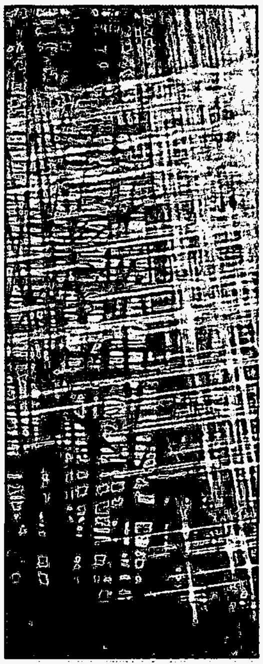

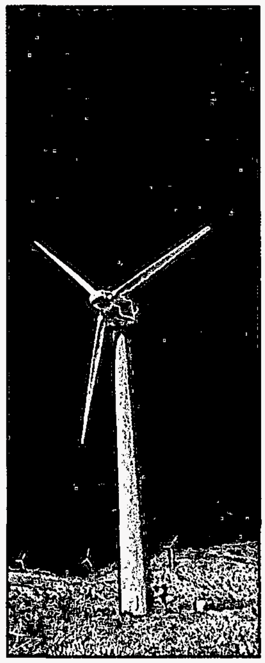

Ballygarvan Waste Water Treatment Plant Upgrade

Design Review Report

September 2010

Cork County Council

Mott MacDonald

Ballygarvan Waste Water Treatment Plant Upgrade

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Cork County Council

County Hall, Cork

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Appendices

Executive Summary

At present the existing Ballygarvan WWTP treats the wastewater discharged from 135 houses within the Bride View Developments housing estates as well as the wastewater discharged from the local school and some commercial units.

The current loading on the treatment plant is estimated to be 29.78 kg BOD/day in a dry weather flow of 99.89 m3/day, corresponding to a population equivalent of 496 pe.

The design loading for treatment is estimated to be 38.06 kg/day in a dry weather flow of 125.91 m3/day corresponding to a population equivalent of 634 pe.

The existing treatment plant does not incorporate all of the treatment processes necessary (primarily nitrogen reduction) to produce a treated effluent in compliance with the current discharge licence, and to enable the receiving waters comply with current water quality objectives.

Although the relevant treatment stages are in place the treatment process operation is not achieving the required level of BOD, suspended solids or phosphorus removal.

This report recommends that to meet the required standards for nitrogen in the treated effluent, a biological nitrogen removal system should be provided. This requires installation of an anoxic tank (32 m3 precast tank) and pumping system to recirculate nitrified liquor from both aeration tanks to the anoxic tank, and a modified sludge system to pump sludge from both settlement zones to the anoxic tank. Furthermore, sludge should be removed more frequently from the system to prevent excessive build up and carryover of solids in the effluent discharge.

To achieve the required BOD and suspended solids standards in the effluent, this report recommends installation of an automatic tertiary sand filter prior to discharge via the existing discharge monitoring chamber.

To meet the required phosphorus standards, a standby dosing pump should be provided. The operation of these could be linked to the operation of the inlet pumps. The tertiary sand filter will also assist in further reducing phosphorus levels in the treated effluent.

A schematic flow diagram of the recommended upgraded wastewater treatment plant is included in Appendix A.

The capital cost of undertaking the recommended upgrade measures is estimated at €131,900.00 plus VAT @ 13.5% (€149,706.50 in total).

1. Introduction

This Design Review Report is prepared by Mott MacDonald Ireland in accordance with the Project Brief for the Upgrading of the Ballygarvan Wastewater Treatment Plant for Cork County Council, and subsequent project meeting on 30th August 2010.

In preparing this report the existing wastewater treatment plant was visited on 3 separate occasions. The following is a non-exclusive list of documents that were referenced and utilised in the preparation of the report.

- Brief for the Engagement of a Consulting Engineer for the Design, Tender, Construction and Handover for the Upgrade of the Ballygarvan Wastewater Treatment Plant
- Cork County Council Discharge Licence No. WP(W) 5/01
- Carrigaline Electoral Area Local Area Plan 2005
- Report on Evaluation of Proposed Upgrade Ballygarvan WWTP (HRA)
- Owenboy River Catchment Assessment & Supplementary Reports (Dixon Brosnan)
- EPA manual "Treatment Systems for Small Communities, Business, Leisure centres and Hotels".
- The European Communities Environmental Objectives (Surface Waters) Regulations 2009
- Cork County Council results of River Owenboy quality analysis for the period January 2005 to April 2010
- Drawings and manuals of existing gravity sewers and wastewater treatment plant provided by Cork County Council
- Proposals for upgrading Ballygarvan WWTP (EPS) dated 2nd September 2008 and 21st January 2010

The primary objective of this review report is to determine the works that are necessary to be undertaken to upgrade the Ballygarvan WWTP to ensure that it complies with its current licence and with applicable and relevant legislation. The design capacity for the upgraded plant has been determined based on servicing the properties currently serviced, the new school and those adjacent houses that can be serviced by gravity connections to the existing sewers.

2. Design Loading

2.1 Current WWTP Loading

At present the existing Ballygarvan WWTP treats the wastewater discharged from 135 houses within the Bride View Developments housing estates of Gleann Rua (36), Gleann Dara (32) and Gleann Alainn (66) as well as the wastewater discharged from the local school and some commercial units.

The 2006 Census recorded an average of 2.9 persons per private household in Cork County (occupancy of 2.6 in Cork City). Application of this occupancy rate for Ballygarvan would yield a population of 392 persons in the Bride View Developments estates. A wastewater flow of 160 l/hd/day is considered appropriate for such new residential development.

The local primary school currently has 265 students enrolled and has a staff of 13. The wastewater loading generated by the school corresponds to a population equivalent of 93 pe (based on 20 g BOD/hd/day and 40 l/hd/day for non-residential schools with no canteen as set out in the EPA manual "Treatment Systems for Small Communities, Business, Leisure centres and Hotels"). Similarly allowing for a nominal occupancy of 20 in the crèche, its wastewater discharge corresponds to a population equivalent of 7 pe.

In addition, there are 4 commercial units/offices plus the local shop (temporarily closed) discharging to the WWTP. The wastewater flow and BOD loading generated by these is calculated based on the recommended wastewater loading rates from offices as set out in the EPA manual "Treatment Systems for Small Communities, Business, Leisure centres and Hotels". Allowing for an average occupancy of 3 people for 8 hours per day in each of these the occupancy of 15 corresponds to a population equivalent of 5 pe (based on 20 g BOD/hd/day and 30 l/hd/day).

The current wastewater flow and load to the treatment plant based on these standard unit rates is calculated and presented in the following table (Table 2.1). It is acknowledged that recent water meter readings at the school indicate a daily water usage of approximately 4 m^3, and the difference between this and the calculated wastewater flow generated may be because this is a primary school with a shorter operating day than a secondary school. However, it is considered prudent to allow for the larger figure in determining the wastewater treatment capacity required.

An allowance for infiltration into the collection system corresponding to 50 l/hd/day has also been included in the calculation of the daily wastewater flow to the treatment plant.

Based on the foregoing the total flow of wastewater being treated was estimated at 99.89 m^3/day (say 100 m^3/day) based on standard unit rates. This compares well with the measured flow of treated effluent currently being discharged from the treatment plant. There is a flow monitoring chamber fitted with a V-notch weir and ultrasonic level sensor fitted on the treated effluent discharge line from the plant. Over a period from 31st August to 3rd September the total daily flow readings were reviewed, and these indicated an average daily throughput of 118 m^3/day. The difference between the measured and estimated flows may be attributed to the continuous spray of potable water onto the surface of the aeration tanks for the purpose of foam suppression. Therefore the estimated flow is considered a good representation of current wastewater flows.

Table 2.1 Current (2010) Wastewater Flows & Loads being treated at the WWTP

Contributor	Quantity	Occupancy	Population	BOD g/hd/day	Flow l/hd/day	BOD kg/day	Flow m^3/day
Residential Units	135	2.9	392	60	160	23.52	62.72
Primary School	1	278	278	20	40	5.56	11.12
Crèche	1	20	20	20	40	0.40	0.80
Commercial Units/shop	5	3	15	20	30	0.30	0.45
Infiltration	142		496	-	50	-	24.8
Total						**29.78**	**99.89**
						496 pe	

2.2 Proposed WWTP Loading

The existing school is to be replaced by a new school constructed across the road and connected to the sewers discharging to the WWTP. The new school will have a student population of 480 with a staff of 20. This will increase the occupancy of the school by 222 corresponding to an additional wastewater flow and load 4.44 kg BOD/day and 8.88 m^3/day and approximately 74 pe.

Consideration is also to be given to the treatment of the wastewater generated by 12 existing properties near the crossroads and the school, as well as from the adjacent area (R-02) zoned for medium density residential development. The latter is an area of 0.4 Ha, and the appropriate housing density for this area is 12-25 housing units per Ha (Carrigaline Electoral Area Local Area Plan 2005). At a density of 25 per Ha, this area would accommodate up to 10 houses. At an occupancy rate of 2.9 people per household, these 22 additional houses would add a further wastewater loading of approximately 64 pe to be treated at the upgraded plant.

The future load to be treated at the upgraded WWTP is set out in the following table (Table 2.2).

Table 2.2 – Future Wastewater Flows & Loads to the WWTP

Contributor	PE	BOD kg/day	Flow m^3/day
Current Loading	496	29.78	99.89
Additional school loading	74	4.44	8.88
Additional existing residential	35	2.10	5.60
Future residential development (R-02)	29	1.74	4.64
Additional Infiltration	132	-	6.90
Total	**634**	**38.06**	**125.91**

In addition to BOD the other characteristics of concern in the influent to the treatment plant are Total Nitrogen, Total Kjeldahl Nitrogen, total phosphorus, Fats, Oils & Grease.

Current results of analysis of the influent were not available at the time of writing, so reference is made to typical concentrations encountered previously on other similar catchments. Since this is primarily a residential catchment located adjacent to the treatment plant, the majority of the nitrogen in the influent will be in the ammonia form, and total nitrogen will correspond closely to total kjeldahl nitrogen. Based on similar catchments (and until results of analysis of raw wastewater at this site are available) the following table shows the assumed influent concentrations and loads for these other parameters.

Table 2.3 – Assumed Influent Concentrations and Loads

Parameter	Concentration (mg/l)	Daily Load (kg/day)
Ammonia	55	6.93
Total Nitrogen	60	7.56
Phosphorus	12	1.51
Fats, Oils & Greases	< 10	< 1.26

3. Existing WWTP

3.1 Description of Existing WWTP

The Raw sewage gravitates to an underground sump at the WWTP site. This is fitted with two submersible pumps that operate on a duty/standby basis. These pumps are fitted with shredding mechanisms to break down gross solids/rags in the wastewater. The pumps were not removed or inspected, but are understood to be in reasonable condition.

The actual output of these pumps was not available, but the overall output rate from these pumps should correspond approximately to the overall discharge rate from the treatment plant (excluding any potable water added) as measured by the final effluent flowmeter because all the tanks are flow through tanks with fixed top water levels. The maximum rate of discharge observed during visits to site in early September 2010 was 3.4 l/s. This gives at least an estimate of the order of magnitude of the pump rate.

These pumps operate for a total of approximately 4.3 hours per day (based on hours run readings from the control panel) to pump the current quantity of wastewater discharged to the wastewater treatment plant. Their operation is controlled by the level in the sump.

The absence of a permanent automatic screening device had been putting pressure on the inlet pumps, and a temporary manual screening system has been utilised in recent times. This comprises a cage with wide apertures between the bars on its sides. This is manually lowered into position in front of the incoming sewer, manually removed from the sump, and the screening manually hosed from its surface.

The pumps discharge to two pre-fabricated package type activated sludge units that operate in parallel on a continuous basis. Each of these units incorporates an aeration zone with an operational capacity of approx. 54 m^3 and a quiescent settlement zone with a plan area of approximately 9 m^2. The air supply to the aeration zones is met by two rootes type blowers (one per tank) that deliver air through submerged coarse bubble air diffusers. Each blower is capable of delivering 180 Nm^3/hr.

Ferric sulphate is added to the contents of both aeration tanks by a single dosing pump. This pump is located in the control kiosk, mounted adjacent to the control panel. A plastic bucket underneath provides the only form of spill collection. Ferric sulphate is stored on site in two IBC units mounted over a designated bund. However, there is no emergency shower or eyewash facility in the vicinity of either the dosing pump or storage bund.

Mixed liquor from the aeration zone passes directly into the settlement zone from where settled sludge is recirculated to the aeration zone using an air lift system. Excess sludge is removed only at intervals of three months, causing a build up and retention of solids with potential for carryover of solids in the treated effluent. The resultant elevated MLSS levels in the aeration tank increase the oxygen demand and can reduce DO levels in the tank.

Treated effluent overflows from the settlement zone of each unit and combines before passing directly to the final effluent monitoring chamber. This incorporates a V notch weir and ultrasonic sensor, and a sampling unit. The instantaneous flows and the cumulative flow measured by this meter are indicated on the plant control panel. These flow readings were reviewed over a few days and the readings corresponded with the total daily flows one would expect from a sewerage catchment of this size.

A pair of tertiary upward flow clarifiers installed downstream of the final settlement zones had been fitted with filter media and sand, but it is reported that these were extremely problematic. These have been taken out of service and are currently being bypassed.

3.2 Performance of Existing WWTP

The Historical records of treated effluent quality were reviewed and these indicate variable quality of effluent. Summary results included in the HRA report for the period 2002 to 2007 indicated very variable and poor quality treated effluent, while those for 2008 showed an improvement, with BOD results within the specified limit of 10 mg/l. However, suspended solids generally exceeded the 10 mg/l limit. Total nitrogen also exceeded its limit of 15 mg/l, and this is directly due to the absence of any nitrogen reduction facility in the treatment process.

Available results of analysis for the period October 2009 to May 2010 are presented in the following table. These are extremely variable, and this may be due to the infrequent desludging of the plant, the build up of solids within the system, and the spraying of potable water onto the surface of the aeration tanks. During the week of 30th August to 3rd September 2010 the treated effluent appeared very clear and of good quality. (Results of analysis of current treated effluent are awaited.)

Table 3.1 – Treated Effluent Quality

Date	pH	B.O.D. mg/l	C.O.D. mg/l	Suspended solids mg/l	FOG mg/l	Ammonia NH_3-N mg/l	Nitrate NO_3-N mg/l	Ortho-phosphate PO_4-P
Discharge licence Limits	**6.0 – 9.0**	**10**	**-**	**10**	**5**	**(total N:15)**		**1.5**
30/10/09	7.01	2	<1	9.7	<1	-	1.2	2.13
25/03/10	7.27	23	105	47				
02/04/10	7.46	5	29	12	?	25	45	4.25
12/04/10	7.27	<2		3.1	1.2		6.4	1.96
14/5/10	7.23	50	180	153				

The existing treatment plant is not producing treated effluent in compliance with the limits set out in the Cork County Council Discharge licence. The existing plant is not capable of consistently meeting these current licensed limits, particularly in relation to nitrogen.

4. Treated Effluent Standard Required

The treated effluent standards required to ensure compliance with current wastewater treatment regulations and to ensure that the receiving waters (Owenboy) comply with applicable regulations are determined in this section. They are determined based on the assimilative capacity of the river, taking into account its flow and both the background and maximum permissible concentrations of relevant parameters. These standards are then compared with those contained in the Cork County Council Discharge licence and those recommended in the Owenboy Catchment Assessment. It should be noted that both of these pre-date the European Communities Environmental Objectives (Surface Waters) Regulations 2009 (S.I. No. 272 of 2009) which has implications for the required quality of the receiving waters and consequently the treated effluent discharged from the Ballygarvan WWTP.

4.1 BOD and Suspended Solids

The Owenboy River has been assessed in accordance with the Water Frame Work Directive and its overall ecological status at Ballygarvan was determined to be "moderate". The European Communities Environmental Objectives (Surface Waters) Regulations 2009 requires that such waters classified as less than good must be restored to at least good status generally by 22 December 2015. Table 9 of Schedule 5 of these Regulations requires that for Good status the BOD must be ≤ 1.5 mg/l (mean) or ≤ 2.6 mg/l (95%ile). Since background concentrations of ≥ 2 mg/l have been recorded at Ballygarvan, it is the 2.6 mg/l (95%ile) limit that is considered appropriate for Ballygarvan.

The 2008 report provided by Dixon Brosnan estimates the BOD concentration in the river at 2.5 mg/l (based on sample concentrations of <2 and 3 mg/l for the year 2006). The Cork County Council results of analyses of samples taken over the period March 2007 to April 2010 indicate an average BOD concentration of 1.33 mg/l, with only 2 samples with BOD concentrations higher than 2 mg/l (3.5 mg/l on 30/05/2007 and 8.4 mg/l on 17/06/2009). Therefore use of 2.5 mg/l as the background river concentration appears appropriate.

At the 95 percentile flow of 0.1497 m^3/s in the Owenboy at Ballygarvan, and the background BOD concentration of 2.5 mg/l, an increase of only 0.1 mg/l in BOD concentration would be permissible if the river concentration is not to exceed the 2.6 mg/l concentration required for it to achieve "good status". Under these conditions the assimilative capacity in terms of BOD at Ballygarvan is calculated as 1.306 kg/day.

Based on this assimilative capacity and the proposed treatment plant throughput of 125.91m^3/day (Table 2.2), the BOD concentration of the treated effluent discharged from Ballygarvan WWTP should not exceed 10.37 mg/l.

If treated effluent is discharged at the 25 mg/l concentration required by the Urban Waste Water Treatment Regulations for secondary treatment plants, it would increase the BOD concentration in the river by 0.24 mg/l.

It is noted that the Dixon-Brosnan report "Owenboy Catchment Assessment" includes calculation of assimilative capacity and determines that a maximum load from 3,595 pe can be treated to a discharge standard of 20 mg/l BOD without increasing the BOD concentration in the river by more than 1 mg/l. (Table 28.2) It further determines that a discharge from 5,000 pe at a suspended solids concentration of 30 mg/l will not increase the concentration in the river by more than 1 mg/l, which will remain below the required 25 mg/l concentration level. (Table 29.1) However, in Section 33.6.3 the same report recommends a treated effluent discharge standard of 10 mg/l for both BOD and suspended solids, without providing any basis for lowering the concentration limits from those calculated based on assimilative capacity and compliance with legislation (i.e., 20 mg/l and 30 mg/l respectively).

The Cork County Council Discharge Licence issued in 2001 also sets limits of 10 mg/l for BOD and suspended solids.

The existing WWTP has been in operation since 2002 and has been generally discharging treated effluent at concentrations closer to, and in excess of 20 mg/l until 2008, and the river retained its Q4 quality status throughout that period.

However, based on compliance of the receiving waters with the European Communities Environmental Objectives (Surface Waters) Regulations 2009 and the Water Framework Directive, a limit of 10 mg/l is considered appropriate for the discharge from the Ballygarvan WWTP.

In achieving a BOD concentration of 10 mg/l, the technology and treatment process to be implemented will also reduce the suspended solids concentration to less than 10 mg/l.

10 mg/l is therefore appropriate as the maximum concentration for BOD and suspended solids in the treated effluent discharge based on a background river concentration of 2.5 mg/l for BOD.

4.2 Phosphorus

In accordance with the assessment of the overall ecological status of the Owenboy River at Ballygarvan as "moderate", and the requirement that such waters classified as less than good must be restored to at least good status generally by 22 December 2015 (The European Communities Environmental Objectives (Surface Waters) Regulations 2009) the Molybdate Reactive Phosphorus (MRP) concentration in the river should be ≤ 0.035 mg/l (mean) or ≤ 0.075 mg/l (95%ile).

The results of Cork County Council analysis of samples of the river water at Ballygarvan for the period March 2007 to April 2010, indicate an average concentration of MRP in the river of 0.032 mg/l. The 95 percentile compliance concentration limit of 0.075 mg/l in the river and this average concentration are adopted as the basis for determining its assimilative capacity and the concentration limit for MRP in the treated effluent discharge.

Based on the 95 percentile flow of 0.1497 m^3/s in the Owenboy at Ballygarvan, and the background MRP concentration of 0.032 mg/l, the assimilative capacity in terms of MRP at Ballygarvan is calculated as 0.561 kg/day. In a treated effluent discharge of 125.91 m^3/day this corresponds to a MRP concentration of 4.46 mg/l, and a total phosphorus concentration of 5.58 mg/l).

This is considerably higher than the MRP discharge concentration limits proposed by Dixon Brosnan. However, the latter were based on compliance with the earlier Water Quality Standards for Phosphorus Regulations 1998 which were based on compliance with a median concentration limit to achieve a particular Biological Q rating. This method has now been superseded by the requirement for compliance with the more recent regulations (2009) to ensure compliance with the Water Framework Directive.

The Cork County Council Discharge Licence issued in 2001 sets a limit of 1.5 mg/l for total phosphorus.

On the basis of the currently licensed limit of 1.5 mg/l and the adequate assimilative capacity available in the Owenboy River for phosphorus, the limit of 1.5 mg/l for total phosphorus is considered appropriate. This is not considered to be a particularly onerous standard to achieve using chemical precipitation.

4.3 Nitrogen

To achieve the "good" status required for compliance with the Water Framework Directive, the total ammonia (expressed as mg/l N) should be ≤ 0.065 mg/l (mean) or ≤ 0.140 mg/l (95%ile).

The results of Cork County Council analysis of samples of the river water at Ballygarvan for the period March 2007 to April 2010, indicate an average concentration of Ammonium in the river of 0.032 mg/l. This corresponds to an ammonia Nitrogen concentration of 0.105 mg/l. The 95 percentile compliance concentration limit of 0.140 mg/l in the river and this average concentration are adopted as the basis for determining its assimilative capacity and the concentration limit for Total Ammonia Nitrogen in the treated effluent discharge.

Based on the 95 percentile flow of 0.1497 m^3/s in the Owenboy at Ballygarvan, and the background concentration of 0.105 mg/l, the assimilative capacity in terms of Ammonia Nitrogen at Ballygarvan is calculated as 0.457 kg/day. In a treated effluent discharge of 125.91 m^3/day this corresponds to an Ammonia Nitrogen concentration of 3.63 mg/l.

In relation to Ammonia Nitrogen, the Cork County Council Discharge Licence issued in 2001 did not include any limit and the report by Dixon Brosnan did not recommend a limit for it.

However, for compliance with the European Communities Environmental Objectives (Surface Waters) Regulations 2009 and the Water Framework Directive it is recommended here that a limit of 3 mg/l be set for Total Ammonia Nitrogen in the treated effluent discharge. This is not considered to be a particularly difficult standard to achieve using biological nitrification.

In relation to Nitrate Nitrogen the European Communities Environmental Objectives (Surface Waters) Regulations 2009 do not set any limits. However, the European Communities (Quality of Surface Water Intended for the Abstraction of Drinking Water) Regulations 2009 set a limit of 50 mg/l for Nitrate. This corresponds to 11.29 mg/l Nitrate N. It is however considered appropriate that the level in the river is not allowed to exceed 50 % of this value, i.e., a limit of 5.65 mg/l for Nitrate Nitrogen.

At an average concentration of 4.936 mg/l Nitrate Nitrogen (21.86 mg/l Nitrate) over the period from March 2007 to April 2010 and a 95 percentile flow in the river, the assimilative capacity for nitrate nitrogen is calculated as 9.26 kg/day. If the effluent is discharged from the treatment plant without undergoing any specific nitrate removal process, the nitrate nitrogen discharged would be in the region of 5.7 kg/day.

The Cork County Council Discharge Licence for this plant issued in 2001 sets a limit of 15 mg/l for total Nitrogen. If 3 mg/l of this allowance is allocated to Ammonia Nitrogen in the discharge, then 10 mg/l would be allocated to nitrate nitrogen, with the balance allocated to other forms of nitrogen in the effluent. Discharge of treated effluent at a nitrate nitrogen concentration of 10 mg/l would correspond to an emission of 1.26 kg/day.

The Urban Wastewater Treatment Regulations sets total Nitrogen limit of 15 mg/l for discharges to sensitive areas from treatment plants with a capacity of between 10,000 and 100,000 pe. Although the Owenboy River is not designated as sensitive and the Ballygarvan WWTP is below the size range specified, it is considered prudent to adopt a total nitrogen limit of 15 mg/l for the treated effluent discharge from the Ballygarvan WWTP. Associated with this limit should be a limit of 10 mg/l for nitrate nitrogen and 3 mg/l for ammonia nitrogen.

4.4 Detergents, Chlorine, Oils, Fats and Greases

While detergents and chlorine are found in domestic wastewater, they are not usually present in quantities that would cause problems to either the wastewater treatment process or the quality of the receiving waters. They generally only reach problematic levels when there is a commercial laundry or industry in the catchment.

Similarly fats, oils and greases (FOG) tend to reach problematic levels only where there are dairy related facilities, mean plants, butchers, abattoirs, fast food restaurants or commercial kitchens in the sewerage catchment. Even then, the discharges from such facilities is controlled by wastewater discharge licences (or planning conditions) which require specific FOG removal facilities to be provided and operated at source. These generally impose limits for FOG.

Neither of these parameters is reported to be causing problems or difficulties in relation to the quality or status of the receiving waters.

Therefore, it is not considered necessary to impose restrictions other than the limits already in place as contained in the existing Cork County Council discharge licence.

4.5 Summary

It is recommended that the concentrations of the key parameters in the treated effluent should not exceed the following limits.

Table 4.5 – Maximum Permissible Concentrations in Treated Effluent Discharge

Parameter	Emission Limit Value
pH	6.0 – 9.0
BOD	10 mg/l
Total Suspended Solids	10 mg/l
Oils, fats & greases	5 mg/l
Total Nitrogen (as N)	15 mg/l N
Ammonia Nitrogen	3 mg/l N
Nitrate Nitrogen	10 mg/l N
Total Phosphorus	1.5 mg/l P
Detergents	5 mg/l
Chlorine	1 mg/l

5. Treatment Plant Upgrade Requirements

5.1 Preliminary Treatment

Inlet Pumping

The existing pumps operate for approximately 4.3 hours per day to pump the current quantity of wastewater discharged to the wastewater treatment plant indicating that they would have adequate capacity to pump the projected increased wastewater flow of 126 m^3/day. The pumps are currently operational and replacement in the short term is not considered necessary.

To achieve the required level of nitrogen reduction it will be necessary to construct an anoxic tank upstream of the two existing aeration tanks. To achieve gravity flow from this to the aeration tanks, the top water level in the anoxic tank will have to be approximately 500 mm higher than that in the aeration tanks. This additional static head should not significantly reduce the output from the inlet pumps, and at worst it is likely that the pumps will operate for more time each day.

The existing delivery pipework from the pumps is only 50 mm in diameter, and replacement of this with 80 mm diameter pipework and fittings will reduce the friction head in the discharge, and this will be directed to the proposed anoxic tank.

Screening

The existing manual screening system currently employed is not sustainable. To protect the inlet pumps and to prevent blockages in the aeration system it is necessary to install an automatically cleaned screen that will remove the screenings out of the flow and dewater them prior to discharge into a designated container. It is not considered necessary to provide for fine screening to 6 mm because the wastewater will subsequently be shredded in any case by the inlet pumps Therefore a screening aperture in the region of 10 mm would be appropriate. The screen should be automatically raked and should discharge the screenings directly into a bin at ground level. There are numerous such screens commercially available.

It would be important that there is an automatic bypass facility around this screen, so that if it breaks down or becomes blocked the raw sewage will not back up in the incoming sewer. This could take the form of an overflow arrangement to the downstream side of the screen.

Fats, Oils & Grease Removal

At this treatment plant there are no visible deposits of fats, oils and grease, (FOG) or evidence in the inlet sump or throughout the treatment plant that these are at a problematic level. It is generally only on catchments with poorly controlled discharges from commercial kitchens and food industries that these are present at levels that can cause problems in the wastewater treatment systems. Removal facilities are usually installed at the source of such discharges to the sewerage network.

It is generally only in the influent to wastewater treatment plants of industrial dairy and meat units and other food industries or in the discharges from commercial kitchens that FOG builds up to the degree illustrated in the photographs included in the HRA report. On municipal and domestic wastewater treatment plants it is not normal to provide specific fats, oils and grease removal facilities, and any FOG present in the influent tends to be removed in the screening process. It is our experience that fine screens installed at the inlet to the treatment plant will capture a considerable proportion of the FOGs within the screenings removed.

Therefore, although the Cork County Council Discharge Licence for the facility states that *"the influent shall pass via fat grease traps prior to biological treatment on site"* (Section 2.1) it is not considered necessary to provide specific fats, oils and grease removal facilities on site.

The FOG level in the treated effluent should continue to be monitored and if the required level of FOG in the effluent discharge is not achieved, then a programme of adding FOG removal enzymes should be implemented on a trial basis for a period of time.

Untreated Wastewater Storage

Section 8.4 of the Cork County Council Discharge Licence states that "*standby storage facility for untreated effluent shall be installed to accommodate untreated wastewater in the event of a malfunction, or, breakdown of the effluent treatment.*" However, the licence does not specify the storage capacity to be provided. Where such storage capacity is included in planning permissions and or licences it is usually specified to be equivalent to the quantity of wastewater generated over 24 hours. For Ballygarvan WWTP this would therefore require a storage capacity of approximately 126 m^3.

While such untreated wastewater storage may be specified in discharge licences or planning permissions for some private residential developments, it is not a normal requirement for a public or local authority treatment plant. Based on the gradient of the incoming sewer and local ground levels around the wastewater treatment plant there is storage capacity within the incoming sewer and inlet pump sump for approximately 10 m^3 (without flow backing up in individual sewer connections to private properties.

Such upstream storage would only be necessitated if the inlet pumps or screen break down or block or if there is a power failure at the site. To overcome these potential occurrences, it is essential that a critical alarm and "call out" signal is sent to the person responsible for the plant operation in the event of either a power failure, screen or pump trip or if high level is detected either upstream of the screen or in the inlet pump sump. To overcome a power outage at the treatment plant, a section and socket should be installed in the control panel to facilitate the plugging in of a portable generator. This would be brought to site and plugged in upon receipt of a power failure alarm. It would not be normal practice to provide a permanent standby power generator (and associated fuel storage system) at a plant of this small scale.

At present there is no overflow connection on the inlet pump sump. An overflow connection could be provided to a new raw wastewater storage tank as required by the discharge licence. The inlet to this tank would be approx 600mm below existing ground level, and it is envisaged that it would normally be empty. It would only receive a flow when the inlet pump sump overflows to it. The tank should be fitted with a continuous level monitor with an alarm signal automatically sent to the person responsible for the plant. Since this tank would be called into service only on rare occasions, a portable pump could be used to empty its contents and either return the untreated wastewater at a very slow rate to the inlet pump station when the operation of the pumps is restored. However, this rate would have to be very slow to avoid the risk of overloading the plant. As a result untreated wastewater may remain in the tank for more than a few days, causing it to become septic and odorous, and discharge of such sewage into the treatment plant may disrupt the treatment process. In such conditions, it would be more prudent to transfer the tank contents to another larger treatment plant by tanker. Once the tank has been emptied it would be essential that it be washed out immediately to reduce odour potential.

It would be essential that such a tank would be roofed and vented through a passive odour treatment unit, because of the potential for odour generation, particularly when it contents are being agitated and emptied. In summary such a storage tank would have to be a below ground tank (due to site topography) with an operational capacity of 126 m^3. The maximum capacity of precast tanks readily available is 36 m^3, and up to 4 of these precast tanks in series would be required, or alternatively a single tank could be constructed in on site.

In summary there is significant capital expenditure required for the provision of untreated effluent storage on site. While there is also considerable operational input required in emptying the tank and cleaning afterwards, it is envisaged that the storage tank would only be called into service on very rare occasions. The cost of providing automatic callout alarms and use of a portable generator is not significant, and would enable the portable generator to be used at other sites (WWTPs or pump stations) when necessary.

5.2 Anoxic Tank

The required operational capacity of the anoxic tank has been calculated as 32 m^3 based on the total nitrogen load set out in Table 2.3 and on the assumption that complete nitrification (to nitrate) will be achieved in the aeration tanks. This has also assumed an average de-nitrification rate typical for domestic wastewater. This would be operated with an MLSS concentration in the region of 3,000 to 3,500 mg/l.

This tank could be a precast concrete or pre-fabricated steel tank. This would reduce construction and installation time on site. The closest standard size of precast concrete tank readily available is manufactured by Carlow Precast with a nominal capacity of 36 m^3. This will provide an operational capacity of 32.5 m^3 with a freeboard of 300 mm. The delivery time for this tank is currently 1 to 2 weeks.

To minimise construction and general civil costs, this tank could be installed overground, however, this would increase the ongoing pumping head for the inlet pumps, nitrified liquor recirculation pumps and the sludge return pumps. To minimise pumping costs the anoxic tank would be partially submerged and its sidewalls would extend to approx 1.0 m above ground level.

The water table at the site is understood to be within 1.0m of ground level due to the proximity of the Owenboy River flowing alongside the site, and the site has flooded in the past. Therefore floatation is likely to be an issue in relation to new structures. To overcome this, reinforcing steel bars would be extended from the anoxic tank walls and a hoop of concrete would be constructed around the tank to anchor it and prevent floatation.

The anoxic tank should be fitted with a mixer (a submersible type is considered most appropriate) that will operate on a continuous basis.

The outlet from the anoxic tank will be connected to the existing inlets to the aeration tanks.

5.3 Aeration Zones

The recommended ammonia and nitrate nitrogen limits in the treated effluent can be met by providing a biological nitrogen removal system incorporating a nitrification stage in the aeration tank and an upstream anoxic zone. The removal of BOD within the anoxic zone will reduce the BOD load to the aeration tank and accommodate the overall increase in BOD load to the plant without the necessity for an additional aeration tanks. The combined operational capacity of the two existing aeration tanks is adequate for nitrification and BOD removal while operated at a minimum MLSS concentration of 2,750 mg/l.

However, the existing aeration system does not have adequate output to meet the required oxygen demand. There are two options available to increase the aeration capacity. The first is retention of the existing coarse bubble diffused air system in the aeration tanks and replacement of the air blowers with much larger units. This option would require a total air throughput of approx 850 Nm^3/hr, or 425 Nm^3/hr per blower. The existing blowers have an output of 180 Nm^3/hr each.

Alternatively if the coarse bubble diffusers are replaced with flexible membrane fine bubble diffusers, with higher oxygen transfer efficiency, the existing air blowers would have adequate capacity and could be retained in use.

To ensure there is sufficient oxygen available (1 – 2 mg/l) the dissolved oxygen content in the aeration tanks should be monitored. If the site is to be visited on a regular (daily) basis a potable hand held DO meter may be adequate. Alternatively this could be done by installing permanent DO sensors and monitors for each tank.

To achieve de-nitrification of the nitrified wastewater, the mixed liquor from the aeration tanks should be re-circulated to the anoxic tank. Since both existing aeration tanks are close together, a high level pipe from each could be diverted to a common sump. This should be fitted with two submersible pumps (duty/standby) capable of pumping up to 16 m^3/hr.

5.4 Settlement Zones

The existing settlement zones in each treatment tank structure have a surface area of approx 9 m^2 each, and are considered adequate for the hydraulic throughput of the plant based on current pumping rates (which are not going to be increased).

In addition to the nitrified liquor from the aeration tanks, the return activated sludge should also be pumped back to the anoxic tank rather than to the aeration tanks. To achieve this, the existing air lift sludge return system from the settlement zones should be modified and a common pumping system provided to divert flow to the anoxic tank at a rate of 8 m^3/hr.

To maintain the MLSS concentration in the aeration tanks at the optimum level, excess sludge should be removed from the system on a regular basis. This would reduce the risk of solids carryover and blockage of the tertiary treatment stage. This could be achieved by conversion of one of the existing final effluent upward flow clarifiers to a sludge storage tank, and connection of a valved branch onto the delivery line from the sludge return pumps.

The existing overflow outlets from the two integral settlement chambers are to be diverted to a new sump from where the effluent will be pumped to a new automatic tertiary sand filter.

5.5 Phosphorus Removal

The addition of ferric sulphate to the aeration tanks is the most widely used method of removal of phosphorus in domestic wastewater treatment plants. The current installation of two IBCs over a common bund is acceptable for current purposes. However, there is no emergency shower or eyewash on site, and such a facility should be provided adjacent to the bund.

The current installation of the dosing pump inside in the control kiosk is not satisfactory, and this should be re-located to the storage area. This pump and a standby pump should be installed on a frame on top of the bund wall, so that any spills or leaks are contained within the bund.

The existing dosing points are acceptable. However, the dosing pipework from the new pump location should be contained within another larger pipe or provided with some form of protection from damage and prevention from freezing.

The ferric sulphate will cause the phosphorus to precipitate out with the activated sludge in the settlement zones. Further removal of the precipitated phosphorus will be achieved in the tertiary filtration stage.

5.6 Tertiary Treatment / Filtration

To achieve treated effluent BOD and suspended solids concentrations of < 10 mg/l it is necessary to provide a tertiary treatment stage. For a plant of this scale the most economic system is an automated sand filter. An alternative would be to install membrane filters in the aeration zones, and discharging treated effluent directly from these. However, membrane filters tend to be expensive and require considerably higher aeration input than the standard activated sludge system. Finer screening of the influent (to approx 3 mm is also necessary) when membrane filters are used.

Therefore, the recommended form of tertiary treatment to be provided here is automatic sand filtration. There are many tertiary sand filters commercially available including the type proposed by EPS. The head loss across a tertiary sand filter and associated pipework and fittings will be in the region of 1.0 m. However, this head is not available between the settlement tanks, the effluent monitoring chamber and the river. It is therefore necessary to pump the treated effluent through the filter. This will be a pre-fabricated/precast sump fitted with two pumps to operate on a duty/standby basis and provided with a level controller. These will have a pumped capacity of approximately 7.5 m^3/hr.

The sand filter proposed by EPS appears to be a downward flow filter that is backwashed when the water level above it rises to a pre-set level. It provides for backwash with filtered effluent only and does not incorporate an air agitation or scouring stage in the cleaning cycle. It is considered likely that this filter will require considerable attention in relation to cleaning and prevention of clogging.

An alternative to the EPS filter is an automatic sand filter manufactured by Toveko. This is a continuous up flow sand filter. As the water flows up through the filter, suspended solids are removed and retained within the sand bed. The filtered water overflows from the filter through V-notch openings along its entire length. An airlift pump continuously transfers the polluted sand from the bottom of the bed to an open sand washer above the filter. The airlift pump is mounted on the sand washer carriage, which moves slowly along the entire filter length. The sand washer consists of an archimedean screw rotating within an inclined channel. The cleaned sand discharges back onto the bed via a chute being distributed evenly by distribution plate and levelled out by a scraper mounted on the carriage. The rate of sand cleaning is controlled automatically according to the head loss across the filter as sensed by a level device in the inlet launder by adjusting the air supply to the left pump and the operating speed of the washer screw.

These filters are provided with an automatic head-loss detection system. This means that, not only will use of wash water stop during periods of no-flow, but also that the rate of sand washing increases and decreases according to the needs of the filter. This means that, unlike other similar products, its washer system is self-compensating when presented with temporarily high feed solids loading.

The unit incorporates a set of duty and standby washwater pumps that utilise filtered effluent directly from the top of the filter (rather than from a separate sump) and a compressor package (to operate the airlift pump and the washwater pumps). The water supply pumps can be mounted directly on a bracket onto the side of the filter tank. The overall power requirement of the filter and its ancillaries (including wash pump and compressor) is 2.2 kW.

The filter tank is fabricated in grade 304 stainless steel and contains approx 2.2 tonnes of filter sand, with a typical media replacement cycle of 5 years. It has an overall footprint of 1.5m x 1.5m and would be installed on a concrete plinth near the final effluent monitoring chamber.

This filter has a delivery time of up to 12 weeks at present, and could be installed over a period of less than a week following completion of the other elements of work on site.

5.7 Electrical Installation and Controls

There is a 63 Amp incoming power supply which would be adequate for a total usage of approximately 30 kW on site. At present the principal electrical power users on site are the inlet pumps (2 kW) and the two duty air blowers (4.5kW), indicating a maximum demand at present in the region of 13 kW.

Provision of a new inlet screen (typical 2.2 kW), anoxic tank mixer (say 1.5 kW), nitrified liquor recirculation pumps (1.5kW), sludge return pumps (1.5 kW), tertiary filter feed pumps (2 kW) and tertiary filter and ancillaries (2.2 kW) would impose an additional demand of approximately 11 kW. This indicates that the existing power supply to the site is adequate to meet the power demand requirements for the upgraded plant.

The control panel in generally in good order but requires some upgrades to fuse protection, and securing of panel equipment. The control panel was energised while the door was in the open position which is potentially dangerous. Some of the door mounted timer equipment was redundant as this had been replaced by other equipment. Ferric sulphate low level alarm was not operational as the probe was not installed in the Ferric Sulphate IBC.

All power/controls cabling enters the control panel from underneath. There appears to be enough slack in the cabling which should allow the control panel to be raised approx. 300mm above its present level to elevate it above the reported flood level. This would require that the existing kiosk be relocated onto a plinth. This could be a solid plinth or a frame.

There is a working telephone line located in the kiosk and a dial out alarm system which appears to send out a common alarm (alarm type to be confirmed by the caretaker) to a pre-programmed number.

A suitable proposal for upgrading the communications at this site would be a local control PLC with a HMI (Human Machine Interface) type operator's interface. We propose that this would have remote viewer access over the internet (no control anticipated) which would allow the council or the caretaker to access the site from any PC with internet access (password protected). Individual parameters required such as treated volume, inlet level alarm, motor alarms, ferric sulphate low level etc. would be available.

A more sophisticated alarm handling system could be installed which could send out voice or text messages for individual alarm events to a variety of phone numbers. A radio based communications system is unlikely to be successful at the WWTP since it is low lying and surrounded by hilly terrain.

5.8 General

There is a potable water supply to site, and this appears to be adequate to provide for spraying onto the aeration tanks, in addition to meeting the requirements of the welfare facilities on site. This should also be adequate to meet the demands of the recommended emergency shower and eyewash.

The existing tanks and facilities currently occupy less than half of the existing site, and there is adequate space available on site to facilitate the construction of the recommended additional chambers and tanks. There is good access to site, and although it isn't tarred, it should be adequate to accommodate the construction traffic and traffic associated with the operation of the plant.

This site flooded to a level of 21.13 mOD. Finished ground level on site varies, with discrepancies in the actual ground level data quoted. However, it is known that the flood level did not reach the top of the existing aeration tanks, but that it did enter the control kiosk. A more detailed review is required to facilitate development of a flood defence system for the site.

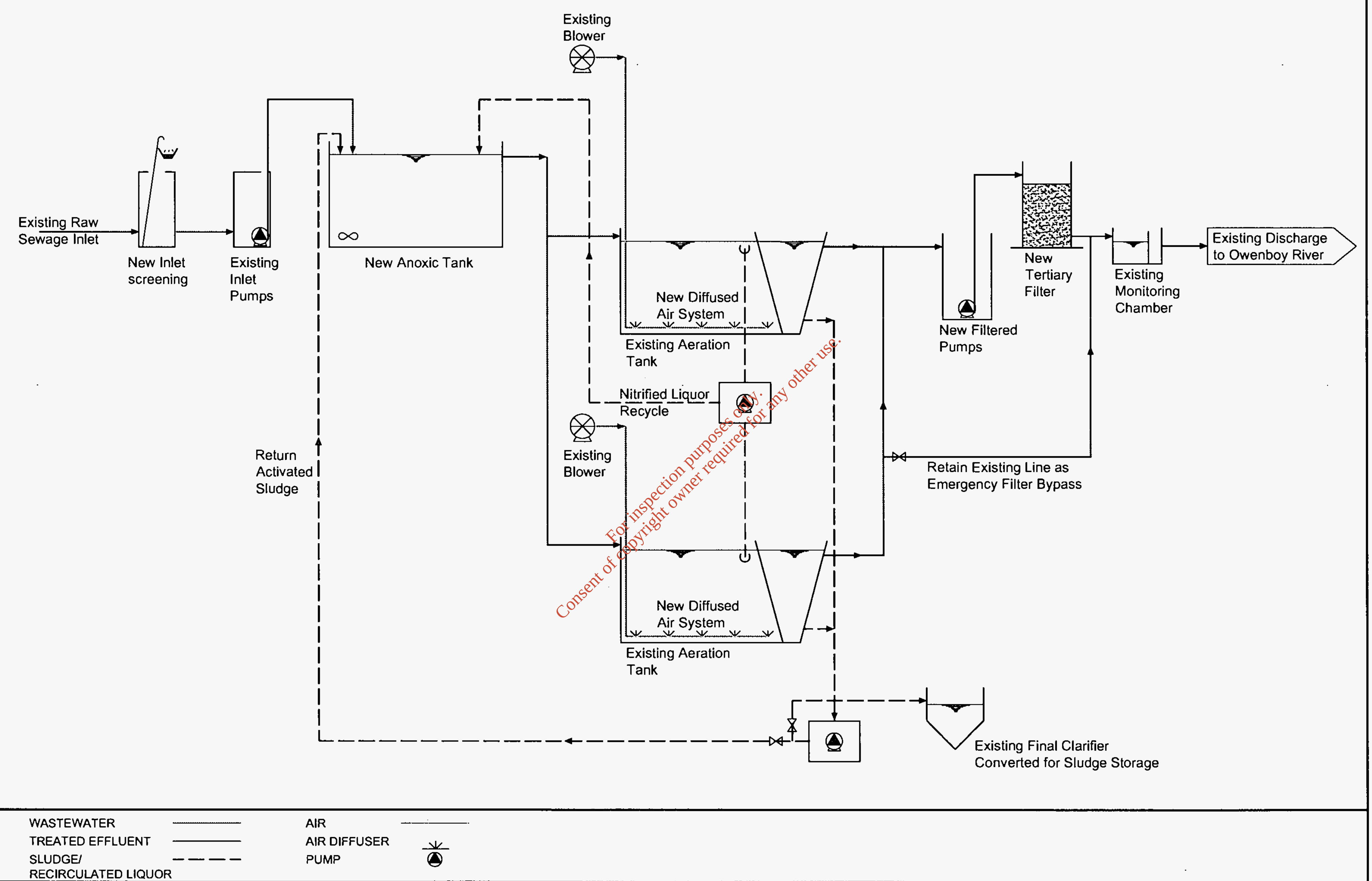
6. Cost Estimate

An outline estimate of the capital cost of undertaking the upgrade works recommended in Section 5 is summarised below. The costs include for civil, mechanical and electrical costs, and are factored up so that each includes installation and wiring and commissioning costs. Modifications to and re-routing of pipework are included where relevant. The costs of provision of flood defence system on site or of a telemetry and data transmission system to Cork County Council office is not included below.

Table 6.1 Budget Estimate

Nr	Item	Total Cost
1	Inlet Screening, chamber	€19,600
2	Modifications to inlet pumping arrangement, chamber and pipework	€5,250
3	Anoxic tank (32 m^3), mixer and associated civil works	€16,800
4	Replacement diffused air system in both tanks, plus provision of nitrified liquor return pumps, pipework, sump, portable DO meter and controls	€12,500
5	New sludge return system including pumps, pipework, sump and controls, plus modifications to existing system	€8,950
6	New ferric dosing pump, pipework and ducting, emergency shower system, relocation of existing dosing pump.	€6,850
7	Automatic tertiary filter with feed pumps, washing and compressed air system, pipework and fittings, support plinth and sump,	€43,050
8	Modifications to existing control panel and kiosk, new control panel, weatherproof kiosk and wiring of new equipment.	€7,900
9	General Items including preliminaries, training, drawings, documentation etc	€11,000
	Sub-Total	**€131,900.00**
	VAT @ 13.5%	**€17,806.50**
	Total	**€149,706.50**

Appendix A – Schematic Flow Diagram



Mott MacDonald

SCHEMATIC FLOW DIAGRAM OF PROPOSED BALLYGARVAN WWTP UPGRADE

BALLYGARVAN WWTP UPGRADE
DESIGN REVIEW REPORT
JOB NR. 282283
DRG NR. 282283-FG001

Mott MacDonald

Appendix B – Automatic Sand Filter

TOVEKO filters explained

Summary of advantages

1. TOVEKO filters are gravity-fed sand filters which continuously wash the dirty sand whilst in service. This means that:
   - a Unlike conventional pressure sand filters, they require no standby unit to maintain continuous service flow.
   - b The sand bed is maintained at a more or less constant condition of cleanliness at all times, giving rise to a constant high filtration quality.
   - c They do not require a large, separate system for batch-wise backwashing.

2. TOVEKO filters have a maximum height of 2.3m and an inlet height of 2.0. This means that:
   - a They will fit into a standard height room where required. (In addition, the filters are designed to pass through a standard 2 metre wide double door [S-type] or 3 metre wide double door [T-type]).
   - b Typically, gravity flow of the feed is possible, avoiding expensive pumping systems.
   - c Delicate sludge particles are not destroyed by pumps, allowing greater removal efficiency.

3. Wash water requirement is typically between 2 & 5% of feed flow, lower than its rivals.

4. The compact design requires only a minimum of space, and no tall structures.

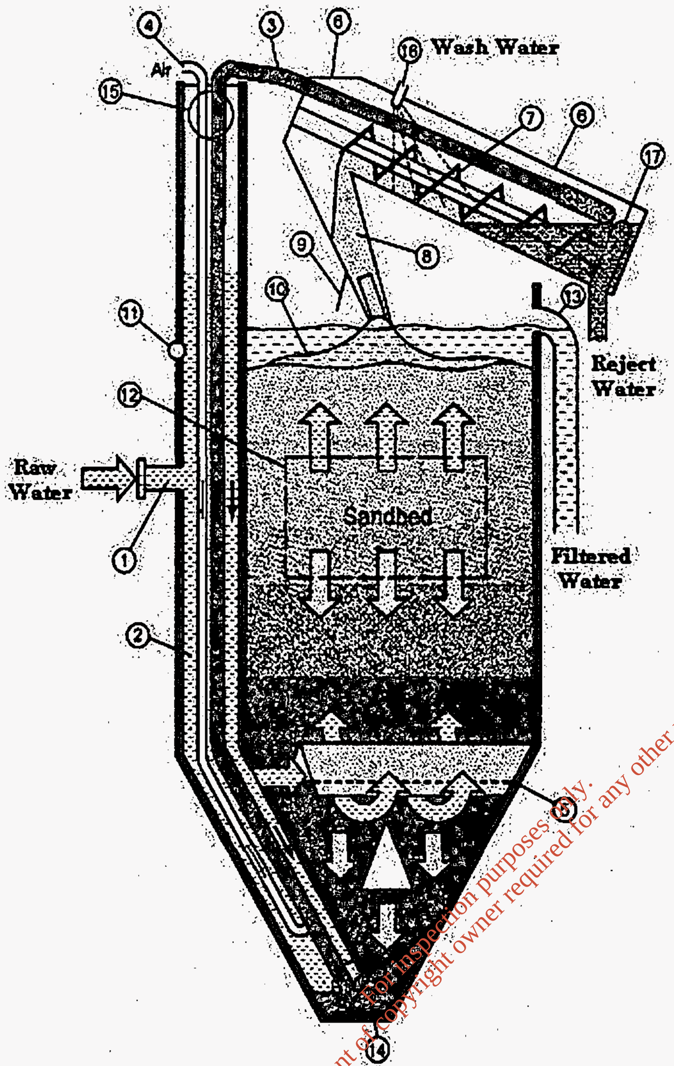
5. The unique sand washer is highly efficient, making it particularly suitable for removal of sticky and stubborn solids such as oil.

6. Each filter is provided with an inlet launder along its length. This enables it to cope with large fluctuations in both feed flow and pressure drop across the sand bed.

7. TOVEKO filters are provided with an automatic head-loss detection system. This means that, not only will use of wash water stop during periods of no-flow, but also that the rate of sand washing increases and decreases according to the needs of the filter. This means that, unlike other similar products, its washer system is self-compensating when presented with temporarily high feed solids loading.

8. There are no "dead" zones: the entire bed participates in the filtration process.

9. The surface of the sand bed is always levelled, eliminating shortcutting.



Functional Description

1. Inlet Pipe
2. Inlet Shaft
3. Air Lift Pump
4. Air Supply to Air Lift Pump
5. Distribution Channel
6. Moving Sand Washer
7. Sand Screw
8. Sand Discharge Chute
9. Sand Distribution Plate
10. Sand Scraper
11. Level Sensor
12. Moving Sand Bed
13. V-Notch Overflows
14. Sight Glass to Air Lift Pump
15. Emergency Overflow
16. Wash Water Nozzle
17. Reject Water Overflow

The TOVEKO CX is a continuous up flow sand filter for any type of wastewater or raw water treatment. The basic design is shown in the picture above. Each filter has the same cross-section, irrespective of its size.
The incoming water (1) is channelled through a longitudinal inlet shaft. (2)
The water then flows into the sand bed via distribution channels. (4)
As the water flows up through the filter, suspended solids are removed and retained within the sand bed. (12)
The filtered water overflows from the filter through V-notch openings along its entire length. (13)
An airlift pump continuously transfers the polluted sand (3) from the bottom of the bed to an open sand washer (6) above the filter.
The airlift pump (3) is mounted into the inlet shaft of the filter. Its suction tube penetrates 10-20 mm into the bottom of the sand bed.
The airlift pump is mounted on the sand washer carriage (6), which moves slowly along the entire filter length.
The sand washer consists of an archimedean screw (7) rotating within an inclined channel.
The polluted water and sand lifted up by the pump pours into the sand washer at its lower end.
As the archimedean screw rotates, the grains of sand are slowly rubbed against each other while wash water is added at the top of the sand washer via a nozzle (16), allowing the filtered solids to separate from the sand.
The cleaned sand then discharges back onto the bed via a chute (8), being distributed evenly by distribution plate (9) and levelled out by a scraper (10) mounted on the carriage.
The rinsing water is piped away (17) for further treatment (if required).

The two smallest filters, S-75 (1 m wide) and S-150 (2 m wide), each have one sand washer moving over the entire filter. Larger filters have two or more sand washers in which case each sand washer operates across a length of 2 metres. For example, the S-600 has 4 sand washers operating in a total filter length of 8 metres.

TOVEKO filters are also built as twin models (T-) in which case they are fabricated back to back with a common partition. Each half operates separately with its own washers and control system. Each filter has an automatic system for sand transfer and washing. The rate of sand cleaning is controlled automatically according to the head loss across the filter as sensed by a level device in the inlet launder by adjusting the air supply to the left pump and the operating speed of the washer screw.