing infrastructure may result in this option being discounted.

- Dirty backwash Return

  Dirty backwash water is returned to the head of the works inlet, the route shown is indicative and the actual route and connection to the inlet will be dependent on information gathered during full utilities surveys and review of as-built information.

- Sludge Pipeline

  Thickened sludge is pumped into the existing sludge treatment area on the WwTW the route shown is indicative and the actual route and connection to the inlet will be dependent on information gathered during full utilities surveys and review of as-built information.

11.3.1.1 Layout version 1

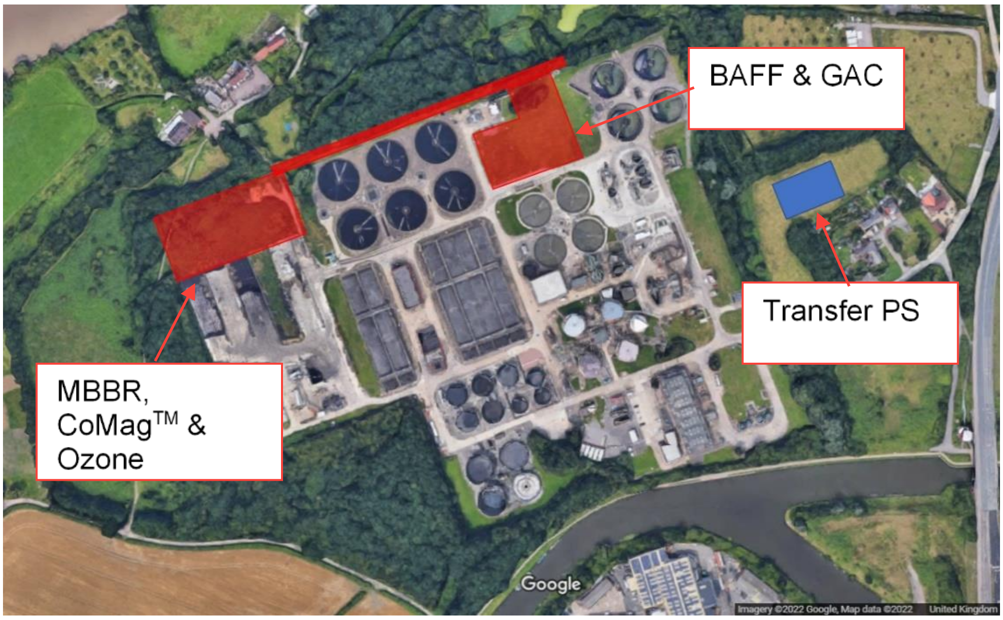
Consideration was given to splitting the process upgrade between two areas of the site. Effluent is pumped from the outlet flume to the Western area for the first stages of treatment, then pumped back to the Eastern area where the remainder of the treatment is situated. The transfer pump station is located in the field at the entrance to the site.

This layout minimises land take outside of the existing site boundary; however, it is congested, requires additional pumping and presents significant challenges with providing adequate access and space around process units for maintenance and operation. There is a potential risk that, if this layout was taken forward, deviation from best practises for spacing would be required during detailed design.

A bridging slab would be required over the existing outfall culvert to protect it & enable an access road to be built over the alignment.

This layout has the potential to impact existing operations during construction as it is situated within the existing WwTW operations area.

Figure 11-7 - Layout Version 1



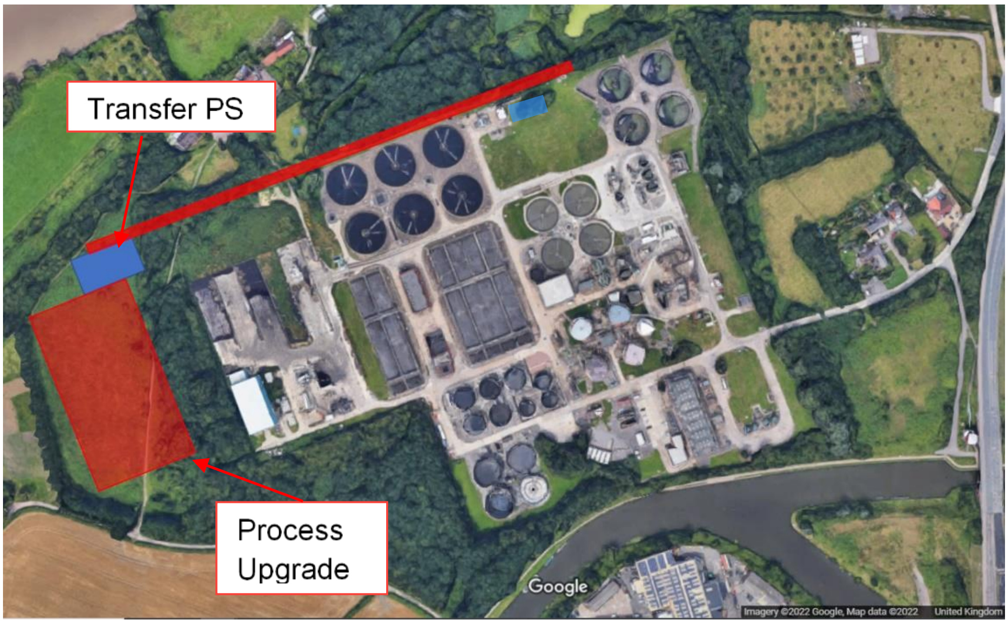
11.3.1.2 Layout version 2

In the second layout all treatment processes were located in the field area to the West of the site. This option utilises STW's large area of owned land outside of the existing fence line.

Locating the treatment process units in one area makes maintenance and operation simpler, ensures that interference with existing operations during construction will be minimised and provides flexibility during detailed design if it is determined that more, or less, land is required.

However, this location increases the distance between the effluent extraction point and the treatment process and requires construction on an area of woodland/greenfield area.

Figure 11-8 - Layout Version 2



11.3.1.3 Layout version 3

Similarly, to Layout 2, this layout places all treatment processes in the field area to the West of the site. This option differs by utilising a brownfield area of the site to the north of the sludge cake storage area and therefore a smaller area of woodland/greenfield.

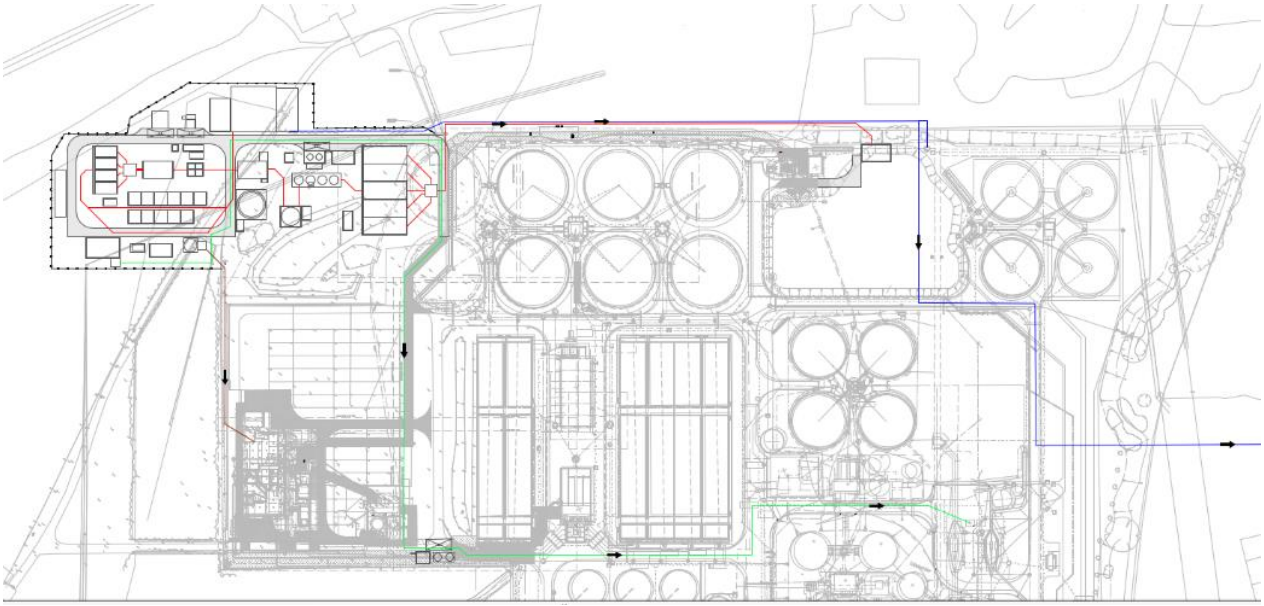
Locating the treatment process units in one area makes maintenance and operation simpler, ensures that interference with existing operations during construction will be minimised and provides flexibility during detailed design if it is determined that more, or less, land is required.

Figure 11-9 - Layout Version 3

11.3.1.4 Final layout

The proposed layout, Version 3 as shown below in Figure 11-10, was chosen because it provides adequate space for safe operation and maintenance of the plant and utilises brownfield areas of the site, minimising use of woodland and greenfield areas. It also minimises interaction and disruption of the current operations on site.

Figure 11-10 - Final Layout



The new process requires several process units each with unique interdependencies with other units and requirements for access to operate and maintain. The units have been arranged to optimise the process by reducing pipeline lengths and pumping whilst maintaining adequate person or vehicular access for operation and maintenance.

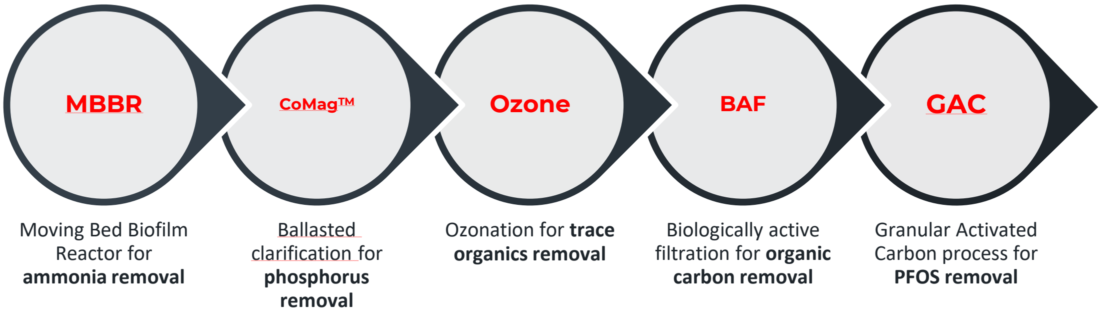
11.4 SITE LAYOUT ASSUMPTIONS

- The layout provided at this stage assumes that all STW owned land is available to the scheme.
- The layout provided at this stage assumes that there are no environmental constraints, other than those noted in Section 11.2.5 that will affect the land available for the scheme.
- The layout assumes that excavated topsoil will be relocated/redistributed onsite and that the mound behind the sludge cake storage is removed offsite.
- The layout assumes there are no geotechnical or contaminated land restrictions.
- All existing utilities that impact the scheme will be diverted/relocated. High voltage overhead lines on the East/West of the site to be converted to underground cables as required.
- The public pathway to the West of the existing site that crosses STW owned land will be re-directed around the new treatment plant area
- An easement can be obtained for the pipeline to the new treatment plant units to pass through the area of land not owned by STW to the North.

12 OPTIONS SUMMARY

12.1 OPTION 1 AND OPTION 2 – DISCHARGE INTO THE RIVER SEVERN AT DEERHURST OR HAW BRIDGE

Figure 12-1 - Option 1 and 2 treatment summary

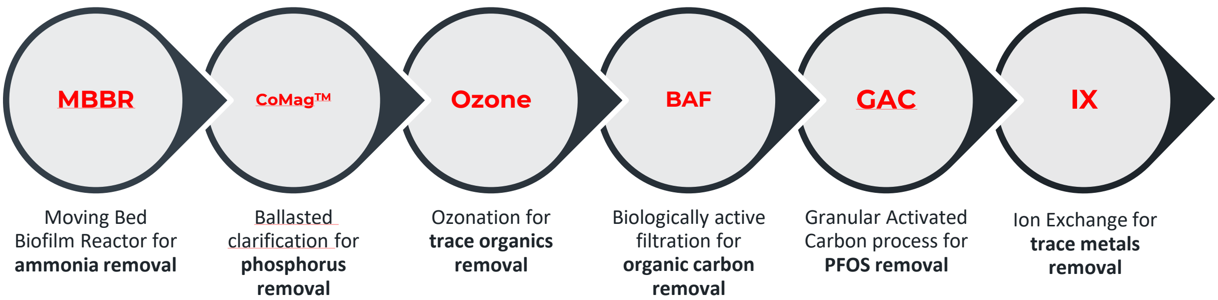


This proposed treatment train is based on the following criteria:

- Provision of treatment equipment that can handle flows between 200 l/s and 550 l/s to match the diurnal dry weather flow pattern at Netheridge.
- An assumed requirement to achieve an ammonia consent of 1 mg/l.
- An assumed requirement to meet a 0.2 mg/l total phosphorus permit. Primary chemical phosphorus removal will be provided by ferrous sulphate dosing into the ASP.
- An assumed requirement to remove the listed pesticides and herbicides to non-detectable concentrations to prevent the introduction of new substances at the point of discharge.
- An assumed requirement to remove PFOS to non-detectable concentrations to prevent impediment towards achieving target WFD status.

12.2 OPTION 3 – DISCHARGE INTO THE EAST CHANNEL

Figure 12-2 - Option 3 Treatment summary



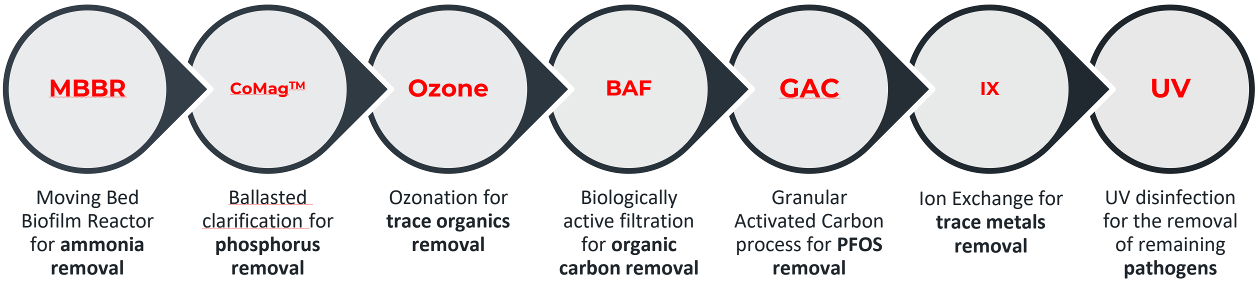
This proposed treatment train for option 3 is based on the following criteria:

- Provision of treatment equipment that can handle flows between 200 l/s and 550 l/s to match the diurnal dry weather flow pattern at Netheridge.

- An assumed requirement to achieve an ammonia consent of 1 mg/l.
- An assumed requirement to meet a 0.2 mg/l total phosphorus permit. Primary chemical phosphorus removal will be provided by ferrous sulphate dosing into the ASP.
- An assumed requirement to remove the listed pesticides and herbicides to non-detectable concentrations to meet likely permitting conditions.
- An assumed requirement to remove PFOS to non-detectable concentrations to meet likely permitting conditions.
- An assumed requirement to remove metals to non-detectable concentrations to meet likely permitting requirements.

12.3 OPTION 4 – DISCHARGE INTO THE GLOUCESTER AND SHARPNESS CANAL

Figure 12-3 - Option 4 treatment summary



This proposed treatment train for option 4 is based on the following criteria:

- Provision of treatment equipment that can handle flows between 200 l/s and 550 l/s to match the diurnal dry weather flow pattern at Netheridge.
- An assumed requirement to achieve an ammonia consent of 1 mg/l.
- An assumed requirement to meet a 0.2 mg/l total phosphorus permit. Primary chemical phosphorus removal will be provided by ferrous sulphate dosing into the ASP.
- An assumed requirement to remove the listed pesticides and herbicides to non-detectable concentrations to meet likely permitting conditions.
- An assumed requirement to remove PFOS to non-detectable concentrations to meet likely permitting conditions.
- An assumed requirement to remove metals to non-detectable concentrations to meet likely permitting requirements.
- An assumed requirement to provide disinfection to permit discharge into the drinking water protected area.

13 OPPORTUNITIES

The options listed in Section 12 are designed to be robust in achieving assumed permit requirements for the purpose of concept design. The removal performances of the proposed treatment process must be confirmed by pilot plant trials. Once the discharge location has been confirmed, along with permit requirements and removal performances, the following opportunities can be investigated at Gate 3.

13.1 PERMITTING

The proposed processes are operationally intensive and chemically and electrically demanding. Taking options 1 and 2 as an example, the overall environmental impact of the scheme may offset the benefit of removing trace pollutants by less than 1 µg/l. Permit requirements should be discussed with regulatory bodies to achieve the most environmentally beneficial solution.

13.2 SOURCE CONTROL / IMPORT REDUCTION

Netheridge WwTW receives trade imports for sludge treatment, and some trade waste from the catchment the WwTW serves. This project would benefit from an investigation (such as a flow and load survey) into trade waste imports which may highlight an opportunity for source control or import redirection to reduce the scope of the treatment options proposed.

13.3 ALTERNATIVE PRIMARY PHOSPHORUS REMOVAL CHEMICAL AND DOSING LOCATION

Dosing ferric sulphate into the crude sewage can reduce BOD load onto the ASPs, increasing the capacity for ammonia removal to achieve the assumed 1 mg/l consent. Ferric salts are more expensive however than ferrous versions and so a cost/benefit analysis should be undertaken.

13.4 BIOLOGICAL PHOSPHORUS REMOVAL

Use of biological phosphorus removal processes to reduce chemical consumption and offer resource recovery. Some biological phosphorus removal processes can remove ammonia, eliminating the need for MBBR processes.

13.5 OPTIMISATION OF THE EXISTING ASP PROCESS

The average ammonia concentration in the existing final effluent is 1.24 mg/l, maximum 5.9 mg/l. The ASP is designed to achieve a 15 mg/l ammonia consent, so actual performance exceeds design expectations, and the construction of the proposed tertiary MBBR process could be deferred until the requirement materialises (driven by population growth). Optimisation of the existing process could provide robustness in the interim. The potential installation of Thermal Hydrolysis process (THP) on site, together with any related changes to the sludge inventory, will impact the liquor load on the ASP and this will need to be considered.

13.6 WETLANDS TECHNOLOGY FOR PHOSPHORUS REMOVAL

Wetlands are a low carbon technology that can provide phosphorus removal. Tertiary solids removal processes will be required for low phosphorus permits however, if after discussion with regulatory bodies the assumed phosphorus permit is relaxed, Wetlands may present themselves as an attractive alternative, or overall, more environmentally suitable option. The associated land take will be significant but there are environmental benefits and water companies are currently being asked by Ofwat to include nature-based solutions in their business plans for AMP8.

13.7 FILTRATION TECHNOLOGIES FOR TERTIARY PHOSPHORUS REMOVAL

Filtration tertiary solids removal processes for low phosphorus permits may be more suitable for Netheridge because they can be turned off when not required and may be better suited to the large variation in flow. Compared to CoMag™ there is no requirement for polymer, magnetite or potable water during normal operation. The impact and control of backwash returns to the head of the works must be reviewed to confirm suitability.

13.8 REMOVAL PERFORMANCE CONFIRMED BY PILOT PLANT TRIALS

It is recommended pilot plant trials are used to assess the removal performance of advanced treatment processes with regards to micropollutants. The results of the pilot plants may remove the requirement for polishing stages. For example, coagulation and flocculation and GAC are effective metals removal process and may prove to remove the requirement for the ion exchange polishing stage. Ion exchange has been included as a polishing stage for metals removal to ensure a robust process has been provided.

13.9 REDUCTION IN SLUDGE VOLUME

The volume of sludge produced is based on the CoMag™ process receiving a high concentration of suspended solids from the MBBR process as a worst-case scenario. If the MBBR performs better than expected with regards to solids carry over or is deemed surplus to requirement because the existing ASPs can be upgraded or there is no requirement for a low ammonia permit, this will significantly reduce the sludge production - potentially by up to 10 times. This reduction in sludge volume could lead to the utilisation of the existing sludge handling facilities rather than constructing new.

14 NEXT STEPS

The ‘next steps’ proposed for Gate 3 are summarised in this section

14.1 PERMIT REQUIREMENTS

The permit requirements, particularly with respect to ammonia, phosphorous and substances deemed to be ‘new additions to the watercourse’, should be discussed and confirmed with the EA.

14.2 DATA CAPTURE

Netheridge WwTW and the discharge locations have been monitored throughout Gate 1 and Gate 2, and this data will continue to be used to improve a model of the stretch of the River Severn at Deerhurst/ Haw bridge to clarify treatment targets for Gate 3 scheme design. It is also recommended that data on sewage throughout the treatment works is collected to better identify opportunities for existing asset optimisation and potentially reduce SRO treatment scope.

14.3 NETHERIDGE WWTW UPGRADES

Seek further confirmation within STW of proposed upgrades to the existing works to comply with potential DWF, phosphorus removal or THP projects. This may lead to opportunities to deliver holistic solutions and reduce the proposed scope of the SRO treatment process, or in the case of a DWF project, highlight an issue with the hydraulic capacity of the existing works.

14.4 17 DAY START UP PERIOD

In theory, by increasing the flow by 1.75 times from 20 MLD to 35 MLD through the SRO treatment process, loads should increase by 1.75 times. This could cause process upset if flow through the system is increased quickly, overloading processes that have become accustomed to 20 MLD when there is no demand from STT. Flow should therefore be increased incrementally, shortening the period for performance testing at full flow.

Equipment failure, chemical availability and process instability all pose risks to achieving water quality requirements and performance validation during this period.

It is recommended further work at gate 3 is undertaken to develop a robust and rigorous strategy to increasing flow and validating performance prior to transfer to the new discharge location.

14.5 FLOW

During successive design stages, the approach to turndown of the treatment processes should be further refined to optimise plant stability and buffering storage volumes required.

The requirement, or not, for pumping between the SRO treatment plant discharge and existing Netheridge final effluent outfall should be confirmed. Flow under gravity may be possible if plant elevation allows.

14.6 FLOW AVAILABILITY

Review the availability of final effluent flow to confirm the viability of the Netheridge STS SRO to provide 35 MLD, and the requirement for storage capacity based on the validity of MCERTS data, which must be verified by STW.

14.7 PRIMARY PHOSPHORUS REMOVAL

Model the existing ASP and FST processes to confirm in more detail the impact of ferrous sulphate dosing into the RAS stream.

Review existing SAS thickening capacity and confirm available headroom (if any).

Produce an alkalinity consumption model encompassing future growth.

Quantify the existing potable water supply and available capacity.

An assessment of the existing primary sludge handling capacity.

14.8 AMMONIA REMOVAL

Confirm with regulatory bodies the requirement for ammonia removal at the proposed discharge locations. It should be noted that ammonia removal will reduce the ozone requirement downstream.

Confirm the existing ASP capacity and when it will be met in relation to expected growth.

14.9 TERTIARY PHOSPHORUS REMOVAL

Confirm the total phosphorus permit requirements for each discharge location.

Review the impact of backwash returns from filtration processes on hydraulics at the head of the works.

14.10 ADVANCED TREATMENT PROCESSES

Confirm the expected advanced treatment removal performance with pilot trials to confirm suitability and design parameters.

14.11 RETURNS

Review the hydraulic capacity at the head of the works and confirm the capacity to receive returns from the tertiary treatment process.

14.12 GEOTECHNICAL INVESTIGATION

Undertake geotechnical ground investigations in the proposed treatment location to confirm ground and groundwater conditions.

14.13 TOPOGRAPHIC SURVEY OF PROPOSED AREA

Undertake detailed topographic survey of proposed construction & tie-in locations. This will facilitate cut/fill calculations to be undertaken as well as provide reliable elevations for further design work. This in turn will inform the system hydraulics and opportunities to reduce pumping requirements.

14.14 UTILITIES

Undertake a full and detailed utilities survey of the proposed areas of construction, roads that could be used for pipeline corridors and any tie-in locations.

Statutory providers should be engaged early to commence discussions around the proposed utilities diversions and decommissioning of the gas main.

14.15 POTABLE WATER SUPPLY

An assessment of the current potable supply to site needs to be undertaken to determine if the existing supply can be improved or if a new supply to site needs to be provided.

14.16 WATER SAFETY PLANNING RISKS

Option 4 discharges treated effluent directly into the Gloucester and Sharpness Canal, a drinking water protected area. This option to supply Purton WTW (Bristol Water) is identified as an opportunity and if selected at Gate 3, the impact of the discharged effluent, which should be confirmed by pilot plant studies, on existing water safety planning risks should be further assessed using the All Company Working Group template.

The discharge locations of options 1,2,3 and 5 do not impact the feed to any water treatment works.

14.17 ENVIRONMENTAL SURVEYS

An appropriate environmental assessment should be undertaken, this will most likely include a Phase 1 habitat assessment as well as reptile surveys, badger surveys and bat surveys to confirm presence of these species already noted to be on site.

The EA flood risk maps show that the proposed location for the new treatment is in an area at very low risk of flooding from rivers or surface water. It should be considered whether further flood risk assessment is required.

14.18 CONTROL SYSTEM

At this point, a single MCC with single PLC has been assumed for the whole treatment plant. With further design development, multiple MCCs local to individual process areas may be preferred. With further design development, a single PLC with subordinate remote IO (at multiple MCCs) or single master PLC with subordinate process PLCs, connected via a resilient control network, may be preferred. With subsequent design development, a dual duty/hot-standby master PLC may be preferred to increase resilience and availability. Further definition of the operational philosophy will help confirm the HMI requirements, an extension to the existing site SCADA or a standalone Netheridge SRO treatment SCADA may be preferred.

14.19 DANGEROUS SUBSTANCES AND EXPLOSIVE ATMOSPHERES REGULATIONS

A dangerous substances and explosive atmospheres regulations (DSEAR) report has not been undertaken at this point. Based on the industry standard approach to explosion risk, there should be no risk from hydrocarbons (which would be dissipated in the upstream treatment processes) or methane (absence of significant methanogenic seed, required temperature or residence time and generally aerobic conditions). Risks associated with the storage and use of liquid oxygen and ozone are noted. A full DSEAR assessment should be carried out once the level of design detail has sufficiently increased.

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