CLARE COUNTY COUNCIL

ENNIS MAIN DRAINAGE INTERIM MEASURES

Preliminary Design Report
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1 **Introduction**

This preliminary design report details progress on the interim measures required to be undertaken at the existing Waste Water Treatment Plant (WWTP) at Clonroadmore, Ennis, Co.Clare. A Preliminary Report completed in 2002 recommended that this facility be replaced by a 50,000 PE WWTP at a new location in Clareabbey in the southern environs of the Town. The 2002 recommendations are currently being reviewed and updated with 2010 loadings and will be progressed by Clare County Council as part of the Department of Environment, Heritage & Local Government Water Services Investment Programme.

1.1 **Clonroadmore WwTP Description of Existing Facilities**

The Clonroadmore WwTP is located on a land bank between the railway and the River Fergus. It was originally constructed in the 1970’s with a capacity of 5,000 PE and has since been expanded to its current capacity of 17,000 PE. It receives pumped flows from the Tulla Road and Francis Street catchments of Ennis Town and liquor from the nearby Mart, nominally several days per week (continuous flow of c14m$^3$/d of high strength liquor).

The WwTW uses utilises an extended aeration process in 2 treatment streams one of which has been in operation since the original works was constructed and the 2nd stream was added during the mid-1970’s. Incoming flows converge in a single distribution chamber at the head of the works. 3mm mechanical screens and a macerator were recently installed to screen the influent flow. There are no grit removal facilities at the WwTW. Flows in excess of that which can be treated, spills over storm weirs to either of two storm tanks (561 m$^3$ each). This flow receives preliminary settlement only before overflowing directly to the River.

Full flow to treatment (FFT) flows into twin inlet channels separating stream 1 and 2. Both streams consist of rectangular aeration tanks each with 2,527 m$^3$ volume. Each tank is aerated by 2. No. 18.5 kW vertical shaft surface aerators. Stream 1 is served by a set of six rectangular upward-flow clarifiers (combined surface area 181.5 m$^2$) (original Imhoff tanks) and Stream 2 is served by a conventional circular clarifier (surface area of 254.3 m$^2$). Final treated effluent discharges to the River Fergus between Clonroad Railway River Bridge and Doora Bridge.

Return activated sludge (RAS) is returned from the settlement tanks to the aeration tanks via RAS pump station. Waste activated sludge (WAS) is withdrawn from the final settlement tanks to a sludge holding tank (2.5m diameter X 2.85 m high) and thickened on site using a double
belt press. Dewatered sludge (>16%) is removed off-site by Aqsolutions for composting. A thermal drying plant for sludge drying was removed from site in May 2007.

An assessment of the WwTW (Review of Clonroadmore WWTP Interim Measures Report May 2008) concluded that the plant has a current throughput to full treatment of 9,600 PE (based on organic load) with the potential to treat a loading equivalent of 17,600 PE. The report also concluded that the current operational inefficiency of the WwTP relates predominately to both dilute influent and a deficient stream 1 clarifier system.

2 Performance Requirements

2.1 Proposed Plant Design Capacity

The current population equivalent (PE), the dry weather hydraulic loading and the organic load for the Clonroadmore WWTP are estimated as circa 27,650 PE, 6,220 m$^3$/day and 1,660 kg BOD/day respectively\(^1\). The domestic population for the Clonroadmore catchment was estimated using the 2006 census Ennis electoral area CSO data and the existing non domestic dry weather flow (DWF) loading was estimated using data from the Clare County Council 2008 EPA licence application. The Clare Mart’s effluent, which is pumped directly to the WWTP, has an estimated loading of 105 PE/day. When added to the average estimated domestic and non domestic loadings gives an overall estimated loading to the WWTP of 27,650 PE.

The overall PE loading for the 2013 design year has been estimated at circa 30,150 PE based on the Ennis Main Drainage population projections. 2008 figures were projected forward to 2013 at a growth rate of 2% per annum for the domestic sector and 1.25% for non-domestic.

2.2 Proposed Effluent Standards

The following discharge standards were proposed in the 2008 Ennis Main Drainage Interim Measures Report based on the assimilative capacity of the River Fergus at the discharge point and relevant Water Quality Standards (prior to July 2009):

- BOD <15mg/l
- TSS 25 mg/l
- Ammonia <5mg/l measured as N
- Phosphorus <0.6 mg/l

\(^1\) Ennis Main Drainage Interim Measures Report 2008
The Publication of the European Communities Environmental Objectives (Surface Waters) Regulations 2009 in July has implications on the discharge standards from the Clonroadmore WwTW. These regulations set Environmental Quality Standards (EQS) for all surface water bodies and public authorities, in so far as their function allow, must ensure compliance with the good status parametric values by September 2015. The EPA have taken these Surface Water Regulations into consideration when issuing the Discharge Licence for the Plant.

The EPA issued a discharge licence to Clare County Council for the Ennis North agglomeration in September 2009 (D0048-01). Schedule A of this licence set out the following effluent quality standards for the Clonroadmore WwTW;

### Schedule A.1 Primary Waste Water Discharge

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Emission Limit Value (ELV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>7-9</td>
</tr>
<tr>
<td>Temperature</td>
<td>25°C (max)</td>
</tr>
<tr>
<td>COD</td>
<td>125</td>
</tr>
<tr>
<td>Suspended Solids (SS)</td>
<td>35</td>
</tr>
<tr>
<td>Orthophosphate (OP)</td>
<td>1</td>
</tr>
<tr>
<td>Total Nitrogen (as N)</td>
<td>15</td>
</tr>
<tr>
<td>Total Phosphorus (TP)</td>
<td>2</td>
</tr>
</tbody>
</table>
| cbBOD                    | 20                         | mg/l note 1
| Ammonia (as N)           | 6                          | mg/l note 2

Note 1  The ELV shall apply until 31st December 2010
Note 2  The ELV shall apply from 1st January 2011

The Emission Limit Values (ELVs) for COD, Suspended solids, TP and TN are as per the Urban Wastewater Treatment Regulations (SI no. 48/2010). BOD, Ammonia (N) and Phosphorus (TP and OP) ELVs have been set to ensure compliance with the Surface Water Regulations and ensure ‘Good Status’ in the River Fergus.
3 Process Design

General

A preliminary optioneering exercise concluded that extending the existing conventional process plant by increasing aeration, replacing of an existing clarifier and installing a tertiary filtration system coupled with improved sludge handling facilities would be adequate in the short term to treat the proposed design population to the required discharge standard.

The proposed upgrade is required to cater for a population equivalent of 30,150 PE with Ammonia removal via nitrification and phosphorus reduction. In order to meet the effluent total nitrogen limit in the discharge licence denitrification has also to be incorporated into the biological process. The activated sludge process was designed using the German Standard ATV-DVWK-A 131E, Dimensioning of a Single Stage Activated Sludge Plant.

Design Loadings

The Design influent loads are shown in Table 3.1

<table>
<thead>
<tr>
<th>Table 3.1</th>
<th>Clonroadmore WwTW Upgrade Design Loadings</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design Loadings</strong></td>
<td>30,150 PE</td>
</tr>
<tr>
<td><strong>Hydraulic Loading</strong></td>
<td></td>
</tr>
<tr>
<td>DWF</td>
<td>Unit</td>
</tr>
<tr>
<td>Infiltration</td>
<td>45 l/person/d</td>
</tr>
<tr>
<td>Total</td>
<td>6,784 m³/d</td>
</tr>
<tr>
<td>Peak Inflow to Works (Formula A)</td>
<td>401 l/s</td>
</tr>
<tr>
<td>Peak Load to Treatment</td>
<td>3 DWF</td>
</tr>
<tr>
<td>Full Flow to Treatment + Supernatant Return</td>
<td>188 l/s + (20 l/s)</td>
</tr>
<tr>
<td><strong>Organic Loading</strong></td>
<td></td>
</tr>
<tr>
<td>BOD</td>
<td>Unit</td>
</tr>
<tr>
<td>Total</td>
<td>1810 Kg/d</td>
</tr>
<tr>
<td>TSS</td>
<td>Unit</td>
</tr>
<tr>
<td>Total</td>
<td>2,111 kg/d</td>
</tr>
<tr>
<td>TKN</td>
<td>Unit</td>
</tr>
<tr>
<td>Total</td>
<td>362 kg/d</td>
</tr>
</tbody>
</table>
Effluent Standards

The Design Treated Effluent Standards are:

- BOD 10 mg/l
- Suspended Solids 35 mg/l
- Ammonia 1 mg/l as N
- Total Nitrogen 15 mg/l
- Total Phosphorus 2 mg/l
- Orthophosphate 1 mg/l

Preliminary Treatment

Preliminary treatment occurs at the Tulla Road and Francis Street main lift pump stations. The principal flows entering the inlet Works at these Pumping Station are generated by domestic, commercial and industrial premises in Ennis Town. All Flows entering the works will be screened through to a 6mm raked bar screen. The screened wastewater shall pass through a grit trap before passing through to the wet well of the pumping stations. New screens shall be located in existing channels. The inlet works equipment requires process water in a number of areas so that the units provided can fulfil their functions. Clean water shall be provided for washing of the raked bar screen. Re-circulated final effluent shall be used for flushing of the screenings launder and the grit classifier. The inlet works at each pumping station shall be capable of removing, washing and dewatering sewage screenings and grit from the process flow. The system shall incorporate air lift pumps to lift grit to an existing collection chamber. Grit is to be elevated to ground level and classified before disposal.

Storm Treatment

Gross solids and grit are removed from the incoming flows at Tulla Road and Francis Street Pumping Stations. It is proposed to upgrade the existing balancing tank and 2 no. stormwater tanks at Clonroadmore WwTW. The stormwater tanks are to be upsized and designed to contain flows in excess of the 3DWF (Formula A – 3DWF) for a period of 2 hours. 3DWF is Flow to Full Treatment (FFT). During periods of low flow, the contents of the storm tank return via gravity to the Flow Balancing, through inlet flow channels onto the Aeration Basins for treatment. Flow to the Aeration Tanks will be controlled by 2 new 300mm wall-mounted non-rising spindle type actuated penstocks which are to replace the existing manual penstocks. During storm return period, the contents of the tank are to be mixed by venturi mixers so as to reduce settlement and eliminate odours from these tanks. Mixers are sized to adequately clean each tank and prevent solids settling on the bottom.

2 Ennis Main Drainage Interim Measures Report 2008
Primary Treatment

None

Activated Sludge Design

Existing aeration tanks on stream 1 and stream 2 and the existing clarifier on stream 2 are to be re-used in design of the upgraded WwTW. Allowance has to be made in the design for the additional loads due to supernatant returns from the final tertiary treatment filters and the sludge treatment area. The main organic load in the supernatant will be primarily from the Gravity Belt thickeners. Existing aeration tanks to incorporate increased nitrification and carbonaceous BOD removal and introduce intermittent denitrification.

Nitrification:
Due to the high aeration and sludge age, nitrification takes place in the two aeration tanks. Nitrification is the biological oxidation of ammonia with oxygen into nitrite followed by oxidation of these nitrites into nitrates. The oxidation of ammonia into nitrite, and the subsequent oxidation to nitrate is carried out by two different bacteria both of which are autotrophic organisms, i.e they take carbon dioxide as their carbon source for growth. Therefore the activated sludge process is designed for carbonaceous BOD and nitrogen removal with an aerobic sludge age of 10 days. To achieve the required capacity in the existing aeration tanks the top water level (TWL) in each tank must be increased by 500 mm.

Oxygen requirements:
The oxygen consumption of the microorganisms results from the degradation of the carbon compounds and the oxidation of the nitrogen compounds. The calculated average standard oxygen transfer rate (SOTR) is 207 kg O$_2$/hr based on FFT. The Alpha and Beta values used in the calculations are 0.6 and 0.98 respectively.

To achieve the required oxygenation in the aeration tanks the aeration capacity must be increased in each basin. It is proposed to installed two 55kw aerators in each tank in place of existing units.

Denitrification requirements:
As a result of the designation of the River Fergus at the WwTW discharge point as sensitive, a total nitrogen standard of 15 mg/l has been included in the EPA discharge licence. To achieve this limit a period of denitrification has to be introduced into the existing activated sludge process. Denitrification is the biological reduction of nitrate (NO$_3$) to nitrogen gas (N$_2$) by
facultative heterotrophic bacteria. This reaction occurs when oxygen levels are depleted i.e. anoxic conditions when the dissolved oxygen concentration is less than 0.5 mg/l. A carbon sources is also required for denitrification to occur.

To achieve total nitrogen removal and conversion to nitrogen gas, a wastewater treatment plant design would include aeration basins for complete nitrification and sufficient tankage for denitrification to take place (anoxic conditions – mixing only). Alternatively, denitrification filters, downstream of aeration basins can be used.

Without denitrification the total nitrogen levels in a typical wastewater effluent are likely to be in the range 20-25 mg/l.

However, the influent Nitrogen (TN mg/l) to the Clonroadmore WwTW is much lower than typical values (Table 3.2).

Table 3.2  Clonroadmore Influent Total Nitrogen (mg/l)

<table>
<thead>
<tr>
<th>Sampling Period &amp; location</th>
<th>Average Total Nitrogen (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2007-2008 Clonroadmore WwTW influent*</td>
<td>20.8</td>
</tr>
<tr>
<td>15/07/09 – 28/07/09 Clonroadmore WwTW Influent**</td>
<td>16.3</td>
</tr>
<tr>
<td>15/07/09 – 28/07/09 Francis St Pumping station Influent**</td>
<td>15.4</td>
</tr>
<tr>
<td>15/07/09 – 28/07/09 Tulla Road Pumping Station Influent**</td>
<td>20.0</td>
</tr>
</tbody>
</table>

* Influuent data provided by Clare Co Co from 11/01/07-10/12/08
** Data from Flow and Load survey carried out by ENVA in July 2009

This is likely due to high levels of infiltration in the system causing a dilution effect. As such, for an interim upgrade to the WwTW, the TN standard may be achieved by introducing a ‘denitrification period’ into the existing aeration basins instead of building new anoxic tanks and extending the aeration basins. The required denitrification can be achieved by intermittent aeration, for which mixers are to be installed. Since the aeration is shut off for approximately 10 to 20% of the time, the oxygenation capacity has to be increased accordingly. Aeration and mixing are to be controlled by measurement of ammonia and/or nitrate (preferably both). When a certain concentration threshold is exceeded (for instance, 1 - 2 mg NH₄-N, or 10 - 15 mg NO₃-N) then aeration is put back on again (NH₄-N) or off (NO₃-N). This allows for variable times of the nitrification and denitrification periods.

The contribution of these very high effluent standards – low ammonia (full nitrification), Total Phosphorus and total Nitrogen means that the plant will need to be operated to a very high standard involving close monitoring and process control in order to avoid breaches of the
licence conditions. For example, MLSS levels may need to be increased to 4,000 mg/l in the Winter (from 3,500 mg/l) when the temperature decreases.

To achieve intermittent denitrification as discussed above, a minimum sludge age of 10 days has been selected. Aeration time will be reduced from 24 hours to 16-18 hours depending on the relative concentrations of nitrate and ammonia. During the period of denitrification the aerators switch off and new mixers take over. It is proposed to install two submersible 11kw mixers per basin. It is intended to install combined aerator/mixers.

**Secondary Settlement:**
The activated sludge from the aeration tanks will be split evenly between the existing clarifier on stream 2 and a new 25m diameter clarifier on stream 2. The design overflow rate for the new clarifier is 0.9 m$^3$/m$^2$/d. The existing clarifier will operate at a slightly higher overflow rate of 1.5 m$^3$/m$^2$/d at peak flow. This overflow rate is within the rates of 0.67-1.167 m$^3$/m$^2$/d at average flow and 1.67-2.67 m$^3$/m$^2$/d at peak flow, recommended by Metcalf & Eddy for secondary clarifier design.

A coagulant dose (ferric chloride) will be added upstream of the secondary clarifiers so that a total phosphorus concentration of less than 2 mg/l is achieved at this stage.

**Tertiary Treatment**

Due to the nature of the proposed discharge effluent standards, the provision of tertiary treatment is compulsory to achieve the necessary reduction in BOD, Ammonia and Phosphorus.

The biologically treated water will flow to a series of sand filters. These tertiary filters will be of a proprietary design but it is proposed that a total of 8 no. sand filters (see Figure 1 for generic sand filter schematic) will be provided. It is proposed to install continuously operating filters, c7 metres high with internal washing systems which eliminates the need for wash water storage tanks, backwash pumps and collecting tanks. The filters will be designed such that when operating at maximum throughput, one filter can be taken out for backwashing, while the other filters cater for the peak flow. Secondary treated water from the Clarifiers distributed over the filters and fed into the base of each. The influent flows upward through a downward moving active sand bed where solids are filtered out. Clean filtrate exits the filter through an overflow weir and washwater is discharged through a separate outlet and returned to the head of the works.
Sludge Treatment

The existing sludge handling facilities comprise a sludge holding tank and a sludge dewatering building with a Solids Technology double belt press which has a reported optimum sludge handling rate of 250kg Dry solids per hour. The excess sludge quantity based on a design population of 30,150 PE is estimated at 1,023 kg Dry Solids / day.

With a view to improving the sludge handling capacity of the plant and decreasing the hydraulic loading on the sludge press (and thereby increasing its efficiency) it is recommended that a Picket Fence Thickener (PFT) tank be included upstream of the sludge press. An effective PFT would increase the dry solids concentration coming from the clarification tanks from 1.5% to 3.5% (2.5% DS is taken as a more conservative figure for PFT thickened sludge).

It is proposed that a 134m$^3$ PFT be constructed on-line between the clarifiers and the existing sludge press. The sludge belt press operation time will need to be increased to c40 hours per week to facilitate treatment of sludge from a 30,150 PE plant. Sludge feed pumps are currently undersized for future loads so these shall be upgraded along with a programme of measures for the double belt presses.
4 Hydraulic Gravity Pipeline Design

General

This section of the report reviews the plant hydraulics for Ennis WwTW.

The existing arrangement consists of a balancing tank which accepts flow via 3 nr. rising mains from Francis Street, Tulla Road and the Mart. Flows in excess of F.F.T overflow via 2 nr. weirs to 2 nr. storm tanks. The flow is split evenly between both tanks. The existing inlet screen is housed in a single channel and flows gravitate via 2 nr 300mm pipes from the balancing tank. Following the screen the flow splits into 2 streams. Stream no. 1 is the original phase 1 and consists of an aeration basin and imhoff tanks for secondary settlement. It is proposed to decommission the imhoff tank and construct a new clarifier. Stream no. 2 consists of an aeration basin and an existing clarifier.

The existing plant currently caters for approx. 100 l/s.

The purpose of this hydraulic review is to determine if the existing plant has adequate hydraulic capacity to cater for the proposed F.F.T for 30,150 PE based on 180 l/h/day.

Peak flow to the treatment works is 401 l/s of which 188 l/s is to go forward to full treatment (30,100 PE x 180 l/h/day). An additional 20 l/s has been allowed in the design to cater for supernatant return which will be delivered back to the aeration basin inlets via a supernatant return pumping station.

- The friction headloss calculations are included in this document.
- The fixed point for Stream no. 2 for the plant is the T.W.L of the existing clarifier 7.79m OD Malin.
- The fixed point for stream no. 1 is the maximum TWL that can be selected in the aeration basin (i.e to maintain a minimum of 500mm freeboard).
- The 1: 100 year flood level is 3.2m OD. The TWL selected in the Final Effluent Outfall Chamber is 4.7m OD. 5.7m will be the top of structure.

This report is to be read in conjunction with the Hydraulic Profile Drawing and the hydraulic calculations, which accompany this report.
Proposed Tertiary Sand Filter to Outfall via existing 80 m long 375mm concrete outfall

The 1:100 year flood level is **3.2m OD Malin**. Hydraulic head of 1.4m over and above the 1:100 year flood level is required to cater for F.F.T of 188 l/s through the existing 375mm outfall pipe.

The maximum TWL in the Final Effluent Chamber is calculated as **4.6m OD** at 1:100 year flood event.

A conservative value of Roughness Coefficient has been selected (3mm) based on slime build up of not more than 6mm in mature pipes, as classified in Hydraulics Research Paper No. 2 (3rd Edition).

The condition of the existing outfall pipe is unknown at present.

It is recommended that the existing outfall pipe is cleaned with high pressure jetting equipment to remove silt and slime build-up. A CCTV survey should be carried out to determine the condition of the pipeline.

**Note:** The roughness value is a theoretical value only. Hydraulic tests should be carried out on-site to determine the actual roughness value of existing pipework, wherever possible.

1.1

STREAM 2 - Existing Clarifier to Proposed Tertiary Sand Filter Pump Station via Ex. MH

STREAM 2 - TWL in Existing Clarifier

The TWL level in the existing clarifier is **7.79m OD Malin**. The existing overflow arrangement is via a weir around the circumference of the clarifier. The rise in TWL over the weir based on the increased flowrates is insignificant due to the length of the existing overflow weir. Therefore **7.79m OD** is assumed as the proposed TWL in the existing clarifier.

STREAM 2 - Existing Clarifier to Ex. MH

The existing storm outfall pipe from Stream no. 2 connects with the existing 300mm effluent pipe from the clarifier at Ex. MH and discharges via a single 375mm outfall.
It is proposed to separate these lines and install a new storm pipe that will divert storm flows from Stream no. 2 into a new combined 500mm storm line.

It is proposed to install a new 400 mm outfall pipe from the Ex. Clarifier to the Ex. MH chamber.

**STREAM 2 - Ex. MH to proposed Sand Filter Pump Station**

Flows will gravitate to the proposed Sand Filter Pump Station via the existing 375mm concrete pipe. This pipe will be intercepted to divert flows to the pump station which will be positioned below ground. The TWL will be approximately 1.5m below existing ground level. The bottom of the chamber will be 3m below ground level.

The proposed pumps will deliver F.F.T to the proprietary Sand Filtration units. Manufacturer details are to be provided as part of the contract procurement stage.

It is envisaged that the sand filters will be continuous up-ward flow type and positioned at ground level (approx. 6.5m high) with F.F.T delivered via suitably sized pumps. The final effluent outlet will be approx. 5.5m above ground level.

T.W.L in the Sand Filtration units is to be confirmed by manufacturer of the proprietary sand filtration unit.

**STREAM 2 - TWL in Aeration Basin No. 2**

The existing 300mm that transfers flows from aeration basin no. 2 to the clarifier is to be retained. Due to the increased proposed flow it will be required to increase the TWL in the Aeration Basin no. 2 by 500mm in order to meet the hydraulic requirements. The proposed TWL in Aeration Basin no. 2 is 8.79m OD. The existing top of chamber is 9.29m OD (freeboard of 500mm). The TWL will be increased by raising the existing weir.

The existing weir effective working length is 4.6 m.

The water depth over the weir can be calculated using the following equation;

\[
Q = \frac{2}{3} 2g C b h^{1.5}
\]

BS 3680 : Part 4a : 1981

\[
H = \left(\frac{Q}{(2/3*\sqrt{2g*C*b})}\right)^{2/3}
\]
Q = Flowrate = 0.104 m$^3$/s
C = Discharge Coefficient = 0.611
b = Effective weir width = 4.6 m
H = water depth over weir (m)

Water depth over the weir at peak flows **0.054m**

Freefall of 150mm following overflow from the weir has been allowed for in the calculations.

**STREAM 1 - TWL in Aeration Basin No. 1 and Proposed Secondary Settlement Tank**

Due to the increased future flow it will be required to increase the TWL in the Aeration Basin no. 1 by 500mm to provide sufficient hydraulic head to **8.77m OD**. The TWL will be increased by raising the existing weir. The flows will gravitate to the proposed secondary settlement tank via a new 400mm pipe. The TWL selected in the proposed settlement tank is **7.54m OD**. Effluent will gravitate via a new 400mm pipe to the proposed Sand Filter Pump Station.

The existing weir effective working length is 6 m.

The water depth over the weir can be calculated using the following equation;

\[
Q = \frac{2}{3} \frac{2g \times C \times b \times h^{1.5}}{\sqrt{2g \times C \times b}}
\]

\[
H = \left(\frac{Q}{2/3\times\sqrt{2g\times C\times b}}\right)^{2/3}
\]

Q = Flowrate = 0.104 m$^3$/s
b = Weir width = 6 m
H = water depth over weir (m)
C = Coefficient of Discharge = 0.611

Water depth over the weir at peak flows is **0.045m**.

Freefall of 200mm following overflow from the weir has been allowed for in the calculations.
STREAM 1 and 2 - Existing Inlet Works and Inlet Flume

Since the TWL in the existing aeration basin will be raised by 500mm to transfer the proposed flowrate to the clarifier, the TWL at the outlet of the existing flume will exceed the invert level of the flume chamber. As a result the flume will not give a correct reading and will not be suitable to meet future flow requirements. It is proposed to remove the existing flumes and install electromagnetic flowmeters on the existing 375mm pipes to the Aeration Basins 1 and 2.

It is also proposed to remove the existing open channel flow and pipe directly from the balancing outlet chamber to connect to the existing 375mm concrete inlet pipe to the aeration basins.

Actuated penstocks must be installed in this chamber to throttle the flow and control F.F.T to the aeration basins.

STREAM 1 and 2 - Inlet Works to Aeration tanks

The inlet to the Aeration Basins no. 1 and 2 will be via the existing 375mm concrete pipework. This existing pipework is sufficient to cater for the proposed flow.

STREAM 1 and 2 - Overflow Weir in Balancing Tank and Storm Tank

Stormtank to Storm Overflow Chamber

The water depth over the Stormtank Overflow Weirs is 0.072m. The existing weir level in the Stormtanks is 11.08m OD. The TWL in the Stormtank is calculated as 11.15m OD.

Balancing Tank to Stormtank

Overflow to the 2 nr. Stormtanks takes place in the Balancing Tank. Flows in excess of F.F.T will overflow via 2 nr. weirs in the Balancing Tank.

The water depth over the weir can be calculated using the following equation;

\[ Q = \frac{2}{3} \times 2g \times C \times b \times h^{1.5} \]

BS 3680 : Part 4a : 1981

\[ H = \left( \frac{Q}{(2/3 \times \text{Sqrt}(2g \times C \times b))^{2/3}} \right) \]

\[ Q = \text{Flowrate} = 0.104 \text{ m}^3/\text{s} \]
C = Discharge Coefficient = 0.611
B = Weir width = 2.5m

Water depth over the weir at peak flows is 0.081m.

Headloss calculations in the pipeline connecting the balancing tank to the storm tank via the existing 300mm pipe results in a reduction of the storm tank volume and a lowering of the TWL below the I.L of the storm overflow weir to the outfall. To maintain the existing TWL in the Stormtank it is proposed to increase the existing 300mm pipe to a new 400mm pipe. The friction losses in this pipeline are estimated at 0.15m. A freefall allowance of 0.15m is included. The water depth over the weir is calculated as 0.081m. The proposed TWL in the balancing tank is 11.46m OD. The existing weir must be raised by 150mm to account for increased hydraulic head over the weir in the Stormtanks.

**Storm Tank to Balancing Tank**

The existing arrangement consists of a 225mm pipe which gravitates from the invert of the Stormtank to the Balancing Tank. This pipe is sufficient to cater for over 1xDWF return during periods of low flow based on the hydraulic head available in the Stormtank. As stated previously, it is proposed to separate the effluent from the existing clarifier and the storm effluent pipe.

**Other Pipelines**

The Head Loss calculations for pumped rising mains have not been assessed in this report. Pumped pipelines will include sludge pumping from clarifiers to the proposed picket fence thickener (PFT), proposed PFT to sludge dewatering unit, supernatant return to the aeration basins and influent pump station to sand filter.
Appendix 1  Hydraulic Design Calculations
**Gravity Pipelines Flow**

<table>
<thead>
<tr>
<th>Stream 1 &amp; 2 - Balancing Tank to Outlet</th>
<th>Stream 1 - Existing Clarifier Stream</th>
<th>Stream 2 - Proposed Secondary Settlement Tank Stream</th>
<th>Stream 3 - Proposed Secondary Settlement Tank Stream</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PIPELINE REFERENCE</strong></td>
<td><strong>PIPELINE REFERENCE</strong></td>
<td><strong>PIPELINE REFERENCE</strong></td>
<td><strong>PIPELINE REFERENCE</strong></td>
</tr>
<tr>
<td><strong>FLUID DETAILS</strong></td>
<td><strong>FLUID DETAILS</strong></td>
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</tr>
<tr>
<td>Fluid</td>
<td>Screened wastewater</td>
<td>Activated Sludge wastewater</td>
<td>Wastewater</td>
</tr>
<tr>
<td><strong>FLOWRATE DETAILS</strong></td>
<td><strong>FLOWRATE DETAILS</strong></td>
<td><strong>FLOWRATE DETAILS</strong></td>
<td><strong>FLOWRATE DETAILS</strong></td>
</tr>
<tr>
<td>Fluid Velocity</td>
<td>1.37 m/s</td>
<td>0.94 m/s</td>
<td>0.83 m/s</td>
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<tr>
<td><strong>PIPELINE DETAILS</strong></td>
<td><strong>PIPELINE DETAILS</strong></td>
<td><strong>PIPELINE DETAILS</strong></td>
<td><strong>PIPELINE DETAILS</strong></td>
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<tr>
<td>Inside Diameter (2 nr.)</td>
<td>300 mm</td>
<td>375 mm</td>
<td>400 mm</td>
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<tr>
<td><strong>PIPEFITTINGS</strong></td>
<td><strong>PIPEFITTINGS</strong></td>
<td><strong>PIPEFITTINGS</strong></td>
<td><strong>PIPEFITTINGS</strong></td>
</tr>
<tr>
<td>Total K-value of Fittings</td>
<td>2.00</td>
<td>6.40</td>
<td>5.00</td>
</tr>
<tr>
<td>Pipe Roughness</td>
<td>0.30 mm</td>
<td>0.60 mm</td>
<td>0.30 mm</td>
</tr>
<tr>
<td><strong>SIMPLIFIED COLEBROOK-WHITE</strong></td>
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<td><strong>SIMPLIFIED COLEBROOK-WHITE</strong></td>
<td><strong>SIMPLIFIED COLEBROOK-WHITE</strong></td>
</tr>
<tr>
<td>Friction Factor</td>
<td>2.07E-02</td>
<td>2.30E-02</td>
<td>1.98E-02</td>
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<tr>
<td>Pipeline Friction Losses</td>
<td>0.010 m</td>
<td>0.125 m</td>
<td>6.92E-02 m/m</td>
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</tbody>
</table>

**DUTY REQUIREMENTS**

<table>
<thead>
<tr>
<th>V-Notch Weir Calculation</th>
<th>Effectual weir width</th>
<th>Freefall allowance after weir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Where ( H = \left( \frac{Q}{0.666 \times \sqrt{2g \times C_{ex} \times W}} \right)^{0.667} )</td>
<td>Effective weir width</td>
<td>Water Depth Headloss over weir (( H ))(mm)</td>
</tr>
<tr>
<td>Existing Ground Level</td>
<td>8.57</td>
<td>8.00</td>
</tr>
<tr>
<td>Existing Top of Chamber</td>
<td>9.14</td>
<td>9.27</td>
</tr>
<tr>
<td>Existing T.W.L</td>
<td>8.55</td>
<td>8.27</td>
</tr>
<tr>
<td>Existing Top of MH</td>
<td>3.00</td>
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</table>

<table>
<thead>
<tr>
<th>Required T.W.L</th>
<th>Proposed Top of Chamber</th>
<th>Proposed T.W.L</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.18</td>
<td>9.14</td>
<td>6.00</td>
</tr>
</tbody>
</table>

**Schedule of Changes**

1. Combine Storm Tank Stream No. 1 Overflow with Stream No. 2
2. Upgrade existing 300mm DUCTILE IRRON pipe to 400mm
3. Use 400mm Ductile Iron pipe as pipework as 375mm Ductile Iron pipe not available
4. weir widths reduced to effective working widths, increase of T.W.L above weir.
5. Raise weir level in Balancing Tank by 150mm. Freeboard 250mm
### Proposed Works

<table>
<thead>
<tr>
<th>Gravity Pipeline Flow</th>
<th>Existing pipeline arrangement Design</th>
<th>Proposed existing pipe with 375mm pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chamber</td>
<td>Balancing tank to Stormtank-Ex. Design Check</td>
<td>Balancing tank to Stormtank</td>
</tr>
<tr>
<td><strong>PIPEDLINE REFERENCE</strong></td>
<td><strong>Design</strong></td>
<td><strong>Exists</strong></td>
</tr>
<tr>
<td>Storm Tank TWL to Overflow MH adjacent to tank</td>
<td>Storm Tank S1 Overflow MH chamber to Stormtank S2 Overflow MH chamber combined to Outfall</td>
<td></td>
</tr>
</tbody>
</table>

#### Fluid Details
- **Fluid** wastewater wastewater wastewater wastewater wastewater wastewater wastewater
- **Fluid Density** 1.000E+03 kg/cu.m 1.000E+03 kg/cu.m 1.000E+03 kg/cu.m 1.000E+03 kg/cu.m 1.000E+03 kg/cu.m 1.000E+03 kg/cu.m
- **Kinematic Viscosity** 1.300E-06 sq.m/s 1.300E-06 sq.m/s 1.300E-06 sq.m/s 1.300E-06 sq.m/s 1.300E-06 sq.m/s 1.300E-06 sq.m/s
- **Reynold's Number** 3.36E+05 2.52E+05 1.49E+05 2.52E+05 2.52E+05 4.04E+05
- **Flow Type** Turbulent Flow Turbulent Flow Turbulent Flow Turbulent Flow Turbulent Flow Turbulent Flow

#### Flowrate Details
- **Design Flowrate** 103.0 l/s 103.0 l/s 34.3 l/s 103.0 l/s 103.0 l/s 206.0 l/s
- **Ratio of Pump Flow to Average Fluid Velocity** 1.46 m/s 0.82 m/s 0.86 m/s 0.82 m/s 0.82 m/s 1.05 m/s

#### Pipeline Details
- **Pipe Material** ductile iron ductile iron Concrete Concrete Concrete Concrete
- **Inside Diameter (2 nr.)** 300 mm 400 mm 225 mm 400 mm 400 mm 500 mm
- **Pipeline Length (m)** 6.00 6.00 5.00 5.00 10.00 150.00

#### Pipe Fittings
- **Velocity Head** 1.08E-01 m 3.42E-02 m 3.80E-02 m 3.42E-02 m 3.42E-02 m 5.61E-02 m
- **Total K-Value Of Fittings** 5.00 4.00 2.00 2.00 2.00 5.00
- **Headloss** 0.541 m 0.137 m 0.076 m 0.068 m 0.068 m 0.281 m
- **SIMPLIFIED COBLEBROOK-WHITE**
  - **Pipe Roughness** 0.30 mm 0.30 mm 0.30 mm 0.30 mm 0.30 mm 0.30 mm
  - **Friction Factor** 2.06E-02 1.98E-02 2.28E-02 1.98E-02 1.98E-02 1.86E-02
  - **Hydraulic Gradient** 7.44E-03 m/m 1.70E-03 m/m 3.85E-03 m/m 1.70E-03 m/m 1.70E-03 m/m 2.08E-03 m/m
  - **Pipeline Friction Losses** 0.045 m 0.010 m 0.019 m 0.008 m 0.017 m 0.312 m
  - **Friction Headloss** 0.59 m 0.15 m 0.10 m 0.08 m 0.09 m 0.59 m
  - **Freefall allowance after weir** 0.15 m 0.15 m 0.15 m 0.15 m 0.15 m 0.15 m
  - **Total Head Required** 1.00 m 1.20 m 1.16 m 1.08 m 1.08 m 1.54 m
  - **Existing Ground Level** 8.57 m 8.57 m 8.57 m 8.57 m 8.57 m 8.57 m
  - **Existing Top of Chamber** 11.79 m 11.79 m 11.79 m 11.79 m 11.79 m 11.79 m
  - **Existing TWL in Balancing Tank** 11.53 m 11.53 m 11.15 m 11.15 m 11.15 m
  - **Existing weir level in Storm Tank** 11.08 m 11.08 m 11.08 m 11.08 m 11.08 m
  - **Proposed Top of Chamber** 11.79 m 11.79 m 11.79 m 11.79 m 11.79 m

The above calculations are based on 50% of the total storm overflow from Stream No. 1. Stream No. 2 has similar TWL.
Appendix 2  DRAWINGS