Water Supply Project
Eastern and Midlands Region

Appendix D

Review of Treatment Technology

Water Supply Project, Eastern and Midlands Region

Irish Water

Final Options Appraisal Report

Appendix D Review of Treatment Technology

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1. Introduction

The Water Treatment Plant (WTP) will need to accommodate, in conjunction with various process treatment technologies, administrative buildings, stores, workshops and storage compounds; effectively forming a Water Treatment Campus capable of meeting the long term water demand for the Midlands and Eastern Region of 330 Mld in 2050. Once fully developed, the plant will be the second biggest WTP in Ireland and it will serve as a key element of Ireland's strategic infrastructure, necessary for the orderly and sustainable development of the country's economy.

A specimen design has been prepared to illustrate:

- How the main elements of the plant will be configured; and
- The way in which treatment plant residuals will be managed.

The specimen design is based on best water industry practice, and process units have been sized on the basis of well-established technology for both treatment of water and for the management of WTP residuals. The design is conservative in its nature, given that it is at a very early status in its development, but it does inform the extent of land take that will most likely be required to deliver construction of a WTP which is suitably sized.

It is envisaged that the project will be procured by way of a 'Design Build' (DB) or a 'Design Build & Operate' (DBO) contract. The procurement process will therefore take a performance based approach which will specify the output or performance requirements for the plant in terms of drinking water quality, treatment of process wastes and residues, energy requirements, site boundary constraints and so on. These performance outputs will have to be delivered within the constraints of the planning envelope established through the planning process and will be subject to detailed sustainability and capital and operating expenditure analysis.

The appointed contractor will be required to take full process responsibility for the design of the works and will be required to guarantee the provision of the necessary quantity of water to the requisite standards at points of delivery into supply, that is to say at designated draw off points along the benefitting corridor as well as at the outlet from the Termination Point Reservoir (TPR) at Peamount, from where the water will be delivered into the Dublin Water Supply Area (DWSA).

This approach will allow Irish Water to go to the market and to establish the optimum design in terms by minimising whole life cycle costs and providing the best value for money for its customers. Tendering consortia will propose designs which will be based on process variations, different methodologies and different approaches to financial modelling, and which, in each case, will provide Irish Water with the most economical offering consistent with meeting and guaranteeing the specified performance requirements.

Specimen designs, outlining the various treatment technologies, have been prepared for both **Option C (Parteen Basin Reservoir Direct)** and **Option H (Desalination)** as they use widely different technologies to achieve to deliver the required

The specimen design for **Option C (Parteen Basin Reservoir Direct)** is presented in Section 2.

The specimen design for **Option H (Desalination)** is presented in Section 3.

The presented specimen designs for the plant are modular in conception. In using such an approach, phasing of development can be arranged so as to develop the Water Treatment Plant in an orderly and structured way, keeping abreast or slightly ahead of water demand requirements between 2024 and 2050.

1.1 Specimen Design Considerations

The Water Supply Project Eastern and Midlands Region have a calculated net demand of 320 Mld, which is the daily volume of treated water to be put into supply. Allowing for process water usage in the WTP of 10Mld, the gross design throughput of the plant will be 330Mld.

The water treatment plant will be constructed in two phases. Phase 1 will initially treat 165 Mld of water (from 2024) rising to 247.5 Mld (up to 2030). In Phase 2 the plant will expand to a treatment capacity of 330Mld, to meet the projected demand for 2050; four modular water treatment streams, each capable of treating 82.5Mld to produce 80Mld of treated water for supply, will be needed to meet these output requirements.

Table 1-1shows the design flow rates in hours for each treatment stream. In general, treatment units have been sized on the Maximum Design Flow, i.e. the plant throughput over 18 hours.

Table 1-1 WTP Design Flows for each Treatment Stream

Design Flow	Calculation Basis	Calculated Flows
Minimum Design Flow (m^3/h)	Daily Design Demand (m^3/d)/ 22 hours	3,750 m^3/h
Average Design Flow (m^3/h)	Daily Design Demand (m^3/d)/ 20 hours	4,125 m^3/h
Maximum Design Flow (m^3/h)	Daily Design Demand (m^3/d)/ 18 hours	4,600 m^3/h

In preparing specimen designs, a number of general considerations and assumptions have been taken into account:

Flexibility in Design of the Plant

Flexibility in design gives plant operators options for management of treatment processes and flexibility in water quality and plant performance monitoring.

It is also important that operators have the flexibility to take advantage of reduced power supply tariffs or to vary the plant throughput in accordance with daily requirements while still maintaining the ability to respond to increased demand for short periods in response to emergency situations.

The principle of dividing the works into four discrete process streams is an important consideration. Over the lifetime of the works, new technologies or treatment methods will become available. Having discrete streams allows improvements to be piloted in one stream without affecting the normal operation of others. Similarly having separate streams will allow operatives to monitor performance of the plant and, in circumstances where there are water quality issues, it will be possible to isolate one stream of the plant without affecting the operation of the others.

Inlet Flow Balancing

Consideration has to be given to transferring raw water from the abstraction source to the 'head' of the WTP in the most energy efficient way possible. This requirement however has to be balanced with the need to operate the treatment plant continuously, as disruption of flow to the plant, or sudden ramping up or ramping down of flows, will adversely affect most (but not all) of the water treatment processes which will come forward for consideration under a 'Design and Build' or a 'Design Build and Operate' procurement competition. If a decision is taken to use off-peak electricity costs to deliver this raw water to the treatment plant, then there will be a requirement to provide an element of raw water balancing at the 'head' of the works in order to allow the flow to be distributed to the process units evenly over a twenty four hour period.

Ability to ramp up flows for short periods of time

The net throughput to supply from the Treatment works will be 320 Mld at 2050. Process wastes / water treatment plant residuals will account for an additional 10 Mld, so that the gross design capacity of the plant at 2050 will be 330 Mld. In normal circumstances this flow will be drawn evenly over a twenty four hour period (or twenty two hour period in the event that off peak pumping is employed).

There may however be circumstances (for example in the case of service outages for maintenance of the pipeline or in the event of a burst in the pipeline) in which daily water demand requirement will have to be drawn over a shorter period - say 18 hours. This would enable strategic storage facilities, such as the Termination Point Reservoir in Peamount, to be recharged in a shorter period if there is an emergency.

Raw Water Pumping Plant were sized in such a way that the pre-set daily demand can, in normal circumstances, be delivered over a twenty four hour (or twenty two hour) period but with the provision that in exceptional circumstances, the flow can be ramped up to deliver the gross twenty four hour requirement over an 18 hour period; and subsequently ramped down to restore normal delivery rates once these circumstances have passed.

2. Parteen Basin – Conventional Water Treatment Plant

2.1 Conventional WTP - Design Considerations/ Assumptions (General)

Inlet Flow Balancing

It was assumed that co-settled supernatants, filtrates/ centrates, back wash waters and filter run to waste waters were returned to the head of the works. Therefore, it would be necessary to blend these elements with the incoming raw water. These returns would be pumped to static mixers on the incoming raw water pipelines and thereafter provided with a nominal 30 minute retention time in the Raw Water Blending Tanks at the head of the works.

There are two possible scenarios which may come forward for consideration under a 'Design and Build' or as a 'Design Build and Operate' procurement competition;

- Proposals may come forward based on the use of variable speed pumping without the need for anything more than a nominal 30 minutes retention time for **blending** the inlet water with re-circulated residuals. Such proposals would be based on the premise of pumping raw water to the WTP at a constant rate over a twenty four hour period; or alternatively
- Proposals may come forward based on the use of fixed speed pumping with a larger **balancing** volume (say two hours) at the inlet of the works. Such proposals would be based on the premise of pumping at off peak times and subsequently discharging inlet flow from the balancing tanks to the process units over a twenty four hour period.

The specimen design has been prepared on the basis of a requirement for two hours balancing at the head of the works.

Ability to ramp up flows for short periods of time

It has been assumed that the WTP would operate at conservative loading rates where the flow is evenly spread over a twenty four hour operating period. Where the daily requirement must be delivered in a shorter time frame less conservative, but nonetheless safe, loading rates would be employed. This is illustrated in Table 2-1.

Table 2-1 Clarifier and Filter Loading Rates at Various Throughput Rates

Throughput	Flocculation Tank Retention Period.	Settlement Tank Loading Rates in each Stream		Rapid Gravity Filter Loading Rates in each Stream	
		All Clarifiers	One Clarifier out of service	All filters	Two Filters out of service
330 Mld over 24 Hours	40 minutes	3.04 m/h	3.55 m/h	4.24 m/h	5.30 m/h
330 Mld over 22 Hours	36.7 minutes	3.30 m/h	3.85 m/h	4.61 m/h	5.76 m/h
330 Mld over 20 Hours	33.3 minutes	3.65 m/h	4.26 m/h	5.09 m/h	6.36 m/h
330 Mld over 18 Hours	30 minutes	4.00 m/h	4.66 m/h	5.58 m/h	6.97 m/h

Distribution of Flow at the Treatment Plant

When the Water Treatment Campus is fully developed, i.e. treating 330 Mld by 2050, the incoming flow from Parteen Basin would initially be distributed across two Raw Water Balancing (or Blending) Tanks, each of which in turn would feed two treatment streams. Therefore, a total of four main treatment streams in all would be available; and flow distribution manifold building(s) would be required in conjunction with the Raw Water Balancing Tanks. These manifold buildings would house flow control valves and flow meters for each of the treatment stream outlet pipelines; flow meter bypass lines along with the necessary cross connection valving would also be required. If there are separate manifold buildings then there would need to be pipeline cross connections between them to allow flexibility in operation during the phased development of the works; which would also provide for flexibility in the day to day routine operation and maintenance of the plant.

Downstream of each of manifold building, separate in-line static mixers would be provided at the injection points for Acid and Coagulant dosing.

Raw Water Quality and Water Treatment Processes

The source water can be categorized as surface run-off water from a rural catchment and river sources, stored in an impoundment reservoir, with low to moderate turbidity, moderately high organic colour and total organic carbon derived from natural vegetation. Algae and microbial pathogens (bacteria, viruses, and cryptosporidium) may also be present in the raw water.

In preparing the specimen design, particular attention was paid to potential issues that may arise seasonally due to the presence of algae in the raw water. For this reason, the following process steps have been included in addition to the normally applied coagulation, flocculation and filtration steps:

- An inlet aeration stage (Cascade Aeration) has been included as a process step in the specimen design. Provision of such an aeration stage would reduce tastes and odours caused by dissolved gases such as Hydrogen Sulphide in the water, which are then released.

- Provision was made in each of the modular treatment streams for the installation of Powdered Activated Carbon dosing which could be employed on a seasonal basis for the control of tastes and odours.

- An enhanced treatment stage of granular activated carbon (GAC) filtration would provide improved removal of circa 5 micron sized parasite pathogens such as Giardia and Cryptosporidium. In the specimen design; GAC is employed in second stage filters to provide additional benefits in terms of absorption of soluble organic compounds not removed by earlier treatment stages and in providing a surface for biological growth (biological activated carbon), which can remove biodegradable organic components that might stimulate biological regrowth in distribution systems. It should be noted that periodic (months to years) renewal or regeneration of the GAC would be required, depending on the input water quality and target output water quality required.

In summary, the proposed treatment process stream includes pH correction, chemical coagulant and coagulant aid dosing, powdered activated carbon dosing, flocculation and clarification followed by first stage filtration and second stage (GAC) filtration. These treatment process steps have the principal role of coagulating dissolved, colloidal and particulate material (mineral and organic turbidity, algae, and microbial pathogens) enhancing removal of these materials during clarification and filtration stages.

Construction of Treatment Streams (4 No)

The design of the WTP would be process led and performance driven, based on results of water quality sampling in Parteen Basin. While the detailed design will not be known until such time as design proposals are sought from tendering contractors, the planning risk for the plant will be carried by Irish Water. This means that the planning application must be flexible enough to allow different treatment process proposals to be brought forward by DBO Contractors at the tender stage of the project. Potentially many process variations could emerge at tender stage such as:

- Conventional flocculation, coagulation, settlement and filtration
- Ballasted flocculation, settlement and high rate filtration
- Dissolved Air Flotation (DAF) and filtration
- Membrane filtration

In all, four discrete housed treatment streams have been considered, each with a gross throughput capacity of 82.5 Ml/d.

It was assumed that each of the four treatment streams would be enclosed. This approach has a number of advantages:

1. It would provide protection from the elements and, in doing so, would offer mitigation against
    a) freezing temperatures during the winter months
    b) problems with formation of algal growth during the summer months
    c) surface wind action which can disturb the operation of process units
    d) aerial pollution of the process units
2. It would promote the ethos of the Water Treatment Plant as a 'Food Factory'.
3. It would provide a secure environment for the production of water,
4. It would provide an improved working environment for the operatives charged with the running and maintenance of the Plant

The adaptation of an enclosed plant approach would also allow a planning strategy to be adopted whereby the outer shell of each of the process buildings can be defined with certainty at the outset, while allowing the procurement process to remain sufficiently flexible to accommodate innovative process designs to be put forward at the time of tender and incorporated within these building shells.

Each of the four treatment streams would comprise the following process units:

- A flocculation tank
- Settlement tanks complete with ancillary de-sludging galleries and channels
- Multi Media (First Stage) Rapid Gravity Filters complete with ancillary Filtered Water Channel and Backwash Outlet Channel
- Granular Activated Carbon (Second Stage) Filters complete with ancillary Filtered Water Channel and Backwash Outlet Channel
- An Intermediate Pumping station to lift water from the outlet of the First Stage Filters to the inlet of the Second Stage Filters

Plant Rooms would be required for the following:

- Polyelectrolyte Storage and Dosing
- Powdered Activated Carbon Storage and Dosing
- On Site Electrolytic Chlorination (OSEC) equipment including brine tanks and Sodium Hypochlorite storage tanks
- Ammonia storage and dosing equipment

- Compressed Air Handling (Filter Air Scour Blowers and Instrument Air Compressors)

It should be noted that filter backwash pumping equipment would be installed within sumps/ compartments of Clearwater Tanks.

A Wet Chemistry room would be required for each process building; and would house instrumentation for analysis of the various water treatment parameters for each of the treatment streams.

Finally, each of the buildings would need to incorporate an Electrical Sub Station along with Low Voltage Switchgear/ Control Rooms, which will be required to distribute electrical power to all elements of the plant contained within that building.

Chemical Bulk Storage and Dosing

The specimen design included for a stand-alone Chemical Bulk Storage and Dosing Building, which would provide process most, but not all, of the chemicals to be used in the water treatment process. This building has been sized to accommodate the following elements:

- Chemical Reception and spillage containment area
- Coagulant Bulk Storage and Dosing Equipment
- Acid Storage and Dosing Equipment
- Post Filtration pH correction chemicals Bulk Storage and Dosing.

Polyelectrolyte and Powdered Activated Carbon dosing would be accommodated within the plant rooms of the individual treatment streams. These chemicals are delivered in 20 kg bags which would have to be manually loaded into polyprep units or powder feed units. It is practical to keep these units close to the point of application of these chemicals, and to minimize the length of small bore chemical dosing pipelines for these particular applications with a view to eliminating blockages. Each process building would also be required to house its own On Site Electrolytic Chlorination (OSEC) plant and its own Ammonia Dosing Plant.

Disinfection

Primary disinfection, to inactivate microbial pathogens, has been addressed using ultraviolet (UV) light irradiation. One disadvantage of UV irradiation is that there is no chemical residual to control regrowth of bacteria or ingress of microbial pathogens into treated water storage tanks, major transmission pipelines and local distribution systems. Consequently, it is good practice to provide some form of secondary chemical disinfection after primary UV disinfection to enable maintenance of a measurable disinfectant residual concentration in the Clearwater Tanks and the long-residence-time treated water transmission pipeline. Maintenance of a trace disinfectant residual within the distribution system to quality assure the wholesomeness and cleanliness of drinking water to the customer can be achieved by using a range of options such as chlorination, chloramination or chlorine dioxide injection.

The specimen design included provision for hypochlorite dosing into a contact tank that provides circa 30 minutes residence time. The potential to add ammonia after the contact time to form monochloramine provides residual chemical disinfection in the Clearwater Tanks and the transmission pipeline. The chlorine dose may be much reduced as compared with conventional primary disinfection if all the preceding treatment steps (including UV) in the specimen design are implemented, reducing the potential to form Trihalomethanes (THMs) to a 30 minute, low chlorine concentration, contact period.

Final Chlorination would be carried out at designated draw off points along the benefitting corridor as well as at the outlet from the Termination Point Reservoir at Peamount.

Fluoridation and Orthophosphate Dosing

The specimen design included for Final Chlorination, Fluorine Dosing or Orthophosphate dosing. It is assumed that these chemicals will be applied at designated draw off points along the benefitting corridor as well as at the outlet from the Termination Point Reservoir at Peamount.

Treatment and Disposal of Water Treatment Plant Residuals

The specimen design was prepared on the basis that treatment plant residuals will not be discharged back to Parteen Basin, which forms part of the Lower River Shannon SAC. The implications of this assumption are that:

- Washwater from filters would be treated in a separate side stream (with duty and Standby facilities) and recirculated to the head of the works.
- Water generated during the filter run to waste cycle (ripening on start-up) would be settled prior to recirculation to the head of the works.
- Treated washwater and filter run to waste water would be combined in a static mixer prior to pumping it back to the head of the works. This flow would be irradiated with UV in order to achieve at least a 6-log protozoal removal/ inactivation credit in accordance with the Irish Water Pathogen Compliance Criteria.
- An emergency lagoon or lagoons would be provided on the site to accommodate temporary storage of untreated washwater in circumstances where routine maintenance of the washwater treatment side stream is required. The lagoon or lagoons would also accommodate tank and pipeline scours from the site. In addition the lagoons will provide a very important function during the commissioning and testing stage of the works. With the lagoons in place, it would be possible to initially run water through the plant and recirculate it to the head of the works until such time as a satisfactory standard is achieved in the treated water output and it meets all of the necessary drinking water quality objectives.
- Rainwater Swales would be provided within the water treatment plant site to allow for disposal to ground of rainwater from roads and other impermeable areas of the site.

It should be noted that while the specimen design for these elements of the works has been developed taking account of Irish Water's 'Design Specification: Treatment and Disposal of Water Treatment Plant Residuals' (Document No: IW-TEC-900-700-01), there are a number of variations to this specification which are included in the Specimen Design, given the large scale of the treatment Plant. These were as follows:

- The IW Design Specification requires in general, that Waterworks Sludge Thickeners are designed to accommodate up to three days sludge storage; the Specimen Design separates the thickening and storage process steps. The Scraper mechanisms in the thickeners would require periodic maintenance and losing the benefit of three days storage of a significant amount of sludge during such maintenance operations could prove problematic. It was felt that providing sludge storage separate from the thickeners provides greater operational flexibility on a large scale works.
- The IW Design Specification requires in general that 'Filter Run to Waste Water' is co settled with Filter Washwater. In the case of this treatment plant, where it was assumed that washwater was treated and returned to the head of the works, the increased hydraulic load to the washwater treatment side stream, and in the event that the 'Filter Run to Waste Water' were to be co-settled with Washwater, would make the design of the washwater treatment side stream uneconomical. The quality of the 'Run to Waste Water' would be of a much

higher standard that the returned filter washwater. In these circumstances and given the scale of the plant, a pragmatic approach is to blend the Treated Washwater with the Settled 'Run to Waste Water' downstream of the washwater treatment side stream and to irradiate the combined flow with UV after this mixing stage.

It was considered, for the purposes of preparing the Specimen Design, that the approach outlined above would deliver robust, reliable and repeatable performance and the best overall solution in terms of sustainability and capital and operating expenditure.

Treated Water Storage
The WTP has been configured on the basis that two hours of treated water storage would be provided in Clearwater Tanks. This would allow for pumping of treated water through the transmission pipeline to the Break Pressure Tank (BPT) using off peak electricity tariffs. A downstream consequence of this decision is that there would be a requirement to provide balancing storage at any point along the benefitting corridor where water is abstracted from the pipeline. This will enable a similar pumping strategy to be utilised at those locations.

Ancillary Buildings and Compounds
In addition to the various process units required for the treatment of water and for management of WTP residuals, a number of ancillary buildings, structures and compounds would be required on the Campus. The specimen design for the Campus included for:

- A Main Administration Building and Visitors Centre.
- A Workshop and Welfare Building.
- A Stores Building.
- A General Materials Storage Compound; and
- A Weighbridge Area.

2.2 Conventional WTP - Detailed Design Considerations

The various process units were sized on the basis of the design assumptions outlined in Section 2.1, but there were a number of other specific and detailed, considerations taken into account. These are discussed in this Section 2.2.

The initial design of each element of the WTP has been based on the Maximum Design Flow, which is defined in the Irish Water Design Specifications, as follows:

Maximum Design Flow (m^3/hr) = Daily Design Demand (m^3/d)/ 18 hours

where the Daily Design Demand for the WTP is defined as follows:

Daily Design Demand = Daily Output of Plant + Daily Process Waste Water Volume

Provision was made at this stage for a 10 Mld recirculation of water, equating to just over 3% of the ultimate treated water output of 320 Mld. The Daily Design Demand for the plant was therefore 330 Mld (330,000m^3/d).

As discussed in Section 2.1, treatment capacity provision was on the basis of four separate parallel treatment streams, each capable of outputting 80 Mld (80,000m^3/day) from a design throughput of 82.5 Mld (82,500m^3/day). The Maximum Design Flow for each treatment stream being calculated at 4,600m^3/hr as follows:

Maximum Design Flow (m^3/hr) = 82,500m^3/d/ 18 hours = 4,583.3m^3/hr (say 4,600 m^3/hr)

Parteen Basin Raw Water Quality Assessment

Raw water sampling has been taking place since May 2015 in Parteen Basin and a summary of results received to date on key parameters is set out Table 2-2.

Table 2-2 Summary Raw Water Sampling Data (Parteen Basin)

	Turbidity (NTU)	pH	Colour	Manganese Total (µg/l)	Alkalinity (mg/l)	TOC (mg/l)
Minimum	0.27	6.68	25.00	0.549	170.00	8.23
Average	1.28	8.17	42.85	3.531	195.11	10.54
Maximum	4.46	8.62	75.10	10.00	235.00	16.30
95%ile	3.12	8.42	72.66	6.796	213.75	13.58

Sampling of the Parteen Basin raw water will continue up to and during the tendering process for the plant, giving the greatest level of data possible to tendering contractors to allow them to prepare their designs.

In addition to the parameters set out above, algal blooms have occurred in a number of years on Lough Derg in late summer and the contractors tendering for the design, construction and operation of the treatment plant will need to include process steps to counteract such events. The specimen design has been prepared for a treatment process of conventional coagulation, flocculation, settlement and rapid gravity filtration. In such a process dosing the incoming raw water with powdered activated carbon (PAC) at times when algal blooms may be present would counteract the adverse effects of taste and odour in the water. There are however alternative processes that are equally well able to treat algal blooms, and remove taste and odour from the final water; and it could be expected that such processes would be proposed during the tender period of the project.

From available sampling data, the raw water in Parteen Basin does not present particular treatment problems. There are, however, a number of considerations with regard to mitigation of process risk. These can be summarised as follows:

Ongoing Sampling and Analysis of Raw Water

- Continued raw water quality sampling will offer a better picture of the full variation and impacts on the treatment process. Temperature of the water source can be an issue for treatment during cold weather periods and additional temperature data will offer an understanding of how water temperature impacts on the treatment process.

- Jar tests of the raw water at the preferred abstraction point, which simulate the coagulation/flocculation part of the treatment process, would need to be conducted between now and the procurement stage of the project, to allow tendering contractors to determine which coagulant is best suited for use in their process and to help optimize the design performance of the proposed plant.

- The option of a second stage of filtration, potentially incorporating GAC, has been allowed for. A decision not to incorporate could be made at this stage pending the results of ongoing raw water sampling and a fully characterized source water quality and risk assessment, together with laboratory tests to simulate the potential for disinfectant by-products (DBP) formation of the selected treatment and disinfection processes.

Selection of Plant Loading Rates

- A surface loading rate of 4 m/hr at maximum design flow has been used in sizing the settlement tanks. Cold water conditions of around 5°C can have an impact on coagulant performance. Floc formation tends to be weaker at low temperature and also requires higher concentrations of chemical dosing due to reduced rates of reaction and higher water viscosity. Therefore, separation of light particles or separation at low temperatures can be challenging. Lower hydraulic loading rates (HLR) would help manage and minimise poor sedimentation during cold water periods.

- Filtration rates of 5.5 m/hr with all filters in service and 7.0 m/hr for two filters out of service, at maximum design flows, have been used to size the filter units. During cold weather periods when the sedimentation processes is less efficient, higher solids loading rates and carry over to the filtration process may occur. Common UK practice and similar noted Irish EPA filtration guidelines where there is concern over *Cryptosporidium* oocysts and *Giardia* cysts in raw water supplies, recommend filtration rates of about 5–7 m/hr to minimise the risk of particulate breakthrough.

Pre-Conditioning of Water

Enhanced coagulation is considered as the industry standard for the treatment of this type of water. This element of the treatment process would aim at charge neutralisation and is defined as the specific and deliberate targeting of dissolved organic carbon (DOC) for removal, which in turn would create the conditions required for subsequent agglomeration of larger colloids, turbidity and biological contaminants. Careful pH control and chemical dosing is fundamental to this mechanism.

The management of raw water pH and alkalinity are critical factors for achieving successful coagulation and flocculation. Jar testing of the Parteen Basin water will continue up to the procurement stage of the project to confirm the most suitable coagulant and pH conditions for optimum flocculation and enhanced coagulation. Typically the optimum pH for coagulation/ flocculation is in the range of 6.5 – 7.5 for lowland surface waters such as Parteen Basin. The average raw water pH at Parteen Basin has been recorded at 8.17 and pH correction would be required to bring the raw water pH into the optimum range for coagulation.

Depending on the type of inorganic coagulant used, the pH at which charge neutralisation occurs has a significant effect on the surface charge of colloidal particles, the charge of the natural organic matter (NOM), the charge of the dissolved phase coagulant species, the surface charge of formed floc particles and the solubility of coagulant in the water.

The metal based inorganic coagulants typically used for charge neutralisation purposes are highly acidic and the addition of this type of chemical to the raw water depresses the pH, which in turn may greatly affect the charges of the colloidal solids, NOM, etc., and consequently prevent effective destabilisation. The extent to which the pH is depressed by the addition of an acidic coagulant salt is governed by the alkalinity of the raw water, which is a critical parameter to be considered when designing a coagulation system. The relatively high stable alkalinity of the raw water source should allow good enhanced coagulation. This could be achieved with the selected chemical coagulant and appropriate pH adjustment to allow for optimised coagulation and floc formation at a defined pH set point following the completion of jar tests. Raw water testing is not complete, but a dose rate of 40 to 55 mg/l of 96% sulphuric acid is assumed to be required for pH and alkalinity control when using Ferric Sulphate as a coagulant.

Coagulant Dosing and Flash Mixer

Once the raw water has been pre-conditioned to the most appropriate pH and alkalinity levels, the charge neutralisation mechanism is initiated by the addition of coagulant, and the vigorous agitation of the raw water/ coagulant mixture. This agitation is known as flash mixing and is necessary both to disperse the coagulant chemical quickly throughout the raw water and to promote collisions between now neutrally charged particles to encourage their growth into a small floc.

Effective coagulant mixing can be achieved by pumps, static mixers or mechanical mixing devices installed in a tank, provided the design allows for a suitable mixing energy and sufficient flow regime to avoid short circuiting. At this stage it has been assumed that mixing would take place in a flash mixing tank.

The charge neutralisation mechanism reaction occurs within 0.5 seconds of adding an inorganic coagulant to the raw water and it is therefore imperative that the chemical is dispersed quickly throughout the flow by a vigorous mixing process. This vigorous dispersion also causes the neutrally charged particles to collide against one another, and thereby agglomerates together, resulting in the completion of the first stage of an effective coagulation process.

In the specimen design it has been assumed that this would be achieved in on-line static mixers upstream of flocculation tanks. Provision has been made for separate static mixers for acid and coagulant dosing.

Flocculation Tanks

Once effective charge neutralisation has been achieved by the coagulation stage, the agglomerations that have been created must be grown to a size appropriate to the selected downstream process in a flocculation system.

The design of the flocculation system is dependent on the type of downstream treatment process. The treatment process assumed is one based on sedimentation followed by filtration.

Basic flocculation and tapered flocculation systems can be provided for effective growth of the agglomerations that have been created following charge neutralisation. At this stage tapered flocculation would be recommended for the water treatment plant based on best practice.

Chemical bridging is a flocculation aid process and is based on binding coagulated contaminants together by the addition of an organic coagulant with a preference by Irish Water for use of cationic polymer. Cationic polymer may be added to the flow immediately after the flocculation mechanism has been completed.

Tapered flocculation mechanisms may be employed post charge neutralisation, and are suitable to create flocs applicable to sedimentation clarification processes. Within the ‘tapered flocculation’ category, mechanisms can be further subdivided into mechanically mixed or hydraulically mixed processes. The preference would be for a mechanically mixed flocculation system due to the operational flexibility afforded by a system with variable speed control.

For low turbidity raw water in cold regions the flocculation time should be at least 30 minutes.[1] The flocculation tank retention time recommended for the water treatment plant is 30 minutes (1,800 seconds). The required volume of the flocculation tanks for mechanically mixed systems, at the maximum design flow rate of 4,600m^3/h, would be 2,300m^3 per treatment stream.

[1] *MWH Water Treatment Principles and Design*

For mechanically mixed systems, tapered flocculation can be implemented by the construction of three flocculation tank compartments in series, with a vertical turbine or a vertical paddle mixer installed in each compartment.

The footprint of flocculation tank compartments which utilise vertical turbine mixers or paddle mixers is required to be, as close to as reasonably practicable, square and the mixer impeller diameter or picket fence gate rotational diameter is required to be between 0.2 and 0.5 times the chamber width. Short circuiting of the flocculation tank is to be avoided by implementing bottom entry/ top exit or top entry/ bottom exit hydraulic arrangements.

The specimen design has proposed a configuration of tapered flocculation tanks three lanes, each with three square compartments measuring 8m x 8m internally and with a water depth of 4m. This arrangement would allow for taking a lane out of service for maintenance or repair as well as allowing for variation in the retention time on a seasonal basis and for varying plant throughputs. The overall footprint of the flocculation tanks in each treatment stream would therefore be 25m by 25m internally.

In order to prevent short circuiting of the mixing system in a tapered flocculation tank, baffle walls are required between each of the flocculation tank compartments. The baffle walls would promote an over and under flow pattern to promote plug flow and minimise stagnant zones in each compartment.

Clarification Process

Notwithstanding that technologies such as membrane filtration may be proposed by contractors in the tendering stage of a DB or DBO procurement process, the specimen design has been based on a conventional clarification and filtration process. The two types of clarification processes are sedimentation (settlement with conventional coagulation/ flocculation or ballasted flocculation) and Dissolved Air Flotation (DAF); as a general guide, the processes are applicable to the raw water quality parameter set out in Table 2-3.

Table 2-3 General Guidance for the Selection of Clarification Process[2]

Peak Raw Water Turbidity	Raw Water TOC	Seasonal Algal Load	Typical Clarification Process
< 10 NTU	< 2 mg/l	High	Dissolved Air Flotation
< 10 NTU	> 2 mg/l	High	Dissolved Air Flotation
> 10 NTU	< 2 mg/l	Low	Sedimentation
> 10 NTU	> 2 mg/l	Low	Sedimentation

Table 2-2 shows the characteristics of the raw water from Parteen Basin with respect to, amongst other things, turbidity and TOC. There are also known to be occasional algal blooms on Lough Derg. While DAF would be a suitable clarification process, sedimentation has been proposed at this stage as this will allow treatment units and buildings to be sized such that they can accommodate a DAF process, or other processes, should they emerge at the tender stage.

The raw water turbidity in Parteen Basin is less than 10 NTU. However, as it will not be possible to return any waste water from the treatment process to the Lower River Shannon SAC, the settled

[2] *Based on Table 48 of Irish Water's Design Specification for CFC*

washwater and various supernatants will be returned to the raw water balancing tank to be blended with the incoming raw water.

One of the key factors influencing the performance of a coagulation and sedimentation clarification WTP is the stabilisation of the hydraulic load over each 24 hour period. Sludge blanket clarifiers that operate sporadically tend to suffer from reduced settlement tank performance, resulting in higher solid loading rates being applied to the downstream filtration system.

The maximum sludge blanket clarifier capacity is calculated based on the maximum design flow and the total number of sludge blanket clarifiers in accordance with the following formula:

$$\text{Maximum Clarifier Capacity (m}^3\text{/h)} = \frac{\text{Maximum Design Flow (m}^3\text{/h)}}{\text{Total No. of Clarifiers}}$$

The Maximum Sludge Blanket Clarifier Capacity is calculated at 660 m^3/h based on the provision of seven clarifiers in each treatment stream, as follows:

$$\text{Maximum Clarifier Capacity (m}^3\text{/h)} = \frac{4{,}600 \text{ m}^3\text{/h}}{7} = 657 \text{ m}^3\text{/h, say } 660 \text{ m}^3\text{/h}$$

The surface area of a sludge blanket clarification tank is determined from the up-flow velocity through the clarifier or as more commonly known as the surface loading rate. With the deployment of lamella plates to aid clarification, surface loading rates of up to 4m/hr are possible.

Whilst the design has been based on lamella plate settlers it is possible, under a Design Build Contract, that Tenderer(s) may propose tube settlers as an alternative to the lamella plate settlers. Typically, tube settlers can be operated at a surface loading rate of 6m/h. However, as discussed earlier, it is deemed prudent at this stage to allow for the greater treatment plant footprint to allow flexibility when seeking design proposals from tendering contractors.

The minimum allowable surface area of each sludge blanket clarifier is a function of the maximum clarifier capacity and the surface loading rate and is calculated based on the following formula:

$$\text{Minimum Sludge Blanket Clarifier Surface Area (m}^2\text{)} = \frac{\text{Maximum Clarifier Capacity (m}^3\text{/h)}}{\text{Surface Loading Rate (m/h)}}$$

The Minimum Total Sludge Blanket Clarifier Surface Area is calculated at 1,146m^2 based on a Maximum Clarifier Capacity of 660m^3/h, a surface loading rate of 4m/h for lamella plate settlers and the provision of seven sludge blanket clarifiers. Table 2-4 sets out the surface loading rates for this specimen design at various throughputs.

Table 2-4 Sludge Loading Rate on Sludge Blanket Clarifiers at Various Flow Rates

Flow Through Sludge Blanket Clarifiers	Resultant Surface Loading Rate (7 Clarifiers)	Resultant Surface Loading Rate (6 Clarifiers)
Minimum Design Flow (m^3/h) = (Daily Design Demand (m^3/d) / 22 hours) = 3,750 m^3/hr	3.30 m/h	3.85 m/h
Average Design Flow (m^3/h) = (Daily Design Demand (m^3/d) / 20 hours) = 4,125 m^3/hr	3.65 m/h	4.26 m/h
Maximum Design Flow (m^3/h) = (Daily Design Demand (m^3/d) / 18 hours) = 4,600 m^3/hr	4.00 m/h	4.66 m/h
Daily Design Demand = 2,362.50m^3/hr	3.04 m/h	3.55 m/h

At a stabilised hydraulic load over each 24 hour period the resultant surface loading rate on the sludge blanket clarifiers would be 3.04 m/h if all clarifiers are operating and 3.55 m/h if one is out of service.

Rapid Gravity Filters

One of the key factors influencing the performance of a coagulation and clarification water treatment plant is the stabilisation of the hydraulic load over each 24 hour period, particularly if a sedimentation process is being used to achieve clarification. Sedimentation plants that operate sporadically tend to suffer from reduced settlement tank performance, resulting in higher solid loading rates being applied to the downstream filtration system.

Irish Water, in the Design Specification for Filtration (IW-TEC-900-04), therefore requires maximum, minimum and average design flows to be determined such that, while each treatment stream is to be operated over a minimum of 22 hours per day, the design of the 1st Stage Filters shall be based on the assumption of the filters being capable of treating the daily demand in an 18 hour period if necessary. This is in order to build a certain level of redundancy into the filter design to accommodate fluctuations in demand.

Table 2-5 Design Flow Requirements for 1st Stage Filtration Process[3]

Design Flow	Calculation Basis	Calculated Flows
Minimum Design Flow (m^3/h)	Daily Design Demand (m^3/d) / 22 hours	3,750 m^3/h
Average Design Flow (m^3/h)	Daily Design Demand (m^3/d) / 20 hours	4,125 m^3/h
Maximum Design Flow (m^3/h)	Daily Design Demand (m^3/d) / 18 hours	4,600 m^3/h

The calculated flows presented in Table 2-5 are based on a Daily Design Demand of 82,500m^3/d for each treatment stream, made up of:

[3] Based on Table 5 of Irish Water's Design Specification for Filtration

- Required Design Plant Output of 80,000m³/d per stream;

- Plus an allowance of 2,500m³/d for recirculated flows in each stream, associated with sludge bleeds from the clarification stage and backwashing of the filters.

The minimum quantity of rapid gravity filter cells based on the Daily Design Demand (Plant Capacity) of 82,500m³/d is eight and the specimen design has allowed for two filter cells being out of service. The maximum filter cell capacity is calculated based on the maximum design flow, the total number of filters and maximum number of out of service filters in accordance with the following formula:

$$\text{Maximum Filter Cell Capacity (m}^3\text{/h)} = \frac{\text{Maximum Design Flow (m}^3\text{/h)}}{\text{Total No. of Filters – Max No. of Out of Service Filters}}$$

The Maximum Filter Cell Capacity was calculated at 764 m³/h based on the minimum number of filters (eight) and the maximum number of out of service filters (two). The calculation is as follows:

$$\text{Maximum Filter Cell Capacity (m}^3\text{/h)} = \frac{4{,}600\ \text{m}^3\text{/h}}{10 - 2} = 575\ \text{m}^3\text{/h}$$

Using the 7.0 m/hr filter rate for eight filters each with a cell capacity of 575 m^3/hr gives a required surface area per filter of 82.14 m^2, and this can be provided with filters measuring 6.5 m by 12.65 m internally on plan.

Backwashing and cleaning of rapid gravity filters is carried out by applying an air scour rinse to the beds followed by or combined with an upward water flow to cleanse the bed at intervals of between 12 and 48 hours depending upon flow rates through the plant and raw water quality at any given time. Backwashing will be triggered by one of the following:

- Turbidity levels in the filtered water;
- Differential pressure across the filter; or
- Time

Washwater from the backwash process will be drained to washwater holding tanks. Backwashing of the filter will be followed by a rinse cycle during which filtered water will run to waste until such time as the turbidity of the filtered water is sufficiently low as to demonstrate that the filter in question is ready to be put back on line.

Treated Water Disinfection and Conditioning

With respect to the disinfection process the Irish Water Specification dictates that all supplies must use chlorination for pathogenic bacteria and virus inactivation to a minimum of 4-log inactivation. For Cryptosporidium and other protozoa, the minimum treatment requirement is a 3-log inactivation. The disinfection requirement is based on a ‘log credits’ system and is illustrated in Table 2-6.

Table 2-6 Irish Water's Log Credit Criteria for Raw Water Catchment Conditions[4]

	Goundwater	Surface water	Log credit requirement
	G1 Low risk (no microbiological contamination) – sealed bored well with source protection, water drawn from deeper than 30m	Not applicable	0
Removal and/or Inactivation	**G2** High risk (with microbiological contamination) – sealed bored well with source protection, water drawn from deeper than 30m	Not applicable	2
Removal and/or Inactivation	**G3** High risk (with microbiological contamination) – sealed bored well with source protection, water drawn between 10m to 30m (Groundwater default)	**S1** Upland catchment - no agricultural activity in immediate vicinity or upstream	3
Removal and/or Inactivation	**G4** High risk (with microbiological contamination) - spring or bored well, water drawn <10m, in upland catchment with low concentration of cattle, sheep, horses or humans in immediate vicinity or upstream	**S2** Upland catchment - low concentration of cattle, sheep, horses or humans in immediate vicinity or upstream (Surface water default)	4
13 Removal + Inactivation	**G5** High risk (with microbiological contamination) - spring or bored well, water drawn <10m, in lowland catchment with high concentration of cattle, sheep, horses or humans in immediate vicinity or upstream or waste treatment outfall upstream	**S3** Lowland catchment – high concentration of cattle, sheep, horses or humans in immediate vicinity or upstream or waste treatment outfall upstream	5

Log reduction relates to the percentage of microorganisms physically removed or inactivated by a given process.

1-log reduction = 90%
2-log reduction = 99%
3-log reduction = 99.9%
4-log reduction = 99.99%

Parteen Basin is a surface water source in a low land catchment with high concentrations of animals and humans and with a waste treatment outfall upstream at Ballina/ Killaloe. Parteen Basin, based on the above classification, would be designated as surface water 'S3' with the log credit requirement of 5 for effective disinfection treatment.

Table 2-7 presents the log credits that are designated to different types of treatment process.

[4] *Based on IW-WT-Pathogen Compliance Criteria v0 11*

Table 2-7 Protozoa Treatment Options and Credits

Treatment	Log credit
Filtration (physical removal)	
Coagulation, flocculation, clarification and rapid gravity filtration	3.0
• Additional log credit for enhanced individual filtration	1.0
Slow sand filtration	2.5
Direct filtration *	2.5
Membrane filtration *	Log credit demonstrated by challenge testing and verified by direct integrity testing
Cartridge *	2.0
Bag *	1.0
Disinfection (inactivation)	
UV	Dose dependent (max 3.0)

In order to minimise the risk of build-up in the treated water of disinfectant by products such as trihalomethanes (THM), it has been proposed to disinfect the treated water with chlorine; only at doses that will leave a very low level of residual in the treated water mains. Chlorination boosting will take place at appropriate points close to where the water is fed into supply rather than at the treatment plant itself. Provision is made, for example, for chlorination equipment at the proposed termination point reservoir at Peamount in Co. Dublin. Similar installations will be required at other take-off points from the transmission pipeline.

There will nonetheless be a requirement to protect against pathogenic bacteria, viruses and protozoa in the treated water leaving the plant and ultra violet (UV) light treatment has been allowed for; upstream of the chlorine dosing points.

Referring to Table 2-7, the proposed treatment process of coagulation, flocculation, clarification and filtration would only provide up to 3 log credits but based on the catchment characteristics, 5 log credits for removal/inactivation of Cryptosporidium and Giardia would be required; additional log credits have been provided through UV treatment.

The design of the UV Treatment system would be in compliance with Irish Water's Design Specification for Disinfection Document No: IW-TEC-900-05. For UV to be effective the final water UV Transmittance (UVT) must exceed 75%. Jar tests, being conducted to determine the optimum pH and coagulant dosing for the Parteen Basin water, should also be used to show what average UVT can be achieved with different coagulants and pH corrections.

A process of dose validation is required, by which suppliers must demonstrate that a UV reactor will apply a target dose under defined operating conditions, in order to provide the necessary confidence that an installed reactor will perform as intended. The UV reactor must demonstrate that it meets target log inactivation requirements under the following variable conditions:

- Flow rate
- UV intensity (UVI)
- UV Transmittance (UVT) of treated water
- Lamp configuration

Table 2-8 sets of typical UV equipment selection parameters.

Table 2-8 UV Equipment Selection Parameters

Equipment Selection Parameters
Establishment of the Reduction Equivalent Dose (RED) to achieve the required log inactivation of the target organisms, as determined by the Protozoa Risk Assessment based on catchment and treatment risks.
Establishment of Maximum Flow and Minimum Expected UVT of the final treated water to allow accurate selection of the UV reactor and ensure dose validation based on manufacturer's certification.
Reactor Redundancy: determine if standby reactor is required by the establishment of downstream water storage capacities. If treated water storage capacity is sufficient (> 12 hours) to allow bulb replacement, typical maintenance, etc., reactor redundancy is not required.
Installation Area: Consideration of available space for installation of UV reactor, pipework and valves and to facilitate reactor maintenance and bulb replacement during operation.
Hydraulics: Effects on hydraulic profile imposed by form losses of the UV reactor as well as associated pipework, fittings and valves.

It has been assumed in the specimen design that 24 hours on-site treated water storage would not be provided; therefore, there would not be buffering capacity to allow shutdown of a UV disinfection system for planned or emergency maintenance. Duty/ standby UV reactors have therefore been incorporated, one set for each treatment stream, and each reactor capable of treating up to 4,600m^3/hr.

In addition to housing the UV reactor, the UV system needs to be sufficiently large to accommodate all process pipework, flow control valves and actuators, if required, and all control instrumentation as well as ballast panels. The area of installation would also allow the UV reactor to be installed at the bottom of a 'U' pipework arrangement in order to ensure that the reactor remains flooded at all times. Sufficient space must be available around all items of plant in the UV room to allow access for planned and emergency maintenance of reactors. This includes allowance of enough space to facilitate the removal of lamps from reactors without the requirement to disconnect the reactor from the process pipework. Reactors with dry weights in excess of 100kg would require facilities for use of lifting equipment to allow their removal from the installed UV reactor location.

Post Treatment pH Correction

Post UV treatment the water could be dosed with a chemical such as sodium carbonate (Soda Ash) for final pH correction.

Washwater and Sludge Treatment

It will not be permitted to discharge wastewater from the treatment process back to the environmentally sensitive Lower River Shannon SAC. Therefore, the process waste water will have to be recirculated through the plant. Waste water will be generated from the following sources:

- Washwater from rapid gravity filters following settlement and decanting
- Filter 'Run to Waste' water
- Supernatant returned from sludge thickening
- Expressate decanted from sludge dewatering process

The volume of recirculated water is variable and depends on such factors as:

- Rate of sludge generation in the sludge blanket clarifiers
- Filter backwash frequency
- Length of Filter Run to Waste Cycle

The sludge draw-off from the sludge blanket clarifiers would drain to sludge holding tanks before being pumped to picket fence thickeners.

Backwash water from the filter cleaning process would drain to used washwater equalisation and settling tanks where settled washwater would be decanted off and pumped to the raw water balancing tanks at the head of the treatment works. This water would need to be treated to an appropriate standard before being reintroduced into the raw water stream.

Filter rinse water would drain to 'Filter Run to Waste Water' balancing tanks. Following settlement, this water would be pumped back to a static mixer located on the rising main from the washwater treatment side stream. The combined treated washwater and recirculated filter run to waste water would be irradiated with UV prior to being returned to the head of the works.

Settled sludge would be pumped from the base of the washwater holding tank to the picket fence thickeners.

Sludge from the picket fence thickeners, at typically 3% solids, would be delivered via sludge storage tanks to a sludge dewatering plant, which would include plate presses to bring the dry solids content of the sludge cake to 25%, followed by a thermal sludge drying process to reduce, to the greatest extent possible, the volume of sludge to be disposed of off-site. Supernatant from the picket fence thickener and expressate from the sludge dewatering process would be pumped, via the washwater treatment side stream, to the raw water balancing tank at the head of the treatment process.

The average total suspended solids (TSS) concentration of washwater is typically of the order of 100 to 1,000mg/l. Because of the low concentrations of solids and their settleability (typically 0.06 – 0.15 m/h) washwater has historically been:

1. Returned direct to the headworks of the treatment plant when comprising less than 10% of the plant flow;
2. Discharged to a flow equalisation basin and then returned to the headworks of the treatment plant;
3. Discharged to washwater recovery ponds, basins, or lagoon where it is allowed to settle for 24 hours or more before being decanted and returned to the headworks of the treatment plant;
4. Discharged to surface waters following appropriate treatment; and
5. Discharged to wastewater collection system.

The current trend for handling washwater is to have a separate treatment facility, especially in larger plants, because of concerns over the presence and recycling of microorganisms. Treatment facilities that can be used to process recycled flows include:

- Flow equalisation without or with chemical addition
- Batch sedimentation without or with chemical addition
- High rate sedimentation without or with chemical addition and pre-flocculation
- Granular filtration
- Dissolved air flotation
- Membrane filtration

- Disinfection
- UV oxidation

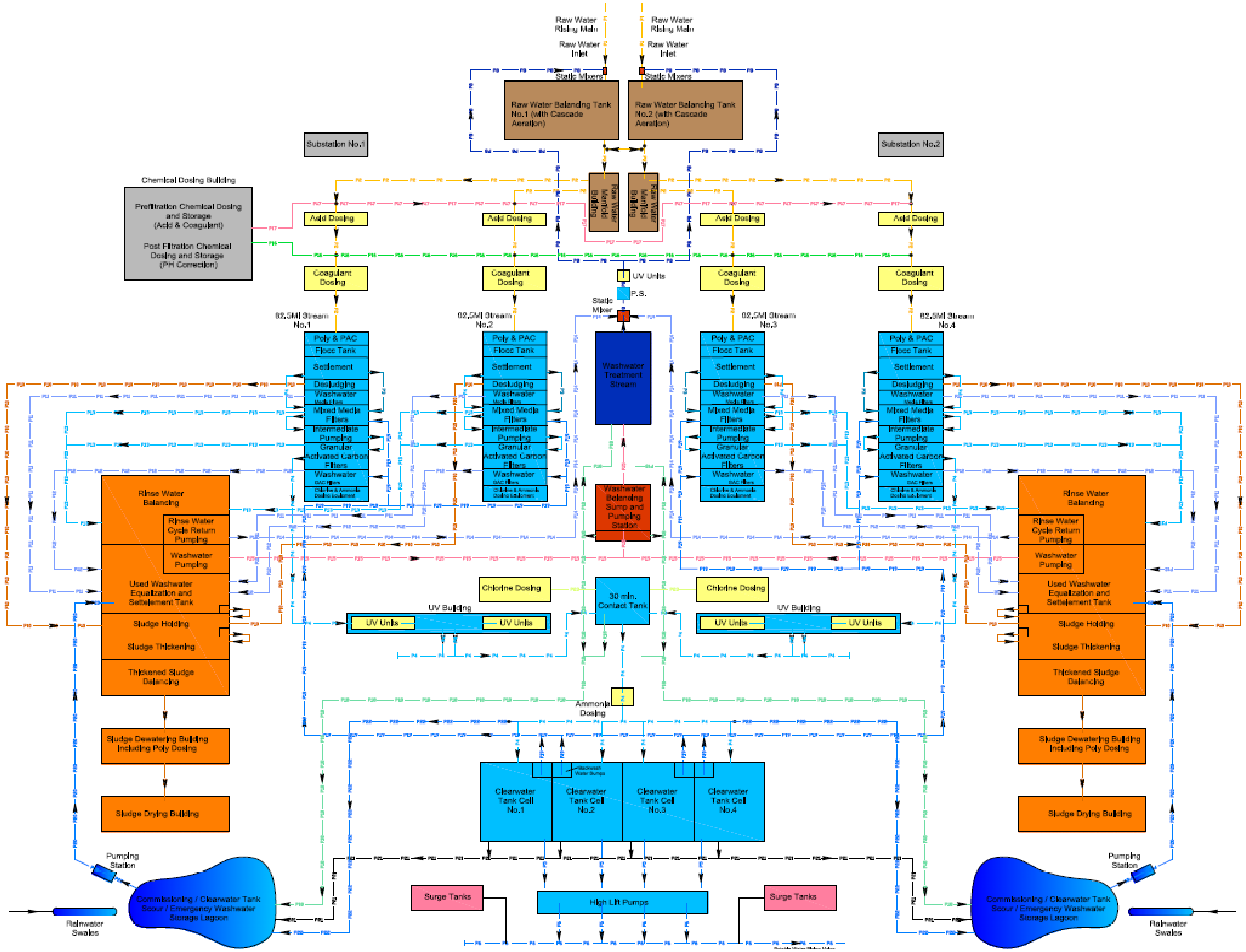
As with other elements of the plant, different processes for the treatment of recirculated waste waters would be proposed during the DBO procurement stage.

2.3 Conventional WTP – Specimen Design (Schematic)

A typical specimen design for a conventional WTP is shown in Figure 2-1.

JACOBS TOBIN

Figure 2-1 Conventional WTP - Specimen Process Design



PIPE DESIGNATION	PIPE DIAMETER	PIPE DESCRIPTION
P1	000Ø	RAW WATER RISING MAINS
P2	000Ø	RAW WATER GRAVITY MAIN
P3	000Ø	SETTLED WATER CHANELL
P4	000Ø	FILTERED WATER
P5	000Ø	CLEARWATER
P6	000Ø	POTABLE WATER RISING MAINS
P7	000Ø	.
P8	000Ø	TREATED WASHWATER RETURN
P9	000Ø	WASHWATER SLUDGE
P10	000Ø	SETTLEMENT TANK SLUDGE
P11	000Ø	WASHWATER FROM MIXED MEDIA FILTERS
P12	000Ø	WASHWATER FROM GAC FILTERS
P13	000Ø	FILTERS RINSE CYCLE WATER
P14	000Ø	RINSE WATER RETURN TO WASHWATER TREATMENT PLANT
P15	000Ø	WASHWATER TO BALANCING SUMP
P16	000Ø	COAGULANT DOSING
P17	000Ø	ACID DOSING
P18	000Ø	PARTIAL / EMERGENCY WASHWATER / SUPERNATANT DISPSAL
P19	000Ø	FILTERS BACKWASH
P20	000Ø	STORAGE LAGOON RETURN
P21	000Ø	CLEARWATER TANK SCOUR
P22	000Ø	COMMISSIONING SCOUR
P23	000Ø	CHLORINE DOSING

3. Desalination

A specimen design has been prepared for **Option H (Desalination).**

It has been prepared with reference to the *Water Supply Project Dublin Region 'The Plan' (October 2010)*. The latter contains a description of alternative desalination technologies and options with regards pre and post treatment for desalination. The most likely desalination process would be Sea Water Reverse osmosis (SWRO); as opposed to thermal or other technologies.

As discussed in Section 1.1 the demand/supply balance will be modular in concept; which particularly lends itself to the design characteristics of SWRO.

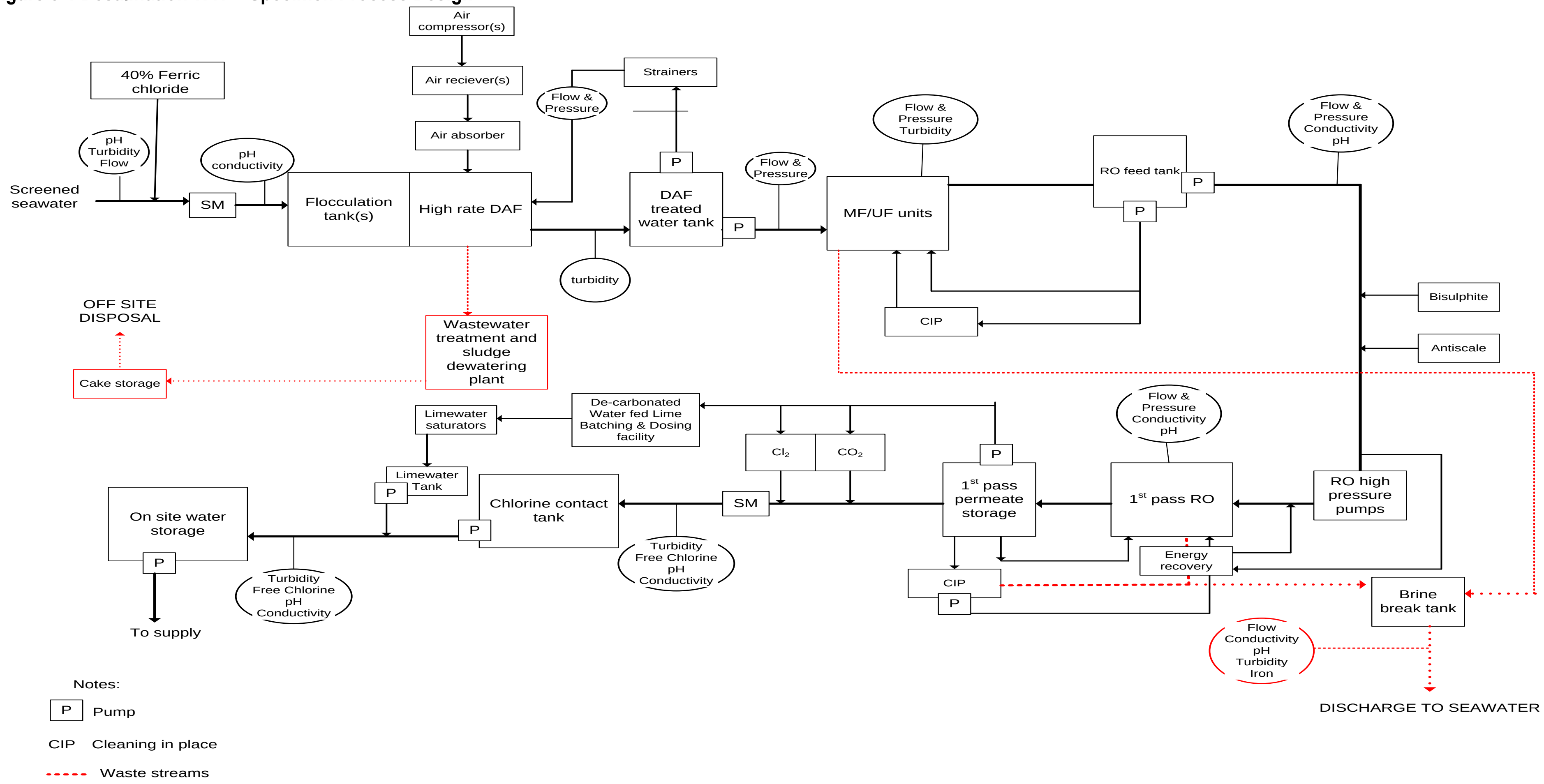
The required land take for a Desalination Treatment Plant would be of the order of 20 to 25 hectares, and would include a number of intermediate storage tanks such as Dissolved Air Flotation (DAF) treated water and micro (MF)/ ultrafiltration (UF) treated water tanks; these tanks are recommended to allow easy commissioning, and aide operational staff in providing ease of operation for start-up and shutdown if required.

A number of items are described in Sections 3.2 to 3.4 inclusive to describe outline process related issues; but does not describe in any detail related civil, mechanical or electrical design issues which will have effects on the project and or costs, such as availability of power and potential requirement for further power generation facilities; and which would have significant cost implications to the desalination treatment option.

3.1 Desalination WTP - Specimen Design (Schematic)

Figure 3-1 shows a typical specimen design for a Desalination WTP.

Figure 3-1 Desalination WTP - Specimen Process Design



3.2 Desalination - Outline Process Description

S.I. No. 122 of 2014 - European Union (Drinking Water) Regulations 2014 states that the maximum boron requirement shall be 1.0mg/l; therefore, with correct 1st pass reverse osmosis membrane selection and a sufficiently high feedwater pH (approximately 7.5 and above), achievement of the required 1.0mg/l can be met using a single pass Reverse Osmosis (RO). A single pass RO will save capital expenditure (CAPEX) costs when compared to a two pass system. The total life cost economics between using ultra low energy 1st pass membrane and a partial 2nd pass RO membrane configuration; and a 1st pass RO membrane selection using low energy (but not ultra-low energy) RO membranes would have to be undertaken to ascertain the most economical solution. However given the adoption of the “state of the art” energy recovery systems using pressure exchangers, where efficiencies as high as 95% can be achieved, it would seem reasonable to assume that Figure 3-1 is a reasonable initial design.

3.2.1 Intake systems- summary and environmental issues

Marine life impingement and entrainment *of organisms* resulting from intake operation of a desalination plant is proving to be one of the most political and highly visible environmental issues facing seawater desalination plants. Impingement occurs when marine organisms are trapped against intake screens by the velocity and force of water flowing through them. The fate of impinged organisms differs among intake designs and the type and population of marine life present in the intake area. Some hardy species may be able to survive impingement on an intake screen and be returned to the sea, but the 24hour survival rate of some species and/or juvenile fish may be less than 15%. The number of affected organisms varies considerably with the volume and velocity of feedwater and the use of mitigation measures developed to minimize their impact. If intake velocities are sufficiently low, fish may be able swim away to avoid impingement or entrainment. The swimming performance for different species of fish can predict the most vulnerable types and ages, however, even large fish are frequently caught on intake screens, indicating that swimming ability is not the only factor in impingement. Cold temperatures or seasonal variations in age-selective migrations or growth are also factors which would need to be considered within a more detailed design.

Large seawater desalination plants have traditionally employed open sea, surface water intake arrangements. Such arrangements are the type through which most electric power generation plants obtain condenser cooling water where water is pre-screened using traveling water screens, mechanically cleaned bar screens, or passive "well screens." In many instances, the screening chamber is located on or near shore and the intake pipe may extend out hundreds of metres into the sea. Travelling Water Screens - Travelling Water (Band) Screens have been employed on seawater intakes since the 1890s. The screens are equipped with revolving wire mesh panels having typically 6mm to 9·5mm openings. As the wire mesh panels revolve out of the flow, a high-pressure water spray removes accumulated debris, washing it into a trough for further disposal. The screens can be located onshore, at the end of a channel or forebay that extends out beyond the surf zone, or at the end of a pipe that extends out into the sea, terminating in a vertical *velocity cap*[5] inlet. Travelling water screens may be modified to collect fish and return them to the water body with a minimum of stress to improve survival. The cover converts vertical flow into horizontal flow at the intake entrance to reduce fish entrainment. It has been noted that fish will avoid rapid changes in horizontal flow and velocity cap intakes have been shown to provide 80 to 90% reduction in fish impingement and a 50 to 62% impingement reduction versus a conventional intake. As with all intake configurations, there are many design issues that must be considered, and the performance of a velocity cap may vary in still water versus areas subject to tidal cross-flows.

[5] The cover placed over the vertical inlet of an offshore intake pipe.

3.2.2 Open intakes

An open (or direct) surface intake can range in design complexity from simply attaching an intake pipe and screen assembly to an existing structure, to modifying an existing intake or outfall line that may have been inactive, to constructing a dedicated stand-alone structure. A typical open intake design includes intake screens, conveyance piping, and a wet well or other mechanism for housing the system pumps. Common intake design alternatives include the following:

- Dock, pier or bulkhead, i.e. existing structure mounted screens;
- Wet well intake sumps with subsurface intake lines that extend to off-shore screens;
- Wet well intake sumps with exposed intake lines anchored on the seabed extending to off-shore screens;
- Wet wells constructed into rock bluffs/cliffs with an intake line drilled through the rock into the seawater with or without an attached screen;
- Shoreline structures with open bay and bar rack screens;
- Directionally drilled lines under and through the seabed with screens; and/or
- Forebay/pump stations in sheltered settings (for example sloughs or coves).

'The Plan' referred to an intake point at approximately 3km "out to sea", due to minimum height requirements from the seafloor (to avoid sand and silt ingress) and a minimum depth below sea top level to avoid contact with shipping; this seems a reasonable approach, but would be expensive compared to systems with shorter intake pipework.

The required pump station is usually a wet well or sump structure in which pumps are mounted. The pump station would be located on-shore at a site that allows easy access and connection to the desalination plant These structures can be quite large, as they usually include pumps, controls, chemical feed equipment (if necessary), large primary screening devices like bar rack screens, secondary traveling screen assemblies, multiple chambers, and a backwashing (sparging) system.

At the open intake located 3km out to sea, a coarse bar type arrangement would be utilised to prevent large marine animals or fish becoming entrained; this would still require regular (once to twice per annum) inspection to ensure large fish and sea mammals are not trapped. Passive Screens at the pump station would be required - Passive Screens' are being considered on many open sea intakes as one proven means of greatly reducing impingement and entrainment. A passive screen intake arrangement at the pump station utilizes slotted screens constructed of trapezoidal-shaped "wedgewire" cylinders. The screens have openings that range from 3 mm to 9.5 mm and are usually oriented on a horizontal axis with screens sized to maintain a velocity of less than 15 *cm/s* to minimize debris and marine life impingement. Passive screens are best-suited for areas where an ambient cross-flow current is present, and air backwash system is usually recommended to clear screens if debris accumulations do occur. As with all submerged equipment, material selection should reflect the corrosion and bio-fouling potential of seawater; copper nickel alloys are generally recommended. Passive screens have a proven ability to reduce impingement and entrainment, and their effectiveness is related to their slot width, and through-flow velocity. It has been demonstrated that 1 mm openings are highly effective for larval exclusion and reduce entrainment by 80% or more. The depth and orientation of passive screens should be such that it will not draw water from a particularly sensitive area, particularly if juveniles or larvae may be present in large numbers during some times of the year.

3.2.3 Beachwells

It is unlikely that beachwells will be economic in this instance given the abstraction requirement. Subsurface intakes may consist of horizontal or vertical beach wells, infiltration galleries, or seabed filtration systems. In each of these designs, the open seawater is separated from the point of intake by a geologic unit. In addition to providing some natural filtration, this arrangement has the advantage of separating most of the marine organisms from the water intake. The use of subsurface intakes offers a distinct environmental advantage because the ecological impact associated with impingement and entrainment of marine life is virtually eliminated. However, subsurface designs should consider their potential negative impact on nearby fresh groundwater aquifers. In some cases, subsurface intakes must be evaluated and regulated as groundwater sources. Vertical beach wells are shallow intake wells that make use of beach sand or other geologic structure as a filter medium. Vertical beach wells can be an economical alternative to open sea intakes for desalination plants with capacities less than 20,000 m^3/day. They have the advantage of delivering "pre-filtered" water that may greatly reduce additional pre-treatment requirements.

A vertical beach well consists a non-metallic casing, well screen, and vertical turbine pump. Site suitability is determined by drilling test wells and conducting a detailed hydrogeologic investigation to determine the formation transmissivity and substrate characteristics. It is preferred to locate beach wells as close to the coastline as possible, and the maximum yield from individual wells range from 0.1 to 4000 m^3/d. Radial Well - Radial collector wells are a variation of the beach well where multiple horizontal collector wells are connected to a central caisson which acts a wet well or pumping station from which water is pumped to the desalination plant. The use of multiple horizontal wells means that the production of each radial well can be significantly greater than a single vertical well. Individual horizontal wells can be drilled or well screens can be hydraulically jacked out from the bottom of the caisson using a direct· jack or pull-back process. The caisson can be completed with a flush grade top slab or in a buried concrete vault and backfilled with 0.5-1.0m of beach sand to reduce visual impact.

For larger capacity plants, a variation that combines concepts from both the beach well and radial well intake arrangements could be feasible. Using a slant -well drilling technique, also referred to as 'horizontal directional drilling', and one or more wells drilled from shore at a 20-25 degree angle under the seabed may be feasible for some locations with seawater intake capacities of up to 189,250 m^3/day.

A beach well gallery system can be an acceptable intake alternative when the thickness of beach or the adjacent onshore sediments is insufficient to develop beach wells with economic yields or when the permeability of the sediment or rock is relatively low. There are several possible gallery design configurations but for this case the more common type was used which is a horizontal collection system with a single trench. The design features common to a beach well gallery system are:

1. The wells are 25.4 cm in diameter, and 6.1 m deep;
2. The horizontal screens or lateral collectors are sized so that the inflow velocity through the intake screen is not greater than 3cmls;
3. The lateral screens are surrounded by a graded gravel pack having a radius from the centreline of the screen of approximately 1.22 m;
4. All gallery pumps are stainless steel, vertical, turbine with a rated pumping capacity of 22 l/s at a total dynamic head of 30.5 m; and
5. A stand-by pumping capacity equal to no less than 25% of the total rated design of capacity of the gallery is provided

With regards to intake pipework length, the absolute minimum length will correspond to a distance whereby the intake pipe opening can be located at no less than 3m from the seabed. There is a risk that by not extending this height to 5 to 6m that increased sand ingress will occur; DAF as a pre-treatment methodology minimises the risk.

3.3 Power requirement and associated specific RO membrane selection

It is likely that a single pass RO system will be sufficient for treating Irish seawater. The power requirements will, be dependent on the salinity of seawater and temperature required. '*The Plan*' states 3.58KWh/m^3 with energy recovery; this is in reality probably based on temperatures above 12°C and salinity marginally below 36g/l. Additionally it would appear that the '*The Plan*' has been based on utilisation of ultra-low energy RO membranes, although there is nothing wrong with this basis, a clear indication of treated water quality with particular emphasis on total dissolved solids (TDS) must be ascertained prior to providing a firm power requirement. It should be noted that consumers not used to saline drinking water may well perceive a salty taste at concentrations as low as 250mg/l to 350mg/l, and hence utilisation of an RO membrane with marginally better salt removal characteristics but unfortunately higher power may be required. The Drinking Water Regulations state that the absolute maximum conductivity of water should be no greater than 2,500 µs/cm at 20^{o}C, this would equate to a total dissolved solids of approximately 1,500mg/l, however as mentioned above some consumers may taste the salt at as low concentrations as 250mg/l to 350mg/l of sodium chloride, hence it is advised that the TDS limit of reverse osmosis permeate be set at 100mg/l, the TDS of water sent to distribution will be higher as carbon dioxide, limewater (and potentially magnesium sulphate) and chlorine (or hypochlorite) would increase the TDS.

Additionally sampling to ascertain seawater boron concentrations in the vicinity of the intake will be required, on the basis that seawater boron in the locality is 5mg/l or below, utilisation of a single pass RO system will be sufficient. In order to guarantee a permeate TDS of below 100mg/l and therefore no potential issue with taste issues associated with salinity, an energy usage of approx. 3.75 to 4.05 Kwh/m^3 may be required (for desalination excluding pre-treatment) based on specific membrane selection, seawater salinity and temperature.

3.4 Treatment processes options

3.4.1 Pre-treatment

Choice of pre-treatment technology is critical for the successful operation and minimisation of desalination operating cost. SWRO success is built on effective pre-treatment; dissolved air flotation (DAF) and membrane (micro (MF) or ultrafiltration (UF)) have been selected as the best available pre-treatment technologies for the proposed desalination facility abstracting water from the Irish Sea.

The choice of pre-treatment is highly dependent on the prospective seawater quality feeding the SWRO plant. Any likelihood of high solids loading, such as may be witnessed if seawater intake point is closer to shore and or possibility of high algal counts mean that utilisation of DAF is recommended. DAF systems have a relatively short "start up and shutdown period" and hence operators can decide when operation of DAF is/ is not required. The system could be designed with UF pre-treatment alone, however the design of UF would then have to be sufficiently conservative to deal with seawater not previously treated with DAF; in essence this would mean a lower UF flux and hence a greater number of installed UF membranes. The cost implications of such options would require a dedicated study.

In summary the benefits of MF or UF as opposed to sand or dual media filters for SWRO pre-treatment are:

- Improved pre-treated water quality;
- Lower suspended solids;
- Lower microbiological contaminants resulting in improved RO performance;
- Fewer RO cleanings - lower operating cost;
- Lower RO pressure drops from fouling – lower energy cost;
- Longer RO membrane life;

- Increased RO flux rates, therefore fewer RO membranes required;
- Smaller footprint size for pre-treatment and marginally smaller for RO; and
- Lower chemical and sludge handling cost compared to granular media filters.

MF and UF technologies are very space efficient as compared to granular media filtration. The smaller footprint benefits of membrane filtration are of greater importance where the cost of new land acquisition is significant. Depending on the type and size of the MF or UF modules and the intake water quality characteristics, the membrane filtration system may have a 20% to 60% smaller footprint than a conventional filtration system. Generally, under typical surface-water quality conditions, the footprint of granular media filters, designed at a surface loading rate of 11 to 12 m/hr. is approximately 50% larger than that of MF or UF systems producing similar filtered water quality. For better-than-average influent water quality (such as when DAF is also utilised as a pre-treatment method), where granular media filters can perform adequately at 15 to 20 m/hr of hydraulic surface loading rate, the footprint difference is usually 20% to 40% in the benefit of MF or UF filtration.

Granular media filtration systems use a limited amount of power to separate particulates in the source water.

MF or UF systems will consume a greater amount of power. More power is not only used to create a flow-driving pressure through the MF or UF membranes but also for membrane backwash and feedwater pumping. The total power use has to be taken into consideration when completing a life-cycle cost comparison of granular medial versus MF or UF systems for SWRO pre-treatment. Single-stage gravity type granular media filtration systems have previously yielded more favourable economy-of-scale benefits; however, this is changing as MF or UF systems and products have been further optimized for large capacity treatment plants such as the plant in discussion.

The MF/UF design could utilise either a relatively large number of pre-engineered units (detailed design dependant on MF/UF vendor selected by contractor) or a “one off design” specifically for this desalination plant. This issue would be resolved during early stage design. The MF/UF plant would include dedicated cleaning in place (CIP) and flushing systems. Use of a dedicated MF/UF treated water tank is recommended to provide easier commissioning and operation of the plant, however contractors may not offer this facility due to its cost impact.

3.4.2 Desalination

Reverse osmosis (RO) is undoubtedly the most economical desalination technology currently for seawater desalination where free or very cheap heat sources (steam) are unavailable.

The configuration of seawater RO units will primarily depend on treated water volumetric and quality requirements. On the basis that the treated water quality required meets the requirements of the Drinking Water Regulations, one important issue in deciding the RO plant configuration would be boron, on the basis that seawater boron was no more than 5mg/l, a single pass RO (with appropriate seawater RO membranes) would be sufficient to meet the required boron concentration of 1.0mg/l; it should be noted that membrane selection will be critical to ensure that a single pass RO system is sufficient. Use of a single pass RO will reduce power requirements compared to utilisation of a 2 pass RO system.

On the basis that a single pass system is utilised, a typical RO membrane plant in this instance would comprise the elements noted in Table 3-1.

Table 3-1 Outline RO Pressure Vessel and Unit Sizing at Different Project Timelines

Plant treated Water flow (Ml/d)	Year	Number of individual 1st pass RO units (racks) (based on 14 pressure vessel high x 18 pressure vessel's)	Number of pressure vessels installed per unit	Total installed RO membranes for plant	Actual unit (rack) capability in terms of space for pressure vessels.
160	2024	8 or 9	252	14,112 to 15,876	280 pressure vessels capacity per unit i.e. 10% additional capacity if required.
240	2030	11 to 12	252	19,404 to 21,168	
320	2050	15 to 16	252	26,460 to 28,224	

Note:

- Each rack having capability to produce 15.8 Ml/d ad flow factor of 0.85 (based on DOW FILMTEC RO membranes).
- 7 membrane elements per installed pressure vessel.
- Unit height based on 10 pressure vessels high; this could in reality be increased to 14 and hence reduces overall land area required.
- RO membrane units; 252 pressure vessels installed, but with capability to install an additional 28 pressure vessels if required, thereby making unit (rack) total gross design capacity of 280 pressure vessels if required.
- Actual design can be adjusted to favour reducing either Capital cost (less installed membranes), which would result in a high RO flux and thereby higher energy costs. Flux in the example above is 12.71 lmh, which could be increased, but provides a relatively lower operating cost. The fact that power costs are relatively high favour utilisation of a conservative (lower) flux.

3.4.3 Post treatment

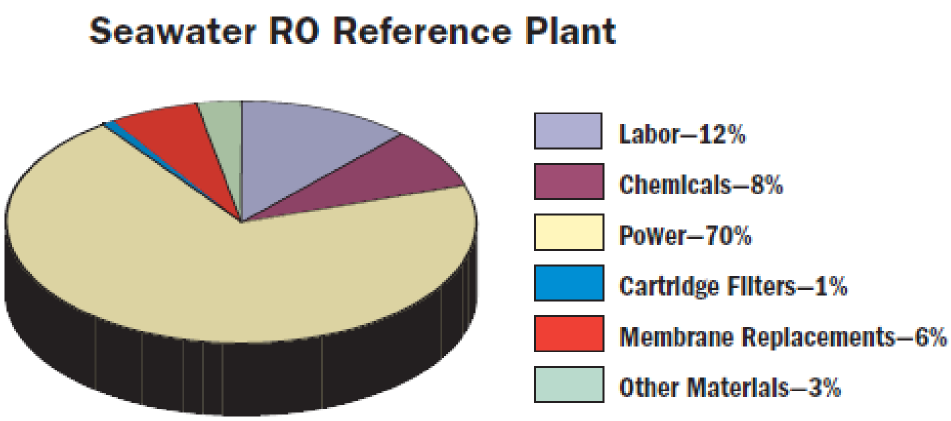
RO permeate will require alkalinity addition, remineralisation and disinfection; see Figure 3-1. There are various options open for such a large desalination plant. Alkalinity addition will undoubtedly be provided for by carbon dioxide addition, the quantity of carbon dioxide addition will depend on the RO feedwater pH and consequently how much free carbon dioxide is present within the RO feedwater (which will be dependent on RO feedwater pH). Carbon dioxide would be stored in cryogenic carbon dioxide storage tanks and delivered to site as and when required. The choice of remineralisation chemical between use of limestone filters and limewater is dependent on whole life costs between the two systems. Limewater will undoubtedly be the cheapest CAPEX option, the higher operating cost of hydrated lime compared to limestone needs to be accounted for in the "whole life cost" calculation. The frequency of plant operation needs to be considered in deciding the optimum solution, in addition to local cost and availability of high quality limestone and hydrated lime powder. The choice of disinfectant will also need careful consideration, and H&S issues as well as disinfectant by-product (primarily potential presence of bromate in bulk hypochlorite solutions) needs to be considered. Chlorine gas will undoubtedly provide the lowest whole life cost for the project; however for such a large plant the presence of multiple one tone chlorine drums on site needs to be considered with regards to potential site location options. Bulk supply of sodium hypochlorite will provide the lowest capital cost option, but the largest operating costs of the disinfectant chemical options listed below:

- On site generation of sodium hypochlorite - highest CAPEX, but lowest OPEX
- Chlorine gas
- Bulk delivery of sodium hypochlorite – lowest CAPEX but highest OPEX

3.5 Summary of Typical Operational costs

Typical split of operating cost profile for a SWRO is shown in Figure 3-2.

Figure 3-2 SWRO Operating Cost Profile



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