SECTION C2: OUTFALL DESIGN & CONSTRUCTION

A description of the primary outfall design is included. Further details can be seen in the Drawings at Attachment B3 (Location of Primary Discharge Point) of this Application.

Details in respect of the secondary discharge points can be seen in the Drawings at Attachment B4 (Location of Secondary Discharge Points) of this Application.

Details relating to the Storm Water Overflows can be seen at Attachment B5 (Location of Storm Water Overflows) of this Application.

Halcrow Group Limited

Carrigrenan WWTW Outfall

Design Report

April 2002

Van Oord ACZ

Halcrow

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Carrigrenan WWTW Outfalls
Design Report

Contents Amendment Record

This report has been issued and amended as follows:

Issue	Revision	Description	Date	Signed
1	0	First Issue	24 Aug 01	F Budge
1	1	Revised	3 Sept 01	R Elvery
1	2	Revised	7 Sept 01	R Elvery
1	3	Final	22 Feb 02	R Elvery
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Contents

1 Introduction

1.1 *Introduction*

The works comprise of the design, construction, installation, testing and commissioning of a long sea outfall discharging secondary treated effluent from the proposed Carrigrenan Wastewater Treatment Works (WWTW) into the River Lee Estuary at Cork in the Republic of Ireland. These works are part of a joint venture to construct new treatment facilities at Carrigrenan, Cork.

The proposed 1600 mm OD (1480mm ID) outfall pipe and diffuser, totalling 887m in length, will discharge effluent by gravity up to a maximum flow of 4160 l/s against a maximum 1 in 50 year tide level.

Treated effluent flow will gravitate overland from the WWTW to the foreshore through a pipeline to be installed by the WWTW Contractor and connected to the sea outfall. The starting point for the sea outfall pipe is taken as the point on the foreshore 177032E, 70507N.

This design report has been prepared for Van Oord ACZ in accordance with the Employer's requirements.

The design for this project is based on achieving a minimum design life of 60 years for the outfall civil works, but excludes associated existing pipelines, fittings and structures. The design of the works will generally follow the recommendations of the WRc Design Guide for Marine Treatment Schemes Volumes I, II, III, IV of August 1990 and is in accordance with relevant International, National or other appropriate standards, as well as the Employer's specified requirements.

2 Outfall Pipeline and Diffuser

2.1 *Pipeline Profile*

The Tender Documents (Volume 4, Employer's Requirements, Particular Requirements for Design, Section 13.2.8) require the outfall to be constructed to a fixed horizontal alignment. It is to be placed within a trench with a minimum cover from the existing seabed level to the crown of the pipe of 2.3 metres between chainages 260 to 620 metres and armoured with a layer of stone not less than 1 metre thick. The main outfall pipe profile shall also have a non-rising gradient throughout its length.

The design involves an 800 metre long, 1600mm outside diameter, polyethylene (MDPE/PE80) SDR 26 pipeline with a wall thickness of 60mm leading to a 20m long 1524mm OD steel pipe, 45^{0} horizontal bend and steel diffuser section. The PE pipeline will be manufactured and towed to site by sea in two sections each of around 400m in length. A system of continuous precast concrete weight collars will be attached to the pipe to provide stability and protection. The pipe string will be connected using proprietary Viking Johnson Aquagrip couplings at the mid-section joint and at each end of the PE assembly.

2.2 *Design Criteria*

The design of the pipeline has assumed that the sewerage system upstream of the outfall has been designed in accordance with best practice to the latest design standards. The maximum flow figure for the design is 4160 l/s as supplied by the Employer for the treatment process.

The tidal water levels in the River Lee mouth in metres relative to OD Malin Head (MH) are as follows. These levels are derived from information provided in the Tender Documentation (MLWS, MHWS and maximum recorded level) and from reference to Admiralty Charts and Tables.

LAT m ODMH	-2.38
MLWS m ODMH	-1.88
MLWN m ODMH	-0.98
MHWN m ODMH	+0.92
MHWS m ODMH	+1.82
HAT m ODMH	+2.22
1 IN 50 year max sea level m ODMH	+2.50

2.3 *Seabed Material*

The Marine Geotechnical Investigations conducted by Norwest Holst for Carrigrenan Outfall indicates that the geology of the area is Limestone bedrock, overlain primarily with gravels and sands and isolated patches of clay and secondarily with soft alluvial silts.

The bed profile is generally flat, falling gently to the main channel of the River Lee. The surface layer of alluvial silt is of varying thickness up to 13m in places along the pipeline route, though the silt thickness is much less close inshore and offshore in the area of the diffuser where there is a thinner silty layer overlaying the sands and gravels.

The site investigation gives borehole data along the proposed line of the outfall. The laboratory analyses undertaken by Norwest Holst on the many samples of silty material taken from the boreholes shows that the bulk density is in the order of 1.8 Mg/m^3.

2.4 *PE Pipeline Stability and Strength*

The main length of the outfall pipeline will be constructed from 1600mm outside diameter MDPE (PE80) fitted with continuous precast concrete weight/armour collars. Polyethylene is a flexible material and this property is utilised in the design of a pipeline that is to be installed within and supported by the soft alluvial silt material. The design principle will be that of 'neutral buoyancy', whereby the pipeline and weight/armour collar assembly when full of water will have a density of 1.4Mg/m^3, which is less than the alluvial material in which it will be installed and there is no requirement for bedding nor risk of flotation. There may be some small localised settlement later due to the additional weight of the granular surround armour stone, but no significant accumulation of stresses in the pipeline, as these would be absorbed by the flexibility of the MDPE. This type of construction arrangement has been successfully employed on previous projects under similar circumstances involving deep soft alluvial silts.

The effect of settlement on the pipeline due to the increased loading on the silt via the backfilling with dredged sand granular surround and the rock armour has been considered. Assuming an increase in load of between 5 and 20 kN/m^2, a 9m thickness of cohesive alluvial clay, and using values of compressibility modulus for soft alluvial clays, settlements in the order of 90 to 360mm are predicted. The borehole logs suggest alluvium is not organic and is quite sandy, suggesting the lower half of this range. It is therefore estimated that the maximum settlement of the pipeline due to the weight of the pipeline, backfill and rock armour will be in the region of 100 to 200mm.

Settlement would cause stress in the pipeline though the values that could occur in practice would be very small. A calculation for what is considered to be an extreme scenario, whereby a 100m long section of pipe settles by 300mm, shows the resulting stress in the PE pipeline at just 0.15N/mm^2.

PE is corrosion resistant and is very suitable for the float and sink method of installation. The pipeline and weight collar assembly when full of air will have a positive buoyancy in seawater and would float at around three-quarters submergence.

The Contract Documentation Volume 4, Particular Requirements for Design, Clause 13.4.11 states that PE pipe should have a designation PE100. However the MDPE pipe proposed has a designation PE 80, due to the fact that according to manufacturer advice only PE80 is available with a good track record in this very large diameter. (It has been reported very recently that PE100 has been developed but is as yet untried). The main difference between these designations is PE100 is a stronger material, with a minimum residual strength after 50 years (MRS) of 10 N/mm^2, whereas PE 80 has a MRS of 8 N/mm^2. Furthermore SDR26 of PE100 designation would have nominal pressure rating of 6 bar, whereas SDR 26 PE80 is rated at 4 bar.

In this particular application the PE80 pipeline in its permanent installed state would be stressed at well below its rated capability. The maximum hydraulic working pressure is just 0.5 bar and the maximum stress due to settlement as stated above is insignificant. The highest stress situations would occur during towing and sinking in the construction phase, though the maximum allowable stress in the short term is 13 N/mm^2, equating to an allowable end pull of up to 400 tonnes and would not be critical.

The weight/armour collars will be constructed in reinforced concrete grade C50 and fitted continuous along the length of the outfall. The collars will be constructed in 1200mm lengths and fitted to the pipe at a later stage. Rubber spacers will allow flexibility in the laying of the pipeline and the annulus in the collars will be offset to allow accurate positioning. The design of the collars is to

BS 8110 (1997): Structural use of Concrete and BS 6349: Code of Practice for Maritime Structures. Each collar will have a lifting lug designed for lifting the self weight of the collar in air.

The design calls for the pipeline to be buried in a trench throughout its length and protected by rock armour. A stability analysis has been carried out to ensure the outfall has sufficient weight to resist wave, current and buoyancy generated forces.

From analysis using wind data the maximum wave height has been calculated at 0.9m with a period of 2.9 seconds. The maximum recorded tidal currents in the area do not exceed 1 m/s at the surface; currents at depth would be unlikely to exceed a maximum of 0.6 m/s.

These maximum figures for wave and current are low reflecting the sheltered location of the site. Local historic knowledge also supports the view of a stable sheltered location. However the effects of higher current velocities due to the action of ships propellers on the stability of the diffuser and the anti-scour rock protection are examined in the calculations.

After installation the trench will be backfilled with selected as-dredged natural marine sand or gravel as surround to the pipeline to a thickness 500mm above the pipe, over which graded 50 to 250mm stone armour will be placed in a layer of minimum 1 metre thickness. The remainder of the dredged trench to original seabed level will backfill by natural siltation. Bunds formed from the 50 to 250mm graded stone will be placed across the seaward end of the trench in order to limit the loose surface material migrating into the diffuser pocket.

3 Diffuser Arrangement and Hydraulics

3.1 *Diffuser Arrangement*

The diffuser position, length and level are in accordance with the mandatory requirements stated in the Tender Documentation. The diffuser consists of a tapered main pipe, the first section at invert level –12.7m ODMH and with a total of 22 nos. 300mm long diffuser ports, all fabricated in steel to Standard API 5L Grade B. This number of diffuser ports is sufficient to achieve the required minimum initial dilution factor of 20 for the effluent in the available depth of water at flow 4160 l/s and tide at MLWS. To increase dispersion potential the ports will be fixed to the main pipe in alternately staggered 2 o'clock and 10 o'clock positions. To ensure satisfactory flow distribution the 12 inshore ports will be 302mm ID and the 10 offshore ports will be 333mm internal diameter. Each port will terminate in a flange to allow the fitting of temporary blanking plates.

Precast concrete anchor block sections will be provided to support the diffuser pipe at nominal 10m intervals. These sections will weigh approximately 15.5 tonnes each and will rest on a prepared bed and be provided with rock armour/scour protection of proprietary Armorflex type 300mm thick submat flexiform mattresses. No significant settlement is expected at the diffuser location as the borehole logs indicate only sand and gravel below formation level and no soft alluvial silt. This construction will provide adequate anchorage and stability for the diffuser against all natural wave and currents action and also higher propeller generated currents of up to 2m/s at the diffuser depth.

The steel pipe, fittings and diffuser assembly will be protected from corrosion with a 1.5mm thickness polyurethane internal and external coating (Durathene P). Additional protection for the full 60 year design life will be provided by a sacrificial anode cathodic protection system designed to Standard DNV RP B401 (1993) for a minimum 60 year design life. This equates to approximately 1400 kg of aluminium alloy anode material. The report regarding the cathodic protection system undertaken by Corrpro Companies Europe Limited is included within Appendix E and concludes that 6 annodes of 240kg will give the required anode material to meet the contract requirements. Also, included within Appendix E is a statement from Corrpro confirming the electrochemical capacity of the anodes and certificates from DNV, which supports the design within the Corrpro report. However, some concern was expressed with regards to the electrochemical capacity of the anodes and therefore, it is proposed that 8 annodes of 240kg are employed. These two additional anodes are extra and above the contract requirement. Therefore, the proposed anode arrangement to be used is that shown on drawing AM 5507 included within Appendix E.

The diffuser section will consist of approximately 12m long sections with flanged bolted joints. The connection of the diffuser to the pipeline will be via a 45^0 steel bend using tied Teekay couplings as shown in Detail 3 on drawing WECROF 102. The connection between the steel and PE pipes will be by means of an Aquagrip flanged coupling.

A duck billed check valve system at the diffuser head has been considered but will not be employed, as there is not sufficient operational head available. The diffuser will be raised well above the seabed and provided with scour protection stone in order to reduce the risk of sediment entering at the seaward end. Given the continual flow of secondary treated effluent from the outfall and the small particle size of the surrounding seabed material, the risk of significant deposition within the outfall is low under normal operation.

Saline intrusion can occur in theory when the Densimetric Froude number at the diffuser ports falls below unity. For the proposed diffuser arrangement this situation will occur at flows below 550l/s. The provision of a regular system for flushing the outfall can negate the adverse effects of saline intrusion. The WRc Design Guide for Marine Treatment Schemes, Vol II page 213, suggests that the situation will be satisfactory with a flushing system providing velocities in excess of 1m/s for a period of at least 15 minutes once a week.

3.2 *Diffuser Pocket*

The requirement is for the diffuser to sit within a dredged pocket with a bottom depth of –13.2m ODMH. It is noted in the documentation (Section 13.4.7) that at some time in the future dredging to a depth of –13.2 metres will take place in the area adjacent to the diffuser to form deeper shipping channels and a turning circle.

The present seabed level at the diffuser location is around –8.0m ODMH, resulting in a dredged pocket of more than 5 metres in depth. It is inevitable that this pocket will tend to silt up, but it is not possible to predict the rate at which siltation would take place. HRC Wallingford have carried out a detailed study on the silt transportation, but provide no useful conclusions. Siltation should be less rapid once the outfall is commissioned, as the flow will tend to scour the silt, but it is still likely that maintenance de-silting will be necessary. A regular inspection programme carried out by the Contractor may determine the siltation rate. It is essential that the diffuser ports be blanked off prior to commissioning to prevent silt ingress into the pipe.

If and when the additional dredging to form the proposed vessel-swinging basin takes place, the siltation problem should be reduced. However, the diffuser will then be exposed to a greater risk of damage from shipping and/or dredging activities, as is implied in section 13.4.7. It is not possible for Halcrow as designers to quantify this risk, though calculations are being undertaken to illustrate possible loading cases and a statement is included here in Appendix C.

Due to the unconventional position of the diffuser (below existing bed levels) no case studies or background information was available. The size of the pocket was therefore determined based on the following constraints:

- Proximity to the future turning circle
- Side slope gradient
- Concrete scour blanket dimensions

Taking the above in to consideration the optimum solution was determined with a pocket of 10.0m width at the base and side slopes at 1:5.

With regards to the potential for scour at the edge of the scour mattress and possible destabilization of the trestles, a full statement is included within Appendix D. The diffuser position is unconventional and therefore no appropriate theories are available to calculate scour and to determine the optimum width of scour protection. For unconventional designs, such as diffusers in pockets, standard formulae cannot be relied upon, and therefore need to employ engineering judgement. Due to concern that were raised regarding the width of scour protection, the diffuser pocket has been increased to 13.0m width at the base with the side slopes remaining at 1:5. This is over and above the contract requirements.

3.3 *Hydraulics*

Head loss calculations for the pipeline have been carried out based on the Colebrook-White equation in order to determine the appropriate pipe diameter and head loss for gravity flow against a 1 in 50 year maximum sea level of +2.50mODMH. The table below summarises the inputs and results.

PE PIPE OUTSIDE DIAMETER	1600 mm
PE PIPE INSIDE DIAMETER	1480mm
MAXIMUM EFFLUENT FLOW	4160 l/s
ROUGHNESS OF PIPE (Ks)	3.0 mm
VELOCITY	2.42 m/s
MAX TIDE LEVEL 1 in 50 YEAR m ODMH	2.50m
MAXIMUM ALLOWABLE HEAD AT LANDFALL CHAINAGE 0.0m	7.56m ODMH

Outputs from a numerical hydraulic model DIFFUSERV for the outfall and diffuser arrangement are included in the hydraulic calculations.

Sheet 1 shows the situation at maximum flow 4160 l/s and 1 in 50 year maximum sea level, showing a maximum operational head requirement of 7.56m ODMH.

Sheet 2 shows the situation at maximum flow and MLWS, with dilution above the minimum requirement of 20 at all diffuser ports (as clause 13.2.8.f). Initial dilution is calculated using the WRc Lee's formula, buoyancy dominated near field case, ambient current velocity at zero.

Sheet 3 shows the situation for maximum flow and at mean sea level (MSL) and demonstrates that the flow distribution through all diffuser ports is within 90-110% of the average port flow as required in clause 13.4.5.

Sheet 4 shows the minimum flow, 550 l/s, at which the Densimetric Froude Number at all diffuser outlets remains above unity (clause 13.4.4.b). ok

Sheet 5 shows the situation at dry weather flow (DWF) 687 l/s and sea state MSL, with flow distribution between all diffuser ports within the range 90-110% of average port flow.

Sheet 6 shows the situation at peak daily flow (PDF) 1330 l/s and sea state MSL, with flow distribution between all diffuser ports within the range 90-110% of average port flow.

The detailed numerical model analysis has demonstrated that the operation at low flow volumes is much better with the diffuser port outlets set at a similar level, as compared to ports set at progressively lower levels to seaward as would occur in a tapering diffuser section with a level invert. The difference is caused by the differential density between the effluent and seawater, the result is that a tapered diffuser section with level soffit would offer much better performance and less risk of saline intrusion than one with level invert in this application. This difference is illustrated in sheets 7 & 8.

Sheet 7 shows a diffuser arrangement with the invert level set at –12.7mODMH throughout its length, with the outlet port levels become progressively lower in level as the main pipe tapers. At a flow rate of 850 l/s the discharge through the first port is almost double that through the last port, where the Froude number is 1.0.

Sheet 8 shows the same level invert configuration at flow rate 635 l/s, at which outflow through the last port has virtually ceased, with a Froude number of just 0.1. At flow rates lower than this inflow and saline intrusion would occur.

The landward intertidal section of the outfall pipe, between chainage 0 to 90m, is purposefully set at a steep gradient in order to eliminate the risk of air choking on a sudden start up at low tide. In this situation the full flow of 4160 l/s would be accommodated with the pipe flowing at no more than half bore, allowing sufficient space for air to travel back up the pipe against the effluent flow.

4 Drawings

4.1 *List of Drawings*

APPENDIX A

GEOTECHNICAL STATEMENT

Carrigrenan WWTW Outfall

Geotechnical Statement August 2001

Introduction

It is proposed to construct an 886m long outfall into the River Lee Estuary at Cork in the Republic of Ireland. The outfall will comprise a 1600mm OD MDPE pipe with continuous precast weight/armour collars. The Estuary bed level along the route varies between -1.14m OD and –7.79m OD. The outfall invert falls gradually from approximately 0m to –13m OD, requiring an excavation depth of typically 2 to 3m over the initial 200m from the shore, increasing to 3 to 5m thereafter.

Geology

The solid geology in the area generally comprises Devonian Sandstones and Carboniferous Limestones, the latter sometimes exhibiting karst features. The solid geology is steeply dipping and significantly faulted beneath parts of the estuary, resulting in highly variable depths to bedrock.

The drift geology comprises boulder clay beneath the foreshore. Beneath the estuary the deepest drift soils comprise river sands and gravels, frequently with coarse cobbles and boulders, overlain by alluvial deposits including loose to medium dense sands, loose and soft sandy clayey silts and soft and very soft silty clays. The alluvial deposits may be slightly organic, particularly at shallow depth.

Site Investigation

A geotechnical investigation was carried out along the route of the outfall by Norwest Holst Soil engineering Ltd. in February 1999, report reference F11270, comprising ten cable tool boreholes carried out from a jack up rig and an eleventh land based borehole. The boreholes reached between 3.05 and 18.3m depth below bed level.

Ground Conditions

The following boreholes (and approximate chainages) are relevant to the outfall:- 122 (0m, offset approximately 30m north); 129A(70m); 129(190m); 129B(335m); 130(435m); 130A(525m); 131(640m); 131A(685m); 132(760)m; 132A(815m, offset approximately 40m south west) and 133(845m). Other than those specifically mentioned the boreholes are within 10m of the outfall centreline.

The landbased borehole 122 encountered claybound gravels to 9m depth over fractured limestone. Non of the other marine boreholes encountered the solid geology to a maximum depth of 18.3m.

Boreholes 129 and 129A beneath the initial approximately 200m of the outfall encountered a very thin layer (less than 0.5m) of soft alluvial silt over medium dense to very dense sand gravels and cobbles.

Beyond approximate chainage 200m, the depth of weak alluvial silts, clays and loose sands increases significantly, reaching 15.25m by borehole 129B at chainage 335m, and remaining reasonably constant at typically 13 to 14m depth up to chainage 700m, before reducing slightly to 10.8m at borehole 132 (760m). These soils are again underlain by medium dense to dense sands gravels and cobbles of river origin.

However, the detailed nature of these weaker alluvial soils appears to vary along the route of the outfall. At some locations (129B, 131, 132A) this stratum appears to be divided into a soft clayey silt overlying a loose silty sand over a deeper layer of soft clay. However, at other locations (130,130A, 131A) this stratum is described as being less obviously subdivided, comprising soft sandy clayey silt, possibly with an increasing sand content at depth. The pattern of these subdivisions is not understood and may be influenced by sampling and/or logging techniques.

Beyond chainage 760m the conditions appear to change again at boreholes 132A and 133. Whilst the depth to the underlying dense sands gravels and cobbles remains reasonably constant at just over 10m, the overlying alluvial soils comprise soft silts to 4 or 5m only, but with predominantly medium dense silty sands between 5m and 10m depth. Again the reason for this is not currently understood, although the increase in density of the sands appears to coincide with less silt content as shown in the grading analyses.

Engineering Considerations

The outfall pipe, effluent and backfill are expected to impart loads broadly similar to the load of the alluvial soils currently in place, depending upon the precise in situ densities. The geotechnical report appears to suggest mean bulk densities within the upper alluvial soils of approximately 18 to 19kN/m3. The calculated loadings from the backfilled outfall trench are expected to be within 5% of the currently existing situation, suggesting an increase in load of no more than $5kN/m^2$.

The compressibility of the weak alluvial soils has been assessed from the consolidation stages of laboratory consolidated triaxial strength tests (no consolidation tests have been seen) and from published literature for normally consolidated alluvial soils. The coefficient of volume compressibility of the more cohesive alluvial soils are considered to have an upper bound value of $2 \times 10^{-3} m^2/kN$, with a lower bound value almost an order of magnitude smaller. Assuming the worst case of a maximum 9m thickness of cohesive alluvium beneath the pipeline, the likely upper bound settlements for the following range of increases in loading are as follows:

Increase in loading kN/m^2	Settlement mm
5	90
10	180
20	360

The settlement that the pipeline actually experiences will be influenced by a variety of issues including

- changes in loading from existing conditions
- compressibility of the alluvial soils and the distribution of the cohesive and sandy horizons
- disturbance of founding soils during construction
- uniformity of load application and avoidance of overstressing the formation soils.

It is currently understood that the increase in loading should be of the order of $5kN/m^2$; that the founding alluvial soils comprise both weak cohesive and loose granular materials; and that the construction method should preclude excessive disturbance or overstressing of the formation. On this basis the settlement of the outfall pipe is not expected to exceed 100mm to 200mm. The section of the outfall pipe most likely to experience differential settlement due to the compressibility of the founding soils are those sections where ground conditions vary most rapidly, namely between boreholes 129 and 129B, and between boreholes 132 and 132A.

The diffuser pocket side slopes will be formed by dredging within the upper alluvial soils to a slope of 1 vertical to 5 horizontal (approximately 12 degrees). The stability of these permanently submerged slopes has been checked using SLOPE/W software package, and employing conservative strength parameters. A minimum Factor of safety of 2 was achieved.

APPENDIX B

CALCULATIONS

REF: VOLUME 2

APPENDIX B – CALCULATIONS

INDEX

APPENDIX C

SECURITY AND ACCIDENTAL LOAD CASES

CARRIGRENAN WWTP OUTFALL
APPENDIX C
ACCIDENTAL COLLISION DAMAGE

Likelihood of occurrence

A qualitative risk assessment of accidental collision damage at the diffuser can be made by considering

- the range of possible causes of accidental damage,
- the likelihood of occurrence above or in the vicinity of the diffuser.

A non-exhaustive list of possible causes of accidental damage to the diffuser is presented in Table C1. There are three main group categories, namely damage by falling object, grounding, and trailing object. Most of these circumstances apply equally to fishing vessels, merchant vessels and leisure crafts. The likelihood of occurrence of each group of event can be assessed on the basis of:

- Amount of traffic
- Projected increase in traffic
- Origin of traffic (local / UK / Europe / International) & associate H&S legislation and training standards
- Pilot / local authorities / resources available to disseminate information
- Marine accident statistics (Europe / UK / Eire / Cork estuary)
- Weather statistics (fog, strong winds/currents, waves)
- Proximity of traffic
- Nature of traffic activities in vicinity of site

Information on the Port of Cork and traffic in Lough Mahon was obtained from the Port of Cork Company from telephone discussions with Captain M McArthy the Harbour Master and Dave Doolan the Berthing Master. The current amount of traffic through Lough Mahon is an average of 25 large vessels per week. This typically consists of 10 containers (draft 5-7m), 5 tankers (draft 5-6.5m) and 10 cargo vessels (draft 4-8.5m). Vessels are generally registered in Europe (Ireland, UK, France and the Netherlands). Smaller fishing vessels and leisure crafts also use these waters but again are limited in number. The Port of Cork Company controls all the traffic. All the large vessels are assisted by pilots, with 2/3 exemptions. Assistance by tug is also provided when necessary. Fog days represent an average of 15 to 20 days per year. Short fetches restrict wave activity in the estuary. The highest storm wind speeds are from the North West. Surface currents can reach up to 2 to 2.5 knots in the navigation channels. Traffic is not permitted during thick fog or severe weather. The proposed diffuser is located close to a navigation channel, and close to a proposed turning circle. Traffic in the vicinity of the diffuser is therefore transient but could include manoeuvring traffic in the future. The site is not a working area. This information is used in Table C1to assess the likelihood of occurrence of an accident.

Consequences of occurrence

Falling objects: In the event of a vessel sinking on the diffuser, the consequences are likely to be heavy damage along most of the length of the structure. In the case of objects falling overboard onto the diffuser, the impact damage will vary depending on the weight, shape, water depth, drop height above water and location of impact. Damage levels could range from nil (e.g. chain rolls on pipe) to heavy damage/destruction of part of the diffuser. Intermediate levels of damage may involve damage to one or more ports, perforation of the pipe, flexural cracks/snap, etc.

Grounding on diffuser: In the event of a vessel grounding on the diffuser, the damage levels are expected to be significant, both on the diffuser and on the vessel. Damage levels will depend on the size of the vessel (up to 12m draught at high tide, 6m draught at low tide) and location of impact/contact.
Trailing objects: Damage levels resulting from trailing objects could range from minor (e.g. damage to a diffuser port) to significant, with possible dragging and pulling of pipe section, displacement of concrete blocks, etc.

ACCIDENTAL PROPELLER DAMAGE

Damage as a result of propeller jet induced scour and/or loading on the diffuser pipe is considered accidental when vessels accidentally leave the navigation channels and/or turning circle to the areas of the diffuser and offshore end of the bund.

Scour: Assuming bed protection in the diffuser pocket is made of 300mm thick Armourflex type mattresses, no significant disturbance of the diffuser pocket around the supporting concrete blocks is foreseen. The protection layer is unlikely to undergo significant deformation unless propeller jet velocities are sustained (use of full propeller power for a significant duration at close proximity).

Loading on diffuser pipe: The calculations demonstrate that the diffuser pipe is stable under (steady) velocities of up to 2-3 m/s. Generation of velocities such as these at the diffuser would require a large container vessel with 9 to 12m draft manoeuvring at full power on the region of the diffuser. The maximum draft for the existing traffic is for container vessels and ranges between 5 and 7m. It is understood that the proposed container terminal would introduce traffic of vessels with drafts up to 12m. By ensuring that a minimum clearance between the proposed turning circle and the diffuser pocket is maintained at around the existing bed level of –9mODMH, vessels of draft 10m and larger would be unable to accidentally manoeuvre above the diffuser at MHWS. Drafts are used here as an indication only. Loading on the pipe would clearly depend on propeller characteristics.

CONCLUSIONS

The likelihood of accidental damage is very low, but not nil. It may occur at any time during the lifetime of the outfall. Damage levels could range from minor to destruction of the diffuser. Mitigation measures can be implemented to further reduce the risks of impact or propeller induced damage. This could be done in consultation with relevant local authorities.

Possible mitigation measures

- Marker buoy
- Notice to mariners / update relevant Admiralty Charts
- Port Authorities (information role)
- Flexible or weak joints employed between the diffuser pipe sections and the main outfall pipe (thus the pipes could deflect or detach on impact allowing flows from the WWTP to continue with less risk of impediment – in addition subsequent repair could be relatively simple reassembly).
- Minimum clearance, maintain a separation bund with crest level at around –9mODMH between the diffuser pocket and proposed turning circle to prevent larger vessels entering the diffuser pocket, grounding on diffuser and unacceptable propeller jet induced loading on the diffuser pipe.

Hazard	Hazard	Likelihood of occurrence with impact on the diffuser		Damage level
Falling object	**Sinking vessel** Collision resulting in capsize, flooding & sinking • Timing • Poor visibility / fog • Communication system failure / misunderstanding • Radar system failure • Crew tired • Crew drunk Capsize / loss of stability, leading to flooding and sinking • Vessel not seaworthy • Overloading • Weather conditions • Topping up of large equipment on board • Poor distribution of loading • Watertight bulkhead open/not watertight Explosion / fire, leading to flooding and sinking • Air compressor • Overheating (e.g. hydraulic system) • Fuel system fire	Low Low Low Low Low Low Low Low Low Low Low Low Low Low Low	Low Traffic control by Port of Cork Company No traffic in poor weather Sheltered waters Well Charted Low volume of traffic Vessels registered in Europe, traffic mostly from France, Rotterdam & UK i.e. with similar H&S and training standards. Cause will often be a combination of circumstances.	Heavy damage, the diffuser could be destroyed along much of its length.
	Objects lost overboard (anchor, container, barrel, chain, etc.) • Poor loading • Weather conditions • Lifting gear failure • Sea-fastening failure • Vandalism	Low Low Low Low Low	**Low** The diffuser is located close to a navigation channel and is not in a working area. Cause most likely to occur in this group is failure of sea-fastening.	Impact effect could range from nil (e.g. chain rolls on pipe) to heavy damage/destruction of part of the diffuser. Intermediate levels of damage may involve damage to one or more ports, perforation of the pipe, flexural cracks/snap
Grounding	**Loss of control of vessel** • Loss of power • Crew drunk • Crew tired • Extreme sea state • Poor visibility / fog • Pilot assistance: insufficient knowledge of vessel's handling characteristics • Strong tidal currents / winds **Manoeuvring error** • Timing / Tide	Low Low Low Low Low/medium Low Low/medium	**Low** Traffic control by Port of Cork Company Good knowledge of vessels as a result of traffic regularity Well charted No traffic in poor weather Vessels registered in Europe, i.e. with similar H&S and training standards. Proximity of proposed turning circle, with handling of vessels in confined waters, would increase the risk.	Heavy damage, the diffuser could be destroyed along most of its length.
Trailing objects	Fishing nets Anchors Anchor lines / cables, etc.	Low Low Low Low	**Low** The site is not a recognised trawl fishing or working area.	Damage to ports, possible pulling of a pipe section with displacement of the diffuser pipe or concrete support

TABLE C1 - ACCIDENTAL COLLISION DAMAGE RISK ANALYSIS

APPENDIX D
DESIGN CLARIFICATIONS

APPENDIX D

DESIGN CLARIFICATIONS

INDEX

PAGE	TITLE	COMMENTS
(i	Diffuser Pocket	Relating to Section 3.2 of the Design Report
(ii)	Comments on Design	Relating to calculations within WE/CROF/CALCS/011
(v	Diffuser System Design	Relating to calculation within WE/CROF/CALCS/005

Caigrenan Outfall
Diffuser

Introduction

A question has been raised at to whether the position of the trestles supporting the pipeline will, due to their acute angle to the currents, cause such turbulence that scour is encountered at the edge of the scour mattress and could this scour destabilize the trestles in the long term.

Scour

Before calculations are undertaken one must realise that at any location where a hard surface interacts with a soft natural surface, some scour will occur under action of currents. Therefore, some scour must be accepted as being inevitable at the intersection of the mattress / bed. This would also occur if rock armour were used. Scour would occur wherever this interface is located, to some lesser or greater extent. If the mattress were made 20 m wide, some scour would still occur at the edge. The <u>real</u> question is whether the proposed width of mattress will be stable under the anticipated conditions and would the trestles be destabilized due to any scour.

Another important factor to bear in mind is that the spacing of the trestles, at some 12 m centres, will mean that the worst scour will be localised within the areas directly influenced by turbulence caused by the trestles and will not occur along the whole length of the mattress edge.

It would not be unreasonable to estimate that turbulence could occur up to a distance of 4.3m from the trailing edge of the trestle, (this is equal to the length of the trestle). By placing this potential 'footprint' of turbulence onto the plan of the diffuser area it can be seen that small localised areas could be affected by scour outside the existing width of the mattress. However, as stated above, the question is whether this will have an adverse effect on the stability of the trestles.

The location of the diffuser within the dredged pocket will potentially reduce currents, in that for the turbulence to occur, the currents need to travel into the pocket and under/ around the trestles/pipes. The sloping sides abutting the mattress will reduce any effect of localised scour by filling by natural accretion over periods of low current / turbulence. One must realise that the currents will flow in both directions, due to tidal action, so there is potential for sediment to be brought into the diffuser pocket and be deposited, as well as being scoured away. It is not like a situation in a river where the flow is in one direction and long term scour around a bridge pier for example, can be a problem.

The trestles are to be founded on compact sand/gravel that is present at the location. The foundations of the trestles will be protected from damage by the mattress and it is not until a distance of 2.8 m from the outer face of the trestle does the natural bed become exposed to potential scour. This distance, although short, will allow turbulence to 'calm' before it reaches the natural bed.

Very little work has been carried out by academia on potential for scour at the edge of submerged concrete mattresses. Most research has been aimed at the problems associated with waves directly impinging on structures, which, as one would imagine, create the worst potential for scour.

However, generally it is accepted that some minor scour will occur due to the soft/ hard interface that is created and that any settlement will be taken up by the flexible nature of the mattress.

It is considered that should any localised scour occur at this location the mattress will accommodate it locally by flexing and settling into the profile of the bed and that the protecting afforded to the trestles is adequate.

It is therefore considered that any scour at the edge of the mattress will not destabilise the trestles.

Carrigrennan Outfall
Comments on design

Comments in relation to Minutes of Meeting 15th Jan 2002

Discussion on design parameters, point 2,2.1, 2.2

When designing the rock covering for the pipe one must consider the location of the outfall in relation to the potential exposure to wind/wave conditions in the estuary. The original calculations for Carrigrenan are considered robust because:

- The Van der Meer equation used in the design was derived for slopes which are not overtopped. The case of the rock covering the pipe is that the structure is overtopped, and therefore there will be a certain amount of wave energy transmitted over the structure. This will increase the stability of the rock compared to the case where the wave is impacting upon the structure. This will build in conservatism to the design. The calculations do not allow for the pipe being sheltered in the trench. As the crest of the rock protection is at existing bed level, or lower, again this would make the design conservative in relation to the rock size.

- From site inspections one can see that at present very little beach movement appears to occur. The existing beach material appears to have a typical size of between 100 and 150mm. During trench dredging this material will be side-casted for reuse as backfill. In this case therefore no imported rock armour is required to reinstate the foreshore without jeopardizing the beach stability. It is anticipated that the volume of sidecasted material is adequate for backfilling the trench until chainage 260m. From chainage 260m to chainage 780m rock armour will be placed. Details as shown on drawing WECROF.106.

- To obtain an estimation of wind data for the site data taken from Cork Airport was used. Alternative locations such as Roche's Point could have been used but wind levels at this location only differ slightly from the airport. Given the generally sheltered location of the site in relation to both the airport and Roche's point it is felt that the wind parameters used are acceptable. Wind levels higher than used in the design have been identified by the ER referring to BS5400 (Bridges) and BS 6235 (Now withdrawn by BSI). However the basic wind speed for the Cork area identified in BS6399 (Part 2) 1997 is given as 25 m/s, although this is for the hourly mean value.

- The reduction factor for Carrigrenan for wind speeds was taken as 0.8. This is considered to be conservative given the location and protection afforded by the surrounding topography in relation to the exposed nature of Cork Airport.

- Whilst consideration of gusts is important it must be remembered that the most damage can occur during storms of long duration with high wave action. The max gust speed at Roche's point recorded over the last 30 years is 41 m/s with the mean 10-minute wind speed of 31m/s. By applying the 0.8 factor to this figure for Carrigrenan the wind speed is 24.8 m/s.

- At the site the waves from the South running up the 'West Passage' will give the most onerous conditions. Due to the orientation of the outfall and its location off Marino Point which will afford some protection, it is considered that the wave conditions, and therefore the rock sizing calculated, are reasonable and robust.

Section 3.0

The issue of scour and subsequent damage caused by propeller scour is one which is difficult to estimate given the lack of information on vessel type. However calculations have been carried out which give some indications as to the velocities near the bed with various power outputs and propeller diameters. These have been forwarded to the ER previously.

At a water level of MHWS directly above the diffuser there will be 13.5 m of water to the crown of the pipe, and approximately 15m to the surface of the concrete mattress. Any vessel with a draft of 12m manoeuvring directly over above the pipe with a water level of MHWS will cause damage to the pipe. With the water level below MHWS such a vessel will not be able to be within close proximity of the pipe as it will ground out first.

With water levels below MLWS vessels with a draft any greater than about 9 m manoeuvring above the pipe and diffuser could cause damage by impact. At this point in time scour due to propeller action becomes irrelevant, as damage to the diffuser by impact will have greater impact on the environment and outfall than scour induced by propeller action.

However, the possibility of propeller scour on the concrete block mattress has been investigated. The concrete mattress can be shown to be stable under propeller action created by a vessel with power output of 7000 Kw with a single screw and rudder (such as a container ship) acting directly above the pipe, i.e. 2m above the mattress.

If further details of the ships anticipated to be in the vicinity can be provided then more detailed calculations can be undertaken as the screw orientation, power and rudder configuration are important in the consideration of propeller induced damage.

It has been suggested that the trestles could be placed on piles. It is felt that this form of construction would not be suitable for this location as this type of construction will not only cause greater damage to shipping that should come into contact with it, due its more rigid nature, but also cost more to repair than the proposed design, should such an impact occur.
It is felt that by adopting the design taken a robust yet economic solution for the client has been achieved.

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Carlos Mollet
Project Manager
VanOord ACZ Ltd
Ballinure Header Chamber Site
Loch Mahon Technology Park
Cork
Republic of Ireland

Our ref
wecrof/61/Cro0086L
Your ref

11 January 2002

Dear Carlos,

Carrigrenan Outfall
Diffuser System Design

Further to our meeting on 9 January 2002, I am writing to confirm, clarify and expand on the principals behind our design of the outfall diffuser system, as addressed at the meeting.

Our design follows the recommendations, criteria and formulae as set out in the Water Research Centre (WRc) Design Guide for Marine Treatment Schemes Volumes I, II, III and IV, Report No UM 1009 dated May 1990. This 542 page WRc Guide was compiled following extensive collaboration involving expert opinion from across the water industry, academic institutions and specialist consultants, bringing together the best advice on investigations, engineering, design, operation and monitoring of marine outfall systems. The Guide has since become the authoritative text for the many dozens of effluent outfalls designed and built in UK coastal and estuarine waters over the past decade. Furthermore design methods and criteria from the WRc Guide have been incorporated into the regulations and guidelines covering discharges into tidal waters issued by the UK Environment Agency (EA) and the Scottish Environmental Protection Agency (SEPA).

The WRc Design Guide Volume II, Section 6.2, covers Initial Dilution. It includes full details of the methods and equations developed by Lee and others in 1987/8 to predict initial dilution, using parameters including plume buoyancy, port discharge, water depth and current velocity. These 'WRc Lee' equations have been incorporated into the EA and SEPA requirements for initial dilution compliance and are considered to be particularly appropriate to the tidal waters around Great Britain and Ireland. Hydraulic models based on these equations have been used to design diffuser systems for most UK outfalls during recent years.

The hydraulic model programme that we have developed to design the Carrigrenan outfall and diffuser system is based on criteria and methods from the WRc Guide and it incorporates the Wrc Lee equations to calculate initial dilution. The model shows that a diffuser with 22 ports would meet the specified requirements.

The HR Wallingford Report of June 1998 'Marino Point Discharge – Sediment Study', was based on the original outline design involving a diffuser at the same location, of similar basic dimensions, but with 36 ports. The HRW Report formed part of the tender information and considers the effects of discharged effluent mixing with the adjacent body of water, the deposition of discharged solids from the treated effluent and the effects of scour.

We can confirm that the proposed 22 port diffuser design is not at variance with the HRW Report in respect of its findings on initial dilution at the surface, jet velocity and sediment deposition. This is primarily because the port discharge velocity would be very similar and the total sediment content should be the same. Therefore the impact on vessel movement along the navigational channel should remain as negligible and the overall deposition potential from discharged solids (predicted by HRW at less than 73mm/year depth) should remain unchanged. Scour is unlikely to be an issue due to the proposal to provide heavy anti scour mattresses.

A further point to consider is plume overlap. Ideally ports should be spaced sufficiently far apart so that the buoyant plumes do not overlap, for if they do the overall dilution and mixing benefits are reduced. The worst case for dilution is in still or slow moving water, when the width of the plume at the surface is around 0.3 times the depth of water. Thus at Carrigrenan plume overlap would occur, even at low tide, if the ports are spaced closer than around 3m apart. It so happens that with a 22 port diffuser the ports are just over 3m apart. Increasing the number of ports above this would give little or no extra benefit for mixing.

Viewed in terms of operation and maintenance a diffuser system with a smaller number of large diameter ports will always be preferable to one with an equivalent larger number of small ports. This is because small ports are more prone to blockage and larger ports allow better access for suction and jetting hoses to remove any deposits within the main pipe.

In conclusion it is our considered opinion that the 22 port design is the best that can be offered in terms of achieving compliance with the specification, enabling practical construction and providing good operational service over the longer term.

Yours Sincerely

Rob Elvery
Principal Engineer

APPENDIX E
CATHODIC PROTECTION DESIGN

CATHODIC PROTECTION DESIGN

DESIGN REPORT UNDERTAKEN BY CORRPRO COMPANIES EUROPE LIMITED

Issue No 1 Rev No. 4
02 April 2002
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CORK MAIN DRAINAGE PHASE III

CARRIGRENAN OUTFALL CATHODIC PROTECTION

FOR STEEL DIFFUSER

Halcrow Ref: WE/CROF/61/094
Corrpro Ref:2242

CONTENTS

1.0 INTRODUCTION

Corrpro Companies Europe Limited (CCEL) have been retained by Halcrow Group Ltd to provide a design for cathodic protection system for the steel diffuser section of the Carrigrenan Outfall which is part of the Cork Main Drainage Phase III Project.

The Employer's consulting engineers are Pettit/Mott Macdonald, the project is being executed by Van Oord ACZ and Halcrow are designers for Van Oord.

This document provides the design for a sacrificial type cathodic protection system to protect the diffuser for a 60 year lifetime.

2.0 REFERENCE DRAWINGS

Following client drawings have been referenced

WE CROF.101E - Plan and long section

WE CROF 102E - Diffuser details.

3.0 BRIEF DESCRIPTION

The outfall will discharge treated and storm sewage effluent into the marine estuarial waters at Lough Mahon, Cork, in the Republic of Ireland.

The steel diffuser section of the proposed outfall pipe is 90 m long.

The coating is a 1.5 mm polyurethane (Durathane P factory applied to SA 2.5 prepared surface) for both internal and external surfaces.

The cathodic protection system is designed to protect the external surface of the pipe against corrosion of seawater; and is complimentary to the external coating.

The stated design life for the outfall is 100 years. For the external surface the assumption is that the CP system will become spent after 60 years and that the remaining 40 years will be achieved by corrosion allowance designed into the pipewall thickness.

4.0 SPECIFICATIONS CONSIDERED FOR THE DESIGN

DNV RPB 401 :1993 : Cathodic protection design.

BS7361 : 1991 : part 1 - Cathodic protection for land and marine structures.

NACE RP-0675-88 - Control of external corrosion on offshore steel pipeing.

5.0 PROTECTION CRITERIA

The Cathodic protection shall achieve a minimum protection level of -900mV for steel wrt Ag/AgCl/seawater after a sufficient polarisation period; assuming that sulphate reducing bacteria (SRB) is active.

If SRB is absent then the minimum protection criteria is - 800mV.

The design has been carried out to achieve -900mV protection levels.

6.0 DESIGN CALCULATIONS

6.1 SURFACE AREA

Assuming that the first 20 meters is partly burried and partly immersed, with the balance pipe completely immersed, the surface areas have been computed as follows:

OD of Pipe Mm	Length M	Area m^2	
1524	20	95.8	50% Buried 50% Immersed
1524	19.7	94.3	Immersed
1321	12.1	50.2	Immersed
1067	15.2	51.0	Immersed
914	9.1	26.1	Immersed
813	3.1	7.9	Immersed
711	3.0	6.7	Immersed
610	3.1	5.9	Immersed
457	3.0	4.3	Immersed
356	3.3	3.7	Immersed
324	3.3	3.4	Immersed
	Total Buried	47.9	
	Total Immersed	301.4	

6.2 COATING BREAKDOWN

Using a 1.5mm coating, the breakdown computed will be 21% as mean/maintenance and 44% final over the 60 year lifetime.

This is based upon calculating the coating breakdown from DNV for a 500 microns (0.5mm) coating (category IV) as well as a 3mm coating and extrapolation at 1.5mm thickness on a coating breakdown v/s log thickness plot.

Coating breakdown for 0.5mm coating as follows: (refer to 6.5.3 of DNV)

Mean - 38%

Final - 74%

Coating breakdown for a 3mm coating as follows: (refer to 6.5.3 of DNV)

Mean - 11%

Final - 23%

See attached plot

6.3 WATER RESISTIVITY

The resistivity of the sea water is assumed to be 20 ohm cm.

6.4 CURRENT DENSITIES

Current density as per DNV in seawater (20degreeC)

Mean 80 mA/m²

Final 110 mA/m²

Current density as per DNV for Buried steel

Mean 20mA/m²

Final 20mA/m²

6.5 CURRENT DEMAND

This is computed as surface area x coating breakdown x current density.

Thus, current demand is as follows:

	Surface Area, m^2	Coating breakdown Final	Final current, Amps
Buried	47.9	44	0.42
Immersed	301.4	44	14.58
		Total	15.00

	Surface Area, m^2	Coating breakdown Mean	Mean current, Amps
Buried	47.9	22	0.20
Immersed	301.4	22	5.06
		Total	5.26

Now the design of a sacrificial anode system must satisfy the anode weight requirement calculated from mean current and output current for the entire anode mass calculated on the basis of final current demand.

6.6 CHOICE OF ANODE MATERIAL

We recommend an aluminium base anode material comprising of an Aluminium - Indium - Zinc alloy having an electrochemical capacity of at least 2500 Ahr/Kg in 20°C seawater with a closed circuit potential of -1.05V in seawater and -0.95V in buried conditions w.r.t. Ag/AgCl reference electrode.

6.7 ANODE MASS CALCULATION

The net anode mass calculated from mean current equals;

$$M = \frac{\text{Mean current x life (60 years) x 8760}}{\text{2500 x utilisation factor}}$$

Where mean current = 5.26A
Utilisation = 0.8

$$\text{Total anode mass} = \frac{5.26 \times 60 \times 8760}{2500 \times 0.80}$$

$$= 1382 \text{ Kgs}$$

6.8 PROPOSED ANODES

Recommended Corrpro anode is AR2400, with anode dimension 210mm depth x 210 mm av width x 2622 mm length av. Length, with 240 Kgs net weight

The anode chosen must satisfy both final current and mass requirements.

Anode current, is calculated from Dwight's formula:

$$R = (\text{Resistance, ohms}) = \frac{P\,(\ln 4L/r-1)}{2.\pi L}$$

Where P = resistivity = 20 ohm cm
L = 238.6 cm, r = 6.58 cm (equivalent radius)

Substituting final resistance =

$$\frac{20}{2 \times \pi \times 238.6} \left(\frac{\text{Ln } 4 \times 238.6}{6.58} - 1 \right)$$

$$= 0.053 \text{ ohms}$$

$$\text{Anode Current} = \frac{\text{Volt drop (0.15)}}{0.053 \text{ (resistance)}}$$

$$= 2.83 \text{ amps (final current)}$$

See spread sheet attached for further details.

Volt drop = Difference between protected potential ie -0.9V wrt Ag/AgCl and anode potential: -1050 mV wrt Ag/AgCl.

No of anodes, therefore

$$\text{Anodes by current demand} = \frac{\text{Final current demand}}{\text{Final anode output}}$$

$$= \frac{15}{2.83} = 5.3 = 6 \text{ Anodes}$$

$$\text{Anodes by mass} = \frac{\text{Total mass}}{\text{Anode weight}}$$

$$= \frac{1382}{240} = 5.75 = 6 \text{Anodes}$$

7.0 ANODE ATTACHMENT

We recommend the 6 anodes mounted on 2 steel skids with 3 anodes per skid as per attached drawing AM5506, with each skid attached to the outfall pipe and on either side of the pipe using continuity cables. Two continuity cables are provided for each skid/sled. The sleds are placed onto the seabed with the support frame buried to prevent any drift.

A monitoring test station is considered, located at the land end to facilitate checks on isolation and protective potentials on the diffuser.

8.0 BILL OF QUANTITIES

Qty	Description	Scope
6 Nos	Anodes, Al alloy 240Kg net wt	Corrpro
2 Nos	Steel Skid	Corrpro/ Van Oord
10 Nos	Thermit weld connections with coating rehabilitation mastic	Corrpro/ Van Oord
1 No	Monitoring station in steel	Corrpro
50m	$1x35mm^2$ EPR/CSPE cable	Corrpro

file: TRAPSECT - Trapezoidal cross-section Stand-off Anodes

PROJECT: *Halcrow-Outfall*
ANODE TYPE: AR 2400

ANODE:

Alloy Density	g/cc	2.71
Anode Dimensions:		
Anode Length	cm	262.2
Anode mean Width	cm	21
Anode Depth	cm	21
Anode nett Volume	cc	88867.293

INSERT:

Steel density	g/cc	7.85
Insert Details:		
Length in Anode	cm	262.2
Tube outer dia	cm	11.4
Total Insert length	cm	277.4
Tube unit weight	kg/M	20.03
Insert Cross-sectl Area	sq cm	102.07026
Insert Vol in anode	cc	26762.907

NETT WEIGHT	kg	240.83
Insert Weight	kg	55.56
GROSS WEIGHT	kg	296.39

ANODE CURRENT OUTPUT:
(DNV RP B401:1993)
Output Design Parameters:

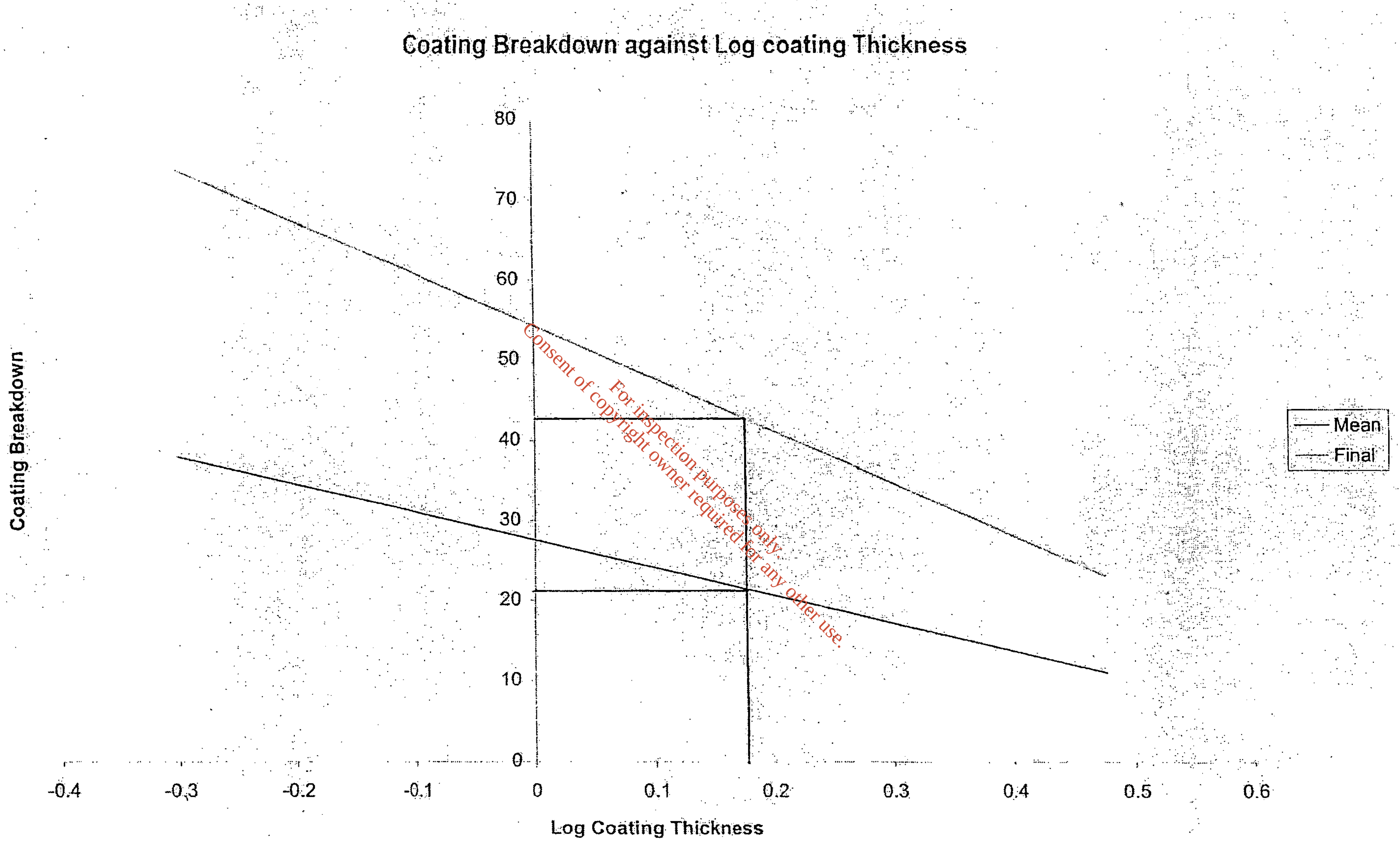
Mean Resistivity	Ohm cm		20
Closed Circuit Potential		-mV	1050
Protection Potential		-mV	900
Driving Voltage		Volts	0.15
Utilisation Factor		mean	0.45
		final	0.85

Mean Anode Length	cm	250.401
Final Anode Length	cm	239.913
Anode Equiv. Radius:		
initial	cm	13.369027
mean	cm	9.5826212
final	cm	6.9765835
Stand-off Distance	cm	300
Stand-off Correction Factor		1

INITIAL CURRENT	Amps	3.675
MEAN CURRENT	Amps	3.233
FINAL CURRENT	Amps	2.881

Initial Resistance	Ohms	0.0408202
Mean Resistance	Ohms	0.0463913
Final Resistance	Ohms	0.0520627

ANODE DESIGN CALCULATION



Coating Breakdown against Log coating Thickness
Coating Breakdown
80
70
60
50
40
30
20
10
0
-0.4
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Log Coating Thickness
Mean
Final

CATHODIC PROTECTION DESIGN

CONFORMITY STATEMENT AND CERTIFICATES

Issue No 1 Rev No. 4
02 April 2002
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Fax Message

No of Pages (including this one):

To: **Halcrow**

Attn: **Rod Elvery**

From: Raju Narayan

Date: 21 March 2002

Subject: Carrigrenan Waste Water Treatment plant
Steel Diffuser – Cathodic Protection

Dear Sir,

This has reference to Mott Macdonald comments on the adequacy of the CP design proposed by Corrpro.

We confirm that the electrochemical capacity of 2500 amphr/kg considered by us for aluminium- indiunm-zinc alloy anodes in seawater under ambient (less than 30 degree C) conditions is adequate. This is based upon several years of proven performance of both alloys namely Impalloy III and BA778S that is usually specified by us.

The long-term capacity results of both alloys certified by DNV (see attached) confirms that the figure of 2500 is the minimum achievable.

Please feel free to contact us in case you need additional information

Regards

Raju Narayan

HALCROW WE6/HGa/WQ	
SCAN Y/N (HGa applicable only)	
LOG No.	583
RECEIVED DATE	21 MAR 2002
ACTION DATE	
ACTION BY	2G
CIRCULATION	
	DC

Corrpro Companies Europe Limited, Adam Street, Bowesfield Lane, Stockton-on-Tees, Cleveland TS18 3HQ. England
Tel: (01642) 614106 Fax: (01642) 614100 E-Mail: ccel@corrpro.co.uk
Company Reg No. 944432 Registered in England

VERITEC

TECHNICAL REPORT

Johan Berenteensvei 109-111
P.O.Box 60
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Telephone (+47) 55 94 00 30
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Registration No. 934 763 052

Date: 29.11.94	Div.-Dept.: T&I-250	
Approved by: Bård Espelid, Head of Section		
Client, Sponsor: Wilson Walton	Client ref.: Michael J. Moffat	Project No.: 250 7 1450

Summary:

The corrosion laboratory of Det Norske Veritas Indistry AS (DNV Industry) in Bergen has carried out a test of anode alloy 778S, according to the DNV Specification RP B401 Appendix B.

Capacity (Ah/kg)	Potential mV vs. Ag/AgCl(seawater) ref. electrode
2644	- 1112
2626	- 1112
2615	- 1117
2621	- 1116
2630	- 1117
2637	- 1117
2420	- 1117
2637	- 1117
2574	- 1116
2617	- 1114

Mean capacity: 2602 Ah/kg, mean potential: -1116 mV vs. Ag/AgCl(seawater) ref. electrode

Report No.: R794424	Subject Group: E7
Report title: LONG TERM TESTING OF BA778 MODIFIED ANODE ALLOY	
Work carried out by: Sille Grjotheim, Bergen Schelderup	
Work verified by: Henning Bødtker	
Date of last rev.:	Rev. No.:
Number of Pages: 1	

Indexing terms

CORROSION TESTS
SACRIFICIAL ANODES
ALUMINIUM

- [X] No distribution without permission from the responsible department/project within DNV Industry
- [ ] Limited distribution within DNV Industry
- [ ] Unrestricted distribution

29 November 1994, BGC/250 O:\WPDATA\RAPPORT\1452.KOR

DNV

DET NORSKE VERITAS

TYPE APPROVAL CERTIFICATE

CERTIFICATE NO. S-2727
This Certificate consists of 2 pages

This is to certify that the
Sacrificial Anode for Corrosion Protection

with type designation(s)
IMPALLOY III

Manufactured by
Trident Alloys Limited
Walsall, West Midlands WS3 2XW, United Kingdom

is found to comply with
DNV's Recommended Practice B401.
DNV's Approval Programme No. IOD-90-TA1

Application
Approval is given for use of the sacrificial anode material in seawater below 30°C.
Approval is given for the sacrificial anode material and not for anode design.

Place and date
Høvik, 1999-04-15
for DET NORSKE VERITAS AS

John Olav Nøkleby
Head of Section

DET NORSKE VERITAS 1864
Local Office
DNV Sheffield

This Certificate is valid until
2003-03-31

Gisle Hersvik
Surveyor

Notice: This Certificate is subject to terms and conditions overleaf. Any significant change in design or construction may render this Certificate invalid.

DNV

DET NORSKE VERITAS

TYPE APPROVAL CERTIFICATE

CERTIFICATE NO. S-2737
This Certificate consists of 2 pages

This is to certify that the
Sacrificial Anode for Corrosion Protection

with type designation(s)
IMPALLOY III

Manufactured by
Trident Alloys Limited
Walsall, West Midlands WS3 2XW, United Kingdom

is found to comply with
DNV's Recommended Practice B401.
DNV's Approval Programme No. IOD-90-TA1

Application
Approval is given for use of the sacrificial anode material in seawater below 30°C.
Approval is given for the sacrificial anode material and not for anode design.

Place and date Høvik, 1999-04-15 *for* DET NORSKE VERITAS AS	DET NORSKE VERITAS 1864	*This Certificate is valid until* 2003-03-31
John Olav Nøkleby *Head of Section*	*Local Office* DNV Sheffield	Gisle Hersvik *Surveyor*

Notice: This Certificate is subject to terms and conditions overleaf. Any significant change in design or construction may render this Certificate invalid.

DNV

Cert. No.: S-2727
File No.: 491.21

Product description

Sacrificial anode material **IMPALLOY III.**

Al-Zn-In alloy with current capacity of about 2580 Ah/kg and a closed curcuit potential of about -1110 mV versus Ag/AgCl/seawater reference electrode.

Type Approval documentation

1. Earlier approval, Certificate S-1405 and S-2297.
2. 12 months testing carried out at DNV Industry, Bergen, ref. Report No. 795057, dated 7 April 1995.
3. Letter from DNV Sheffield dated 4 February 1999, including Survey report SHF 99/5022 dated 28 January 1999.
4. Emails from DNV Sheffield, dated 17. March 1999 and 14. April 1999.

Tests carried out

Type Testing as per **Type Approval documentation.**

Marking of product

For traceability to this Type Approval Certificate, the products are to be marked: **Impalloy** with cast number, where "X" in the cast number denotes **Impalloy III.**

Certificate Retention Survey

The scope of the Retention/Renewal Survey is to verify that the conditions stipulated for the Type Approval is complied with and that no alterations are made to the product design or choice of materials.

Survey to be performed after two (2) years (Certificate Retention Survey) and at renewal after four (4) years (Certificate Renewal Survey).

The main elements of the survey are:

- Ensure that **Type Approval documentation** is available.
- Review: design, materials, production process, and performance with respect to possible changes, in order to ensure compliance with **Type Approval documentation** and/or referenced material specifications.
- Ensure traceability between manufacturer's product marking and the DNV Type Approval Certificate.

END OF CERTIFICATE

F.A.O RAJU

trident alloys
moving metallic innovation into new world markets

MATERIAL SPECIFICATION

IMPALLOY III

Impalloy III is an aluminium alloy developed and manufactured by Impalloy. It is an AL-ZN-IN alloy specifically suitable for the North Sea environment and is approved by Det Norske Veritas - Type Approval Certificate No. S-2727 (copy attached) which also shows the alloys long term free running characteristics.

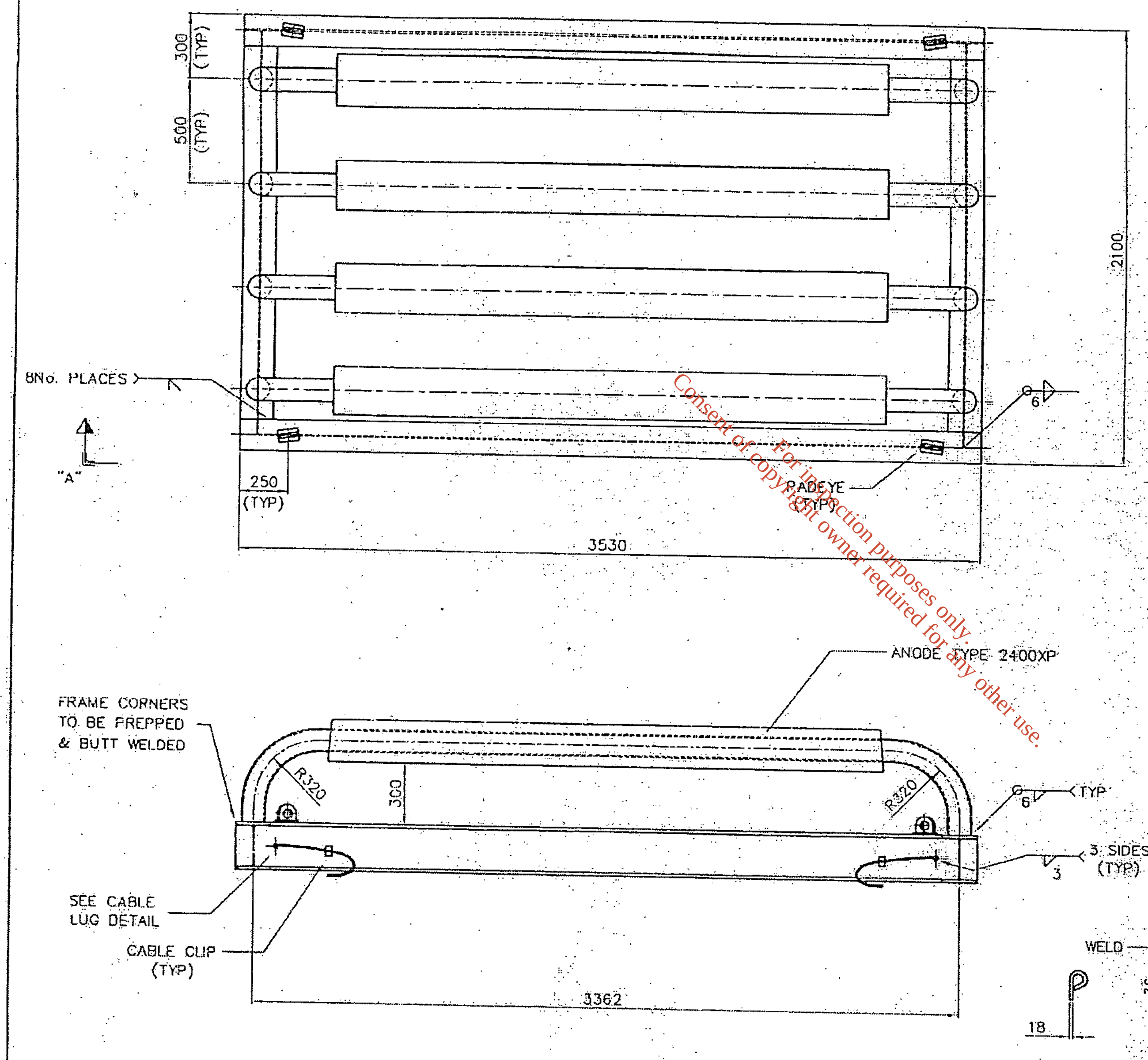
IMPALLOY III

Chemical Analysis Percentage by weight as follows	
Fe	0.12 max
Si	0.10 max
Cu	0.006 max
Zn	2.8-6.5
In	0.01 - 0.02
Ti	0.025 max
Others (each)	0.02 max
Aluminium	Remainder

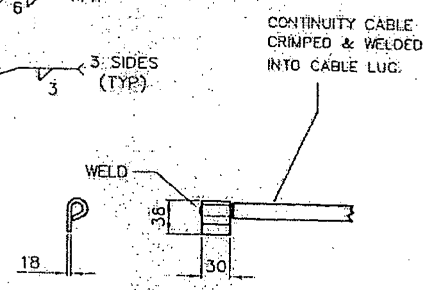
The performance of Impalloy III when tested in accordance with DNV RPB401 at 20°C ± 1°C will be	
Capacity	2500 Ampere Hrs/kg minimum DNV RPB 401 (1993) Appendix A Test
Closed Circuit Potential	-1.05 Volts or more negative w.r.t. Ag/AgCl when measured at the end of 24 hour period operating at 0.4mA/cm^2

CATHODIC PROTECTION DESIGN

DRAWING AM 5507 – PROPOSED ANNODE ARRANGEMENT



ELEVATION "A–A"



CABLE LUG DETAIL
MATERIAL:– 3 THK. M.S. PLATE
TO BS EN 10025 FE 430B.

NOTES

FRAME SECTION 203 x 133 x 25 UB
ALL SLED MATERIAL TO BE EN 10025 FE 430 B

CONTINUITY CABLES TO BE $35mm^2$ EPR/CSP x 10m LONG. TERMINAL TO BE ADVISED.

SLED TO BE BLASTED TO ISO 8501/1 SA 2½ AND ALL EXPOSED STEELWORK, INCLUDING ANODE LEGS TO BE GIVEN 2 No.COATS OF EPOXY PAINT TO 200µm DFT TOTAL

ALUMINIUM ANODE TO IMPALLOY III

REMOVE ALL SHARP EDGES

PADEYES TO BE OLD AP No. 4018 WITH PROOF TEST LOAD OF 9.5625 TONNES. PADEYE WELD SUBJECT TO NON–DESTRUCTIVE TESTING. (SUPPLIER TL LIFTING LTD)

WEIGHTS (Kg)			
ITEM	NETT	INSERT	GROSS
ANODE	240	86.9	326.9
SKID			234.8
ASSEMBLY	960		1542.4

Carrpro Companies Europe Limited
"For Every Corner of Your World"

HALCROW
CARRIGRENAN OUTFALL
ALUMINIUM SLED ANODE

Drawn	Date	Checked	Date
RO	25/3/02	RN	25/3/02
Scale: NTS	Dwg No. AM 5507		Rev. 01

PLANT

WASTE WATER TREATMENT PLANT & OUTFALL

CLIENT

CORK CITY COUNCIL

DOCUMENT TYPE

FINAL DESIGN – BASIC PROCESS DESCRIPTION

DOCUMENT NUMBER

E6010091 – DA401

5	Final Design Report Resubmission	FVO	DCH	VCA	18/07/02
4	Final Design Report	FVO	DCH	VCA	29/07/02
3	Draft Design Report Resubmission	FVO	DCH	VCA	08/04/02
2	Draft Design Report Resubmission	TSI-FVO	GTO	VCA	31/01/02
1	Draft Design Report	TSI	DBA	GTO	08/11/01
0	First Issue	TSI/ODSA	DBA	GTO	31/10/01

SCOPE

Scope of this document is mainly to report the main figures and parameter used for the detailed calculations and the basic sizing of water line, sludge line and of the drying section.

Further information is submitted with the design report of each individual section.

Following documents can be consulted for a better understanding of this process report:

- Mass Balance — doc. No. DA 402 rev. 3 dated 26/07/02
- Process data — doc. No. DA 403 rev. 2 dated 26/07/02
- Hydraulic Gradient — doc. No. L 002 rev. 8 dated 28/05/03
- Hydraulic Calculation Report — doc. No. DA 404 rev. 3 dated 24/06/02
- Future imported sludge layout — doc. No. A 018 rev.0 dated 26/07/02
- Inlet works & Pre-aeration tanks — doc. No. S 001 rev.6 dated 13/06/03
- Primary clarification — doc. No. S 002 rev.6 dated 13/06/03
- Storm tanks — doc. No. S 003 rev.6 dated 13/06/03
- Cyclazurs — doc. No. S 004 rev.6 dated 13/06/03
- Cyclazurs — doc. No. S 005 rev.6 dated 13/06/03
- Secondary sludge thickening — doc. No. S 006 rev.5 dated 13/06/03
- Sludge treatment — doc. No. S 007 rev.6 dated 13/06/03
- Digestion: heater & water line doc. No. S 008 rev.7 dated 13/06/03
- Digestion: digesters — doc. No. S 009A rev.6 dated 13/06/03
- Digestion: gas line — doc. No. S 009B rev.7 dated 13/06/03
- Sludge dewatering — doc. No. S 010 rev.6 dated 13/06/03
- Final effluent chamber — doc. No. S 012 rev.6 dated 13/06/03
- Odour control chemical dosing — doc. No. S 013 rev.7 dated 13/06/03
- Polyelectrolyte dosing — doc. No. S 014 rev.4 dated 13/06/03
- Potable water and compressed air — doc. No. S 015 rev.5 dated 13/06/03
- Odour control — doc. No. S 016 rev.7 dated 13/06/03
- Drying system (Innoplana) — doc. No. S 018A rev.7 dated 11/06/03
- Drying system (Innoplana) — doc. No. S 018B rev.8 dated 11/06/03
- Drying system (Innoplana) — doc. No. S 019 rev.7 dated 11/06/03
- P&ID diagram symbolisation — doc. No. S 020 rev.4 dated 13/06/03
- Generator fuel line schematic — doc. No. S 021 rev.0 dated 13/06/03

Explanation of the abbreviation and symbol used is given at the paragraph No. 6

INDEX

1 General Description

Carrigrenan Waste Water Treatment Plant will treat the domestic and industrial waste emanating form the city of Cork and discharge it to the sea via a submarine outfall.

1.1 Flow and Loads

1.1.1 Flow

The average dry weather flow that expected in AD 2020 for a population of 194.000 Inhabitants is 59.359 m^3/d

DWF	0.687 m3/sec
Maximum flow treated (during wet weather and including returns liquors generated in the plant)	4.18 m3/sec

The plant will treat 4.18 m3/sec in the preliminary unit, up to 2,2 m3/sec in the primary sedimentation (3 x DWF) and up to 1,93 m3/sec in the biological treatment (2,5 x DWF). The storm overflow will be temporarily stored in four storm tanks and pumped back to treatment during dry weather.

1.1.2 Loads

Civil loads correspond to 47% of total load (53% industrial).
The equivalent treatment is then of about 413.000 inhabitants.

Pollutant – Including return flows

BOD (Biochemical Oxygen Demand)	25872 kg/day 381 mg/l
COD (Chemical Oxygen Demand)	55338 kg/day 814mg/l
TSS (Total Suspended Solid)	26020 kg/day 383 mg/l

1.2 Final Effluent

The final effluent characteristics are as follows:

BOD (Biochemical Oxygen Demand)	25 mg/l
COD (Chemical Oxygen Demand)	125 mg/l
TSS (Total Suspended Solid)	35 mg/l

1.3 Unit of Treatment

Flow arrives to the plant from Header chamber (3.64 m3/sec) and from two local pumping stations (Flaxfort and Courtstown: 0.522 m3/sec)

WATER LINE
Inlet Building

-*Screening section* to remove materials having size bigger than 5 mm diameter. The screened material is washed, compacted and bagged.

-*Degritting* to remove sands and grit. Sand and grit are washed and discharged to skips

Flow from Header chamber has already been screened and degritted.

After screening and degritting sections, the two flows will enter directly into the preaeration section.

Preaeration

Preaeration is provided for the removal of odorous gases generated from the sewage septicity. The gases removed are treated by the odour control system.

Primary Treatment

After the first storm overflow, crude sewage is fed to 2 No. Primary Settlement tanks each of 33.75 m nominal diameters. Sedimentation removes settable solids and associated BOD.

Secondary Treatment

The primary clarified effluent flows by gravity from primary settlement tank via second storm overflow and flow measurement to the Sequencing Batch Reactor.
These are 8 No. rectangular basins each 8491.5 m^3 that are intermittently filled and draw in sequence. During the filling period air is injected and with the help of the activated sludge stored in the basin the dissolved pollution is converted in new cellular material (Activated Sludge). When aeration stops the activated sludge settle and the clear supernatant is drawn off and discharged as final effluent.

Effluent discharge
Final "clear" water is discharged through a sea outfall off Marino Point.

SLUDGE TREATMENT

Thickening

Sludge separated in the primary sedimentation and in the settling phase of Sequencing Batch Reactor is thickened the former by gravity picket fence thickeners and the latter flocculated with polyelectrolyte, with belt Thickeners.

Digestion

Thickened sludge is anaerobically digested in 3 No. 3400 m3 steel digesters with outside insulation. The sludge is maintained for 21 days at a constant temperature of 35°C. In this way, the organic material is stabilised (no more fermenting) and partially converted to biogas (Carbon Dioxide + Methane).

Dewatering and Drying

After digestion the solids content in the sludge is 3.6 %. . For a safe disposal needs to be dewatered to a solid content of about 23% and dried to a solid content of 90%.

The biogas produced by the digestion, stored in a gasholder, is used for the thermal drying of the sludge.

2 Data used for the sizing of the plant

2.1 Flow data (year 2020 figures)

2.1.1 Crude Sewage Flow (excluding the returns)

- Crude Sewage Dry Weather Flow (DWF) = 59,359 m3/day => 2,473.3 m3/hour => 687 l/s (from Specification).

- Crude Sewage Maximum Flow (WWF) = 14,983 m3/hour => 4.16 m3/s
(From Specification)

2.1.2 Flow to Primary treatment:

- Maximum Flow = 7920 m^3/hour (2.2 m^3/s) (from Works Performance Guarantee + liquors returns).
- Flows in excess to 2.2 m^3/s are diverted to the Storm Tanks (up to 2.08 m^3/s).

2.1.3 Flow to Secondary treatment:

- Maximum Flow = 6,948 m^3/hour (1.93 m^3/s).

- Flows in excess to 1.93 m^3/s are diverted to the Storm Tanks (up to 0.25 m^3/s).

2.1.4 Peak Flows

- According to the Works Performance Guarantee, the plant has to be designed for « the Design DWF as expressed in table 5.3 of volume 4 plus 20% is exceeding as an average daily dry weather flow calculated on a monthly basis ».

- To get the Average Daily Flow (ADF), is assumed that the ADF is 1.2*DWF = 71,231 m3/day => 2,968 m3/h.
  => 3,378 m3/h (including returns).
- To get the Average Daily Flow Peak (ADFP), is assumed that the ADFP is 1.6*ADF => 4,749 m3/h.
  => 5,171 m3/h (including returns).

2.2 Loads data (year 2020 figures):

2.2.1 Loads:

Design load (from the Specification)
- BOD = 24,792 kg/day
- COD = 49,938 kg/day
- TSS = 23,320 kg/day

These loads include the industrial loads indicated just below.

Industrial contribution to the load (from the Specification):
- BOD = 13,103 kg/day (52.85 %)
- COD = 25,391 kg/day (50.85%)
- TSS = 9,683 kg/day (41,52%)

2.2.2 Peak factor (for loads)

– According to the Works Performance Guarantee, the plant has to be designed for the « Design Loads of year 2020 as expressed in Table 5.4 of Volume 4 plus 20% are exceeded as individual average loads calculated on a monthly basis».

– Furthermore, plant design incorporates a peak-loading factor of 1.6 as per standard industry practices and schedule of modular expansions Vol. 5 of Tender Submission. On this basis the plant is designed to cater hourly organic loads into the works of up to 60% greater than the hourly average daily loads as calculated from table 5.4 Volume 4.

– Concerning the sludge line the following parameters have been considered:

- For 1.0 * Design Load the Sludge Production corresponding to 1.0*DL
- For the 1.2*DL the Sludge Production corresponding to 1.2*DL.
- For 1.6 * DL hourly peak factor has been considered that the corresponding sludge production will last for no more than 5 days (non-consecutive) over one month.

As consequence the maximum capacity for the Sludge Line will be:

- 5 days of sludge production @ 1.6 * DL (5 * 1.6 = 8)
- 25 days of sludge production @ 1.2 * DL (25 * 1.2 = 30)

Maximum capacity = (30+ 8) / 30 = 1.27 * DL

3 Treatment Process:

3.1 Pre-treatment:

3.1.1 Screening:

Local Flows from the Flaxfort and Courtstown pumping stations:

These flows are screened on automatic screens:
- Type : 2D perforated plate
- Opening : 5mm
- Capacity : 1,880 m^3/hr

Screening.

Start stop of the clogging type screens is automatic according to the signal coming from differential level transmitter. In case of failure of one screen the other one will be still working. At the inlet and at the outlet of each automatic screening channel one manually operated penstock (normally open) is provided in order to stop the flow in case of maintenance. Channel drainage system is also provided

In case of emergency, the Local Flows can be screened through a manually raked bar:
- Opening :10 mm
- Capacity : 1,880 m3/hr

Upstream this screen a manual penstock is provided. In emergency, it will be possible for the flows to overflow the penstock without operator's intervention.

The screenings, coming from the automatic screens, will be washed and compacted on two units (1 duty / 1 assist).

Flow screened material = 3.6 kg/1000m3 (assumed)
Maximum production = 165 kg/d
Dry solid content of compacted material > 30 %
Compacted screened material = 550 kg/d

A submersible pump will be provided to empty the screen channels.

Header Chamber flow:

No equipment is provided since the Header Chamber effluent is already screened down to 5 mm.

It has been assumed here that the screening system was the same than the one required in the Specification: « apertures not exceeding 5 mm in either direction ». This is very

3.1.2 Degritting:

Local flows from the Flaxfort and Courtstown pumping stations:

These flows are degritted on one unit. Characteristics are:
- Jones and Atwood Circular Covered Type or similar.
- Capacity: 1,880 m^3/hr/unit

Grits are fluidised with wash-water and pumped to a grit classifier to separate them from water.

In case of emergency, the degritting unit can be bypassed.

Grits production (max) = 0.04 kg/m^3 => 1,804 kg/d
Concentration of grit as extracted from grit trap = 2 kg/m^3
Volume extracted = 902 m^3/d

Water consumption:

Water consumption for screens cleaning and grit washing = 1500 m^3/d => 0.018 m^3/s

Total inlet flow = 0.522 m^3/s
Water consumption = 0.018 m^3/s

Total Maximum flow = 0.540 m^3/s

Header Chamber flow (3.64 m^3/s):

No equipment is provided since the Header Chamber effluent is already degritted.

We assumed that the degritting system had at least the same efficiency than the one required in the Specification: « the grit separators shall be capable of removing at least 95% of particles with a specific gravity of 2.65 g/cm^3 and with a diameter of 0.2 mm and greater ».

3.1.3 Major flow measurement

Location of flow measurement point complies with the requirement.

The following flow measurements are provided as the minimum requirement from clause 8, volume 4.

Flow line	Measurement point	Flowmeter type	
		Venturi flume	Magnetic
1.From Flaxfort	Pipe upstream screen		ND 500
2.From Courtstown	Pipe upstream screen		ND 150
3.Downstream pre-aeration storm overflow	Pipe to Primary sedimentation		2 x ND 1000
4.From pre-aeration storm overflow to storm tank	1.5 m wide channel	OK	
5.Downstream primary sed. Storm overflow	Pipe to SBR		ND 1000
6.Flow discharge from biological treatment	1.5 m wide channel	OK	
7.Flow discharge from storm tanks	1.5 m wide channel	OK	
8.Return liquors from sludge treatment	Delivery pipe to Primary Clarified Effluent chamber		ND 200
9.Surplus Activated Sludge	Delivery pipe to SBR sludge holding tank		ND 200
10.Combined thickened sludge	Delivery pipes to digesters		3 x ND100
11.Screenings & grit washing water	Branch pipe to inlet building from effluent water ring mains		ND 80
12.Potable water	Booster set delivery pipe		ND 80
13.Dried Sludge	Weighting system	Weight measurement	

Notes:

- Combined flows downstream of the pre-aeration are measured by summing 3 & 4
- Influent counting for payment purpose is achieved by subtracting 11.from 3 & 4.
- Measurement point 3 has 2 flowmeters in serial arrangement: these meter the same instant flow and are designed to control each other.
- Measuring point 10 has 3 flowmeters in parallel arrangement: the total thickened sludge flow is achieved by summing the 3 individual measures.
- The above table gives the minimum flow measurements required by the ER. However other flow measurements are supplied along the sludge and gas lines, for regulation or operation purposes. All flowmeters are indicated on P&IDs.

3.1.4 Sampling points

Sampling equipment will be provided according to the requirement of volume 3 (Operation and Maintenance) and Volume 4 Clause 8 Flow Measurement

The minimum sampling point provided are listed in the following table

Location	Sampling type
Combined flows upstream preaeration tank	Automatic
Overflow from the storm tank	Automatic
Flow to secondary treatment	Automatic
Final effluent	Automatic
Returns liquors	Manual
Surplus activated sludge	Manual
Combined thickened sludge to digesters	Manual
Dried sludge	Manual
Odour control plants outlet stack	Manual
Odour at the site boundary and receptors	Manual

3.1.5 Preaeration:

The characteristics of the preaeration are:

- 2 basins of 2,500 m^3 total volume each
- length = 30.0 m
- width = 9.3 m
- side water depth = 9.0m

- Flows
- DWF = 0.687 + 0.018 = 0.705 m3/s
- ADF = 0.687 x 1.2 + 0.018 = 0.842 m3/s
- WWF = 4.18 m3/s

- retention time
- @ DWF = 1,97 h
- @ ADF = 1.66 h
- @ WWF = 0.33 h

- *3 blowers (2 duties and 1 standby)*
- Unit flow = 4,500 Nm3/h
- Delivery pressure = 10 m

At the pre-aeration outlet, floating material and grease will be collected with a scum-baffle from then this material will be removed and sent to disposal.

One pre-aeration tank can be bypassed for emergency. In this case, all the flow will go through the other pre-aeration tank.

3.2 Primary Clarification:

Two Primary Tanks of 33.75-m diameter and 3.5 m sidewall water depth will be provided. Each Primary Tank will be equipped with a rotating half-diameter scraper bridge.

To avoid shock loads on the treatment process, the return liquors are pumped upstream the Primary Tanks.

The two tanks will be covered and connected to the odour control system.

The upflow velocity in the primary tanks will be:
- for DWF: 1.58 m/h
- for ADF: 1.89 m/h
- for WWF: 4.43 m/h

Characteristic of the influent in front of the Primary Clarification (with the return liquors)
- BOD (1.2 DL) = 31,030 kg/d => 383 mg/l
- COD (1.2 DL) = 66,326 kg/d => 815 mg/l
- TSS (1.2 DL) = 31,164 kg/d => 384 mg/l
- VSS/TSS ratio: it was assumed to be about 76%. This ratio is pretty high due to a high food industrial load (especially coming from the brewery influent).

Percentage removals in primary sedimentation.

Following removal percentages are selected for the sizing of the whole plant (secondary water treatment and sludge treatment):

	Percentage Reduction
TSS	31.50 %
BOD	17.58 %
COD	18.55 %

The sludge production (1.2*DL) with the above removal will be:
- flow = 982 m^3/d = 41 m^3/h
- TSS = 9,817kg/d
- Design concentration = 10 kg/m^3

The sludge extraction will be done at about 10 g/l to avoid risk of septicity in the Primary Tanks. For that, three pumps will be installed (2 duties and 1 standby). In order to face the possible flow and load peaks, each pump will have a maximum flow of 45 m^3/h.
The nominal flow rate will be set manually, and will have sequenced operation.

For the load condition 1.6*DL the sludge production will be:
- flow =1282 m3/d = 53 m3/h
- TSS = 12824kg/d
- Design concentration = 10 kg/m3

As stated at point 2.2.2 the maximum capacity considered for the sludge treatment is equivalent to an average sludge production calculated for 5 days of sludge production at water 1.6*DL and 25 days of sludge production at 1.2*DL and averaged over 1 month. This is equivalent to:
- Flow = 1032 m3/d = 43 m3/h
- TSS = 10318 kg/d
- Design concentration = 10 kg/m3

Scums from the Primary Tanks will be collected in a sump and pumped to the Thickened Sludge Holding Tank before the digestion unit. (1 duty and1 standby pump).

One Primary Tank can be bypassed for emergency. In this case, all the flow will go through the other Primary Tank.

3.3 SBR:

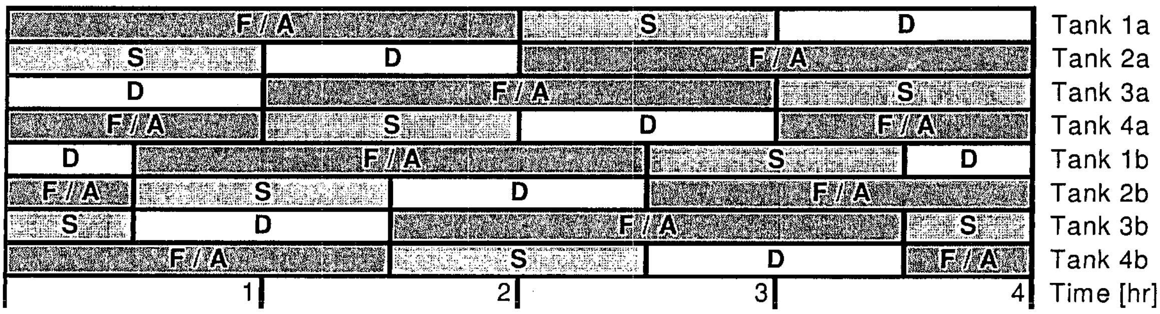
3.3.1 Brief description:

The primary clarified effluent flows by gravity from the Primary Tanks, via the second storm overflow to the eight SBR basins.
A standard 4-hour cycle per basin is used during DWF conditions. This changes to a 3-hour cycle per basin during prolonged Wet Weather Flow (WWF) conditions.
WWF conditions concern all the flows above ADFP (1.4 m3/s)

At the start of each cycle, the volume of liquid in the tank increases from a set minimum Bottom Water Level (BWL) in response to the varying influent flow rate. Aeration stops after a pre-determined time to allow the biomass to flocculate and settle under quiescent conditions. The treated supernatant is then removed by lowering the decanter weir arms until the water level returns to the BWL.

The eight tanks will have the following cycle profile during DWF conditions:

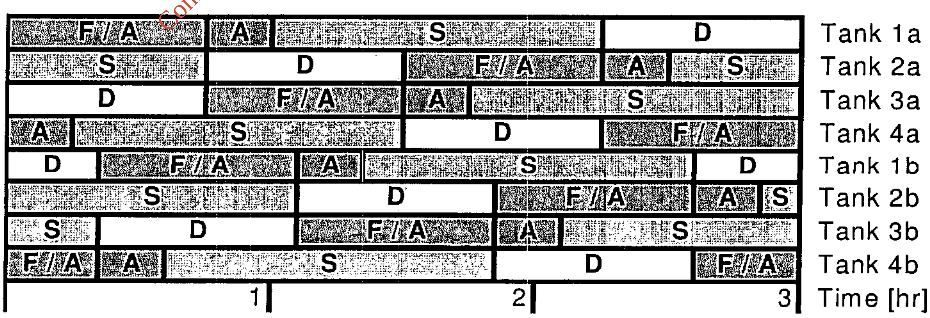
DW Cycle



Where:
F = Fill phase
A = Aerate phase
S = Settle phase
D = Decant phase

The 3-hours WWF cycle will start after 30 minutes of flow exceeding average daily peak flow by 1% which correspond to a flow rate of about 5225 m^3/h

WW Cycle



Selector zone:

Selector zone is provided for each one of the eight cells.
The selector is designed to provide plug-flow characteristics. Vertical baffles are installed in this zone to ensure that the velocities are adequate to prevent settlement.
Bubble air diffusers are installed within this zone to aerate sludge and to prevent the deposition of solids.

The principal functions of the selector zone are:

- To control the development of organisms that can lead to sludge bulking such as Nocardia and Microthrix Parvicella

- To ensure the rapid removal of the readily biodegradable pollution.

For this plant, the total volume of the selector zone will be quite significant due to the high biodegradable pollution (coming from the food industrial wastewater). The volume of the selector zone will be about 1400 m3 in each basin (about 15% of the total volume of the basin).

Recirculation sludge will be mixed with the influent in the first compartment of the selector zone.

As most of the scums and greases will build up into the first compartment of the selector zone, a scum removal system will be installed in this compartment to take them out.

Fill – Aerate phase:

The oxygen required for the biological reaction is introduced during this first phase. The airflow is controlled by a biological control system, which monitors dissolved oxygen, and level within each tank. This approach optimises the microbiology, whilst minimising the process air requirements. The use of fine bubble diffusers ensures an enhanced level of oxygen transfer efficiency.

During this phase, mixed liquor from the aeration zone is recycled into the selector zone in order to maintain an optimum Food/Mass ratio for the selection of floc-forming bacteria.

Aeration capacity will be as follows:

AOR (Actual Oxygen Requirement) = a' x k x kg BOD/d + b' x kgMLSS

With:
a' = kgO2/kgBODin =0.67
b' = kg O2/kgMLSS/d= 0.05
k = peak factor for O2 = 1.25
For the 1.6 *DL condition
AOR =0.67 x 1.25 x 33814 + 0.05 x 67932 m3 x 3.3 kg/m3 = 39528 kg/d

ONDEO Degrémont	FINAL DESIGN – BASIC PROCESS DESCRIPTION	JOB: E601009I SPEC. DA401		
CLIENT:	CORK CITY COUNCIL	Rev. 5		
PLANT:	WASTE WATER TREATMENT PLANT & OUTFALL	sheet n:	18	of 37

SOTR (Standard Oxygen Transfer Rate)

$AOR/\alpha SOTR = \vartheta^{T-20} (\beta \times \tau \times C^*_{\infty 20} - C)/ C^*_{\infty 20}$
$\alpha = 0,65;$
$\beta = 0,98;$
$\vartheta = 1,024;$
$\tau = C^*_{ST} /C^*_{S20}$ @ 20°C = 1; @ 10°C = 1,24

$C^*_{\infty 20}$ =10, 30 mg O_2/l @ average depth of 4,5 mH20;

C = 2.5 mg/l;

Considering T = 10 °C: (AOR/αSOTR) = 0,767

Considering T = 20 °C: (AOR/αSOTR) = 0,737

(AOR/αSOTR) = 0,737

SOTR = AOR /(0,737 x 0,65) = 39629/ 0.479 = 82522 KgO_2/d

Daily aeration time (at WWF)
8 cells, 8.57 cycles (day) x 56 min = 64 h/d;

SOTR for each basin = 82522/64 = 1289.4 KgO_2/h
Average diffusers efficiency = 25,5% (@ 4,5 m depth);
Oxygen content = 0,3 KgO_2/Nm^3;
Air requirement = 1289.4/(0,3 x 0,255) = 16855 Nm^3/h -> 17540 Nm3/h

Blowers needed:
4 cells in contemporary aeration x 17540 Nm^3/h/each.
That is 4 blowers x 17540 Nm^3/h + 2 spare

This blower sizing includes some safety margin and ensures the achievement of performance standards in all conditions.

Settle phase:

At the end of the Fill - Aerate phase, the influent is diverted to another tank and the aeration is stopped. The settling process then proceeds under quiescent conditions.

The proposed design is based on a Sludge Volume Index (SVI) of 140 ml/g with an MLSS concentration at BWL of 4.8 g/l. The maximum sludge blanket level is one meter below the BWL.

The section will be also able to handle a DSVI of 150 ml/g while keeping a minimum clear water depth of 1 m below the BWL.

Calculations are as follows:

- sludge volume after 30 minutes settling = 4.8 x 150 = 720 ml
- considering a water depth at BWL 3.85 m
- level of sludge blanket after 30 minutes settling = 3.85 x 0.72 = 2.77 m
- clear water depth = 3.85 – 2.77 = 1.07 m

For this type of influent, the concentration of the settled sludge blanket will be approximately 7 to 8 g/l.

Decant phase:

The treated effluent is discharged by lowering a 26 m (No. 2 x 13 m each) decant weirs at one end of the tank. These decanters are motor driven from their rest position above the TWL to the BWL position. The decanter arms rest above the TWL during the Fill-Aerate and Settle phases to avoid the risk of sludge deposition in the troughs.

The treated effluent is removed from approximately 200 mm below the liquid surface in order to avoid the collection of scum and/or other floating matter.

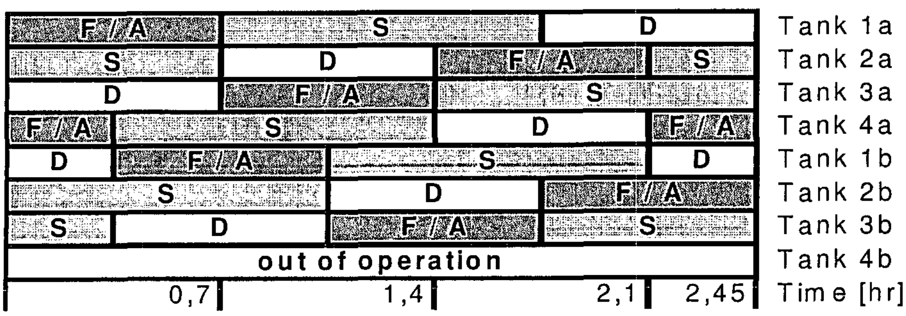
Excess sludge extraction is performed during the latter part of this phase.

Scheduled Emergency and Maintenance cycle:

Emergency & Maintenance Cycle
For maintenance one basin could be put out of operation.
The system acts as 7-basins system, and will be operated as follows:

Maintenance	WWF cycles [hr]
Fill / aeration (FA)	0.7/0.7
Settle (S)	1.05
Decant (D)	0.7
Total time each cycle	**2.45**
Cycles per day	**9.8**

Emergency Cycle 7 basins



Performance standards are also achieved with this emergency / maintenance cycle.

OUTFALL MAINTENANCE

To overcome the problem of saline intrusion at low flow into the outfall diffuser, it is necessary that velocity in excess of 1 m/s should be maintained for a period of at least 15 minutes once a week.

When this will be necessary, the decanter rate will be variable with a greater flow at the start of a cycle that will create velocities in the outfall pipeline necessary for flushing. In this case two SBR cells will discharge together at the beginning of the decant phase the required flow.

Calculation is reported in DA 404 Hydraulic Calculation Report – General - Appendix B.

3.3.2 Main inlet pollution data from the Primary Tanks:

- BOD : from 25,575 kg/day (1.2*Design Load) to 33814 kg/day (1,6*Design Load)
- TSS : from 21347 kg/day (1.2*Design Load) to 27888 kg/day (1,6*Design Load)
- VSS/TSS ratio should be about 79%

3.3.3 Results downstream the SBR:

According to the Works Performance Guarantee:

- Effluent BOD :

- 25 mg/l. Concentration that shall not be exceeded in more than 3 out of 60 consecutive daily samples collected during the Performance Tests.
- 50 mg/l. Maximum concentration permitted.

- 125 mg/l. Concentration that shall not be exceeded in more than 3 out of 60 consecutive daily samples collected during the Performance Tests.
- 250 mg/l. Maximum concentration permitted.

- Effluent TSS :
- 35 mg/l. Concentration that shall not be exceeded in more than 3 out of 60 consecutive daily samples collected during the Performance Tests.
- 87.5 mg/l. Maximum concentration permitted.

Expected outlet TSS concentration is 16 mg/l as an average value.
The results indicated above have been given if Conditions 1,2 and 3 listed in the Works Performance Guarantee are met.

3.3.4 Characteristics of the SBR:

Characteristics of the eight basins:

Characteristics of each basin will be:

- Length = 45.00 m
- Width = 34.00 m
- TWL = 5.5 m / 5.55 m (maintenance)
- BWL = 3.85 m
- Decanting depth = up to 1.70 m
- MLSS TWL = 3.3 g/l
- MLSS BWL = 4.8 g/l
- Organic loading of about 0.15 kg BOD/kg MLSS/day at 1.6*Design Load
- Sludge age = about 10 days (at 1.2 DL)
- Aerated sludge age = about 5 days. (at 1.5 DL and DWF)
- SVI of 140 ml/g

Characteristics of the aeration:

To treat the pollution, the daily standard oxygen requirements (AOR) will be:
- for 1.2*Design Load = 32628 kgO_2/day
- for 1.6*Design Load = 39529 kgO_2/day

The corresponding SOTR will be:
- for 1.2*Design Load = 68118 kgO_2/day
- for 1.6*Design Load = 82524 kgO_2/day

Depending on the type of cycle, the hourly oxygen demand will then be:

- for 1.2*Design Load = 710 kg O_2/h
- for 1.6*Design Load = 860 kg O_2/h

- In case of a 3 hours cycle :
  - for 1.2*Design Load = 1064 kg O_2/h
  - for 1.6*Design Load = 1289 kg O_2/h

Using 0.3 kg O_2/Nm^3 air and a diffuser efficiency of 20 to 30% depending on the water depth (at average depth 25.5%), the air requirements will be:

- For a 4 hours cycle:
  - for 1.2*Design Load = 9275 Nm^3/h
  - for 1.6*Design Load = 11237 kg Nm^3/h

- For a 3 hours cycle and calculated on WWF :
  - for 1.2*Design Load = 13913 Nm^3/h
  - for 1.6*Design Load = 16855 Nm^3/h

Sludge recirculation and extraction:

Proposal is made to install one recirculation pump and one extraction pump per basin instead of one pump doing both as foreseen at tender stage.
First because functions are different and second because it will be more convenient for operation. Characteristics of the proposed pumps will be:

- Recirculation pump : flow of about 450 m^3/h
- Extraction pump : flow of about 150 m^3/h

The recirculation pump has been calculated on a basis of 27% of ADF.

The sludge production will be about 22191 kg TSS/day for 1.2 Design Load and 30'852 kg TSS/day for 1.6 DL. The maximum quantity of sludge sent to treatment - calculated as done for the primary sludge – will be 23634 kg TSS /day.

With a sludge concentration of about 7 to 8 g/l and a working time of about half an hour every cycle, a 150-m^3/h pump is required by basin (at DWF, only 6 cycle).

3.4 Storm tanks:

The Storm Tanks are collecting the influent water up to 2.33 m^3/s from two different locations:

- downstream the inlet works: flows in excess to 2.2 m^3/s going to the Primary Tanks
- downstream the Primary tanks: flows in excess to 1.93 m^3/s going to the SBR

The Specification requires a total storage volume of 13,940 m^3 consequently the design of each Storm Tank will be:

- Storage volume : 3,485 m^3
- Selected dimensions: 33.75 meter diameter * 3.35 m side water depth (which gives a total volume of about 15.400 m^3
- Sloping bottom of about 11.25 degrees

Each Storm Tank will be equipped with one rotating half diameter Scraper Bridge and water cleaning system.

Four actuated penstocks will be provided to allow the storm water to enter in each Storm Tank. These penstocks open automatically in association with the level sensor in the tanks to allow for sequential filling. When all the tanks are filled up and storms are still coming they will start to overflow to the discharge. If one storm tank is closed for maintenance the flow will go through to the other three tanks. When the tanks are emptied it will be possible to clean the bottom and a part of sidewall with a set of sprinklers installed on the scraper and connected to the washwater system.

When the secondary flow will be down to 6,000 m^3/h, and by considering that stored storm water is raw water, it will be pumped back from the Storm Tank Sump to the distribution chamber upstream the Primary clarification as requested in the contract Volume No.4 paragraph 7.5.
The maximum storm water returning flow will be about 600 m^3/h (2 pumps duty and 1 pump standby rated at 300 m^3/h).
The Storm Sludge will be pumped at the beginning of the emptying cycle from the Storm Tank Sump to the Sludge Treatment Thickeners. Two dedicated Sludge Pumps (1 duty and 1 standby rated at 40 m^3/h) will be foreseen in the Storm Tank Sump. Running is pre-set according to average observation. Storm water pumps are started after stopping storm sludge pumps.

FLOWS AND LOADS TO THE SLUDGE TREATMENT

Taking into account sludge removed in primary sedimentation and sludge extracted from the biological treatment we have the following quantities of sludge for the various loads.

		1.0 DL	1.2 DL	1.27 DL
PRIMARY SLUDGE				
Quantity	kg/d	8196	9817	10318
Volume	m3/d	820	982	1032
Concentration	%	1	1	1
BIOLOGICAL SLUDGE				
Quantity *	kg/d	17876	22191	23634
Volume	m^3/d	2235	2774	2954
Concentration	%	0.8	0.8	0.8
RATIO Primary/Biological	%	31/69	31/69	30/70
TOTAL QUANTITY OF SLUGE TO BE TREATED BY THE SLUDGE TREATMENT	kg/d	26072	32008	33952

Note: In order to increase the safety margin on the sludge line sizing, TSS escaping with the final effluent (as mentioned in section 3.3.3) are not subtracted from the sludge production.

3.5 Gravity Picket Fence Thickener for primary sludge:

The primary sludge is pumped from the 2 Primary Tanks directly into the thickeners via 3 progressive cavity pumps (2 duty and 1 standby) having a manual speed variator and a maximum capacity of 45 m^3/h each.

Main characteristics of the Primary thickener unit are:

- No. 3 units of 7.68 meter diameter, cylindrical height 4 m, covered and connected to the odour treatment plant.

Side water depth = 4.0 m.

- Total area = 139 m^2
- Rate of 70.6 $kg/m^2/day$ for 1.2*Design Load.
- Design underflow sludge concentration about 60 g/l.

Thickened primary sludge:

- Design Load : 7786 kg/day => 130 m^3/day at 60 g/l
- 1,2*Design Load: 9326 kg/day => 155 m^3/day.
- Maximum design capacity 1,27*Design Load: 9802 kg/day => 163 m^3/day.

Maximum amount of sludge that can be treated

*Considering the maximum solid loading allowed of 110 $kg/m^2/d$ and total area of the thickener of 139 m^2 the maximum capacity of this section is 15.290 kg SS/d which corresponds to a capacity of 48 % higher than the 1.27 * DL Primary Sludge Production.*

For the pumping of the thickened primary sludge to the Thickened Sludge Holding Tank, No. 3 pumps of about 5 m^3/h are needed:

- For the Design Load, the pumps will be working about 8.7 hours/day
- For 1,2*Design Load, the pumps will be working about 10.3 hours/day

- For 1.27*Design Load, the pumps will be working about 10.9 hours/day

3.6 Secondary Thickening:

The secondary sludge is pumped from the SBR (end of the Decant phase) to the SBR Sludge Holding Tank from whom it is thickened on 3 Gravity Belt Thickeners. After the thickening step, the sludge is sent to the Thickened Sludge Holding Tank.

Amount of secondary sludge to thicken:
- Design Load : about 17876 kg/day => 2235 m^3/day at 8 g/l
- 1.2*Design Load: about 22191 kg/day => 2774 m^3/day at 8 g/l.
- Maximum design capacity 1.27*Design Load: about 23634 kg/day => 2954 m^3/day at 8 g/l.

Since the secondary sludge arrives in batch at the end of the Decant phase, a holding tank to smooth out the quantity of sludge during each batch shall be provided. Every hour, at 1.2*Design Load, there is about 100 m^3 of sludge (max) arriving in about 30 minutes. The holding tank will have a volume of 200 m^3.

According to the Specification:
- Two Gravity Belt thickeners and ancillaries equipment (pumps…) have to be sized to treat the Design Load in 20 hours.
- To treat the peak loads (over the Design Load), the third additional stream will be used. The third additional stream serves as a standby during the Design Load.

For that:
- The 3 feeding pumps to the Gravity Belt Thickeners will have each a capacity of about 65 m3/h (the calculations give 55.9 m3/h but some margin in case of lower sludge concentration is foreseen).
- The three Gravity Belt Thickeners will be sized to handle each 450 kg/h of sludge

Performance of the Gravity Belt thickeners:
- About 95% of solids capture
- Polymer dosage of about 5 kg / ton TSS
- Thickened sludge concentration of about 50 g/l.

With a polymer dosage 5 kg/ton TSS, the four polymer pumps (3 duty / 1 standby) will have each a capacity of about 800 l/h (the calculations give 447 l/h with a polymer preparation at 5 g/l). Polymer is post-diluted to 1 g/l before being mixed with sludge.

With 95% of solids capture and a sludge concentration of about 50 g/l, the Gravity Belt Thickeners will produce:
- Design Load : 16983kg/day of sludge at a concentration of 50 g/l => about 340 m^3/day

– 1,2*Design Load: 21082 kg/day of sludge at a concentration of 50 g/l => about 422 m^3/day.

– Maximum design capacity 1,27*Design Load: 22543 kg/day of sludge at a concentration of 50 g/l => about 449 m^3/day.

Maximum amount of sludge that can be treated
The selected equipment running 24 hour/day with a flow rate of 60 m^3/h and a sludge concentration from 5 to 10 g/l can treat a maximum sludge quantity of 32.400 kg/d.

3.7 Thickened Sludge Holding Tank and Thickened Sludge Storage Tank:

To store thickened primary and secondary sludge (1836 m^3) produced during at least three days, at maximum design load, a volume of 2300 m^3 is proposed.
Equipment selected:

– Thickened Sludge Storage Tank underneath the Sludge Building with total volume of 1900 m^3

– Thickened Sludge Holding Tank with a total volume of 400 m^3

To pump the sludge to the digester, No. 3 pumps of about 10 m^3/h were selected (calculation give 8.5 m^3/h on a 24-hour basis).
Continuous pump running is preferred to intermittent feeding for process reason. Therefore frequency converter according to the level in the Thickened Sludge Holding Tank will control the pump flow

3.8 Digestion system:

The sludge is pumped from the Thickened Sludge Holding Tank to 3 No. Digesters.

Volume = 3*3,400 m^3 = 10,200 m^3

3.8.1 Sizing of the digester:

	Units	1.0*DL	1.2*DL	1.27*DL
Flow	m3/d	469	577	612
Load	Kg SS/d	24769	30408	32255
Concentration	Kg/m3	52.8	52.7	52.7
Inlet ratio VSS/TSS		0.77	0.77	0.77
Digesters volume	m3		3*3400 = 10200	
Retention time	Days	21.7	17.7	16.7
Load	Kg TSS/m3*d	2.43	2.98	3.16
Load	Kg VSS/m3*d	1.87	2.29	2.43

Maximum amount of sludge that can be treated
Considering the minimum retention time of 14 days allowed in the contract the treatment capacity of the digestion will be 720m^3/d.

Digesters heating

An optimum temperature of 35°C has to be maintained inside digesters, for high kinetics of VSS reduction without destroying anaerobic bacteria.

Therefore heat is transferred to avoid cooling by fresh thickened sludge feed flow and natural heat dissipation.
Considering a specific heat of 1 kcal / kg °C, requirement for fresh sludge at 10°C is:

- 1*Design Load : 569 kW or 11745 Mcal/d
- 1.2*Design Load : 699 kW or 14428 Mcal/d
- 1.27*Design Load : 742 kW or 15315 Mcal/d

Heat losses at 0°C are estimated at 69 kW (1424 Mcal/d).

Digesters are heated either by the low grade heat recovered from the drying plant, or by a dedicated boiler (or by a combination of these two heat sources).

3.8.2 Sizing of the mixing:

The mixing will be done using biogas; one compressor rated 465 Nm3/h will be provided for each digester.
Total mixing power is 31700 / 3400 = 9.3 W / m3
Power dissipated through gas expansion is 5.2 W / m3.
Mixing method: spiral flow.

3.8.3 Sizing of the gas holder and the flare:

The amount of biogas produced by the digestion system is linked to the VSS reduction.

An increasing of the capability of the gas section is proposed according to expected possible value of about 42% VSS reduction.
The total reduced VSS will then be between 7997 kg/day (Design Load) to 9803 kg/day (1.2*Design Load), to 10394 kg/day (1.27*Design Load).

With a specific biogas production of about 0.9 Nm^3/kg VSS reduced, we will have respectively 7197 to 8823 to 9354 Nm^3/day of total biogas production.

Compared to the tender value 30 % of VSS reduction, the production of biogas will be higher (7197 Nm^3/d at Design Load compared to 5,800 Nm^3/d in the tender). Flare size shall be consequently increased compared to the tender size.

- Gas flare proposed capacity : 600 Nm^3/h instead of 380 Nm^3/h as foreseen at tender stage
- In order to prevent too much switching from biogas to natural gas the gasholder capacity is defined considering a minimum retention time of 5-6 hours shall be considered (as per standard Ondeo Degrémont sizing) consequently the proposed capacity is 2150 m^3.

3.8.4 Characteristics of the digested sludge:

	Units	1.0*DL	1.2*DL	1.27*DL
Flow	m3/d	469	577	612
Load	kg SS/d	16772	20604	21861
Concentration	kg/m3	35.7	37.5	37.5
Outlet ratio TSS/VSS	%	65.8	66	66

3.9 Digested Sludge Holding Tank – Digested Sludge Storage Tank:

Specification require 5 days available storage based on maximum flow (plus storage Capacity required for normal operation).
We will provide a 330 m^3 digested sludge holding tank and a 2900 m^3 digested sludge storage tank. Total capacity 3230 m^3 equivalent to a retention time of 3230/612 = 5,27 d.
As storage volume for normal operation we have considered 612/24 = 25.5 m3/h
25.5 x 5 hours = 127.5 m^3

An additional storage of about one-day has been provided between the dewatering system and the dryer system.

3.10 Dewatering:

The digested sludge is pumped from the Digested Sludge Holding Tank or the Digested Sludge Storage Tank to the Belt Filters. After the dewatering, the sludge will then be stored in a Dewatered Silo prior to the drying system.

Amount of secondary sludge to dewater:
- Design Load : about 16772 kg/day => 469 m^3/day at 35.7 g/l
- 1.2*Design Load: about 20604 kg/day => 577 m^3/day at 35.7 g/l.
- Maximum design capacity 1.2*Design Load: about 21861 kg/day => 612 m3/day at 35.7g/l.

According to the Specification:
- Two Belt Filters and ancillary equipment (pumps…) have to be sized to treat the Design

- To treat the peak loads (over the Design Load) a third additional stream will be used. The third additional stream serves normally as a standby.

For that:
- The 3 feeding pumps to the Belt Filters will have a capacity of about 15 m^3/h each; 11,73 m^3/h will be necessary and a margin in case of lower sludge concentration has been considered.
- The three Belt Filters will be sized to handle each 420 kg/h of sludge
- The three outlet pumps downstream the Belt Filters will have each a capacity of about 3 m^3/h (the calculations give 1.73 m^3/h).

Performance of the Belt Filters:
- About 95% of solids capture
- Max polymer dosage about 6 kg / ton TSS
- Max Dry solid content from 20 to 23%.

With a 6 kg / ton TSS polymer dosage, the four polymer pumps (3 duty / 1 standby) will have each a capacity of about 800 l/h (the calculations give 503 l/h with a polymer preparation at 5g/l). Polymer is post-diluted to 1 g/l before being mixed with sludge.

With 95% of solids capture and a maximum design dryness of 23 %, the Belt Filters will produce:
- Design Load : 15934 kg/day => about 69.3 m^3/day
- 1.2*Design Load : 19574 kg/day => about 85.1 m^3/day
- Maximum design capacity 1.27*Design Load : 20768 kg/day => about 90.3 m^3/day

Maximum amount of sludge that can be treated
The selected equipment running 24 hour/day with a sludge concentration of 35,7 g/l with a flow rate up to 11,7 m^3/h can treat a maximum sludge quantity of 30.000 kg/d.

3.11 Dewatered Sludge Storage Silo:

A 75 m^3 Thickened Sludge Storage Tank will be provided in order to have about one day retention time. This silo is equipped with a sliding frame to feed the Dryer Feed Pumps.

Capacity of the sliding frame: about 5 – 6 m^3/h (the calculations give 3.76 m^3/h on 1.27*Design Load).

3.12 Drying system:

3.12.1 Brief description of the drying system:

The dewatered sludge is pumped from the Dewatered Sludge Storage Silo and conveyed to the inlet of the sludge dryers. Dryers and all their allocated equipment are located in the Sludge Building.

Two dryers are provided, with a maximum evaporation capacity of 1350 kg/h each.
Each dryer is composed of the following stages:

- A thin film evaporator which brings sludge dryness from 23% to 43% (still in sticky phase)
- A chopper that transforms bulky sludge into strings with high specific surface.
- A belt dryer where sludge strings are dried from 43% to 90%, without friction or shocks. Strings are cooled to 40°C by a closed air loop in the last section of the belt dryer.
- A crusher to break strings at the required granulates size.

Granulates are conveyed by a bucket elevator to a 30 m3 storage silo which can either fill bags or directly a truck. No dust is generated by this system

Thin film evaporators (first stage) are heated by thermal oil (primary circuit).
Belt dryers (second stage) are heated by hot air in closed loop.
Hot air and saturated water vapour at +/- 70°C from stage 2 are cooled to +/- 62°C in a condenser where heat is transferred to the digester loop (first recovery).
Air is then heated to 88°C by the condensation of first drying stage vapours. Resulting condensates exchange heat with the digester loop (second recovery).
Before returning to belt dryers, recirculated air is heated to 100°C by thermal oil (secondary circuit).
A vent valve located upstream the latter heating allows negative pressure in the system.
Vented air and vapour at 88°C (+/- 2000 m3/h total) are dust free and connected directly to the odour treatment, where they are diluted and cooled by other air flows.

3.12.2 Feed pumps:

Three feed pumps will be provided: 2 duty / 1 standby.
Pumps can feed dryers or directly a truck.
The capacity of each pump will be higher than the maximum feed flow to each dryer, Pumps will have variable speeds to suit the incoming sludge flow and ensure continuous drying, which is quite important.

The quantity of sludge to dry is 663 kg SS/h for the Design Load and 816 kg SS/h for 1.2*Design Load and 865 kg SS/h for Maximum design capacity 1.27*Design Load.

Moreover dryers are designed to receive 125% of the contractual sludge production at 1.2 Design load: maximum feed is 1030 kgSS/h (for dewatered sludge at 23%).

3.12.3 Dryers:

To get a dried sludge with 90% of dry solid content from a dewatered sludge of 23%, we need the following capacity to evaporate water:

- Design Load 15934 kg/d: water to be evaporated 51572 kg/d (2150 kg/h)
- 1.2*Design Load 19574 kg/d: water to be evaporated 63355 kg/d (2640 kg/h)
- 1.27*Design Load 20768 kg/d: water to be evaporated 67220 kg/d (2800 kg/h)

Characteristic of the dried sludge:

- Dewatered sludge solid content: 23%
- Dried sludge solid content: 90%
- Dried product range = 2 – 4 mm (95%); < 0.5 mm (2%); 0.5 – 2 mm (2%);
- > 4 m (1%)
- Product bulk density = about 700 kg/m^3

Due to the selected process, TSS losses in condensates and air vents are negligible. TSS flows in inlet dewatered sludge and outlet granulated are assumed as equal.

So granulate production is:

- Design Load 15934 kg/d: 25.3 m3/d
- 1.2*Design Load 19574 kg/d: 31.0 m3/d
- 1.27*Design Load 20768 kg/d: 32.96 m3/d

Maximum amount of sludge that can be treated

*As per specification, dryers must have a capacity of 1.25 times the maximum daily sludge production, which has been agreed to be 1.2 * DL. So maximum evaporation capacity will be 3300 kg/h, and related inlet sludge flow will be 24720 kg TSS / d.*

Heat balance:

The selected technology involves low energy consumption as part of the heat required by the second stage comes from the condensation of first stage vapours.

As a consequence, low grade heat available for digesters' loop is available at a lower temperature than other kinds of dryers where all the water is evaporated by direct heating.

To enable heat recovery anyway, one common hot water loop is proposed, flowing from digesters to dryers.

Dryer heat data are (with thermal oil boiler efficiency: 87%)

	units	1.0*DL	1.2*DL	1.27*DL
Net absorbed heat	kW	1319	1619	1717
Heat losses (belt dryers)	kW	62	62	62
Power by boiler	kW	1587	1932	2045
Heat recovery	kW	919	1128	1196
ΔT water loop	°C	6.32	7.76	8.23

Assuming a lower heating value of 20000 kJ/Nm3, biogas consumption amounts:

- 6857 Nm3/h at 1*DL
- 8347 Nm3/h at 1.2*DL
- 8834 Nm3/h at 1.27*DL

In normal operating conditions, the energy recovered from dryers is higher than the energy needed by digesters, and available water temperatures make heat exchange with digested sludge possible: for a constant water circulating flow of 125 m3/h, the water temperature at the digester inlet is 53.3 °C at 1*DL, 54.7 at 1.2*DL and 55.2 at 1.27*DL.

The difference between the heat recovered from dryers and the heat transferred to digesters is eliminated through exchange with cool effluent water, which then joins return liquors. This cooling effluent water flow is the maximum one when dryers and digesters are disconnected. So effluent flow for dryers' cooling, with an initial temperature of 15°C and a final temperature of 35 – 40°C can be, at 1.27*DL, as high as 1498 m3/d and as low as 293 m3/d when the hot water loop is operated.

3.12.4 Dried Product Transfer to Storage Silo:

The Dried Product will be conveyed to Storage Silo with a bucket elevator.

The capacity of the conveying system will be about 3 m^3/h (the calculations give about 1.63 m^3/h of bulk product).

3.12.5 Dried Sludge Storage Silo:

The Dried Sludge Storage Silo is provided as a buffer between the drying system and the Bagging Plant.
The operating volume of this silo will be 35 m^3.

3.13 Imported sludge:

The proposed design would be amended as follows in the event of future importation of thickened sludge (primary / biological) from external plants.

A daily sludge quantity of 2400 kg/d with an average solid content of 5% is specified, that means 48 m3/d.
Assuming 2 tankers with a capacity of 10 m3 each, discharging together during 30 min, the maximum sludge flow is 40 m3/h.

Tankers are directly connected to a pump (rated 20 m3/h) via a quick coupling and a hose. Sludge is pumped to the inlet flange of a "strain press" well adapted to sewage sludge screening.

The press mainly comprises a screw; a 5 mm perforated mesh screen and a retention cone for solid discharge regulation. The screw conveys sludge along the screening section; liquid matter crosses mesh openings whereas particles larger than 5 mm are retained, and transported by the screw to the pressing zone. Screened fluid is discharged from the machine by a flanged connection. Remaining material is compacted in the continual action of the pressing zone screw. The retention cone situated at the end of the compaction zone controls the discharge of solid matter. Further information on the "strain press" can be provided upon request.

The "strain press" is located on a high floor, so that screened sludge can be discharged by gravity into the existing sludge holding tank (22).

A sump located below the hose / quick coupling area collects lost sludge, which are directed back to the tanker by a submersible pump.

The main scope of equipment would include 3 sludge pumps 20 m3/h / 2 bar (2 duty / 1 standby), 2 strain presses (1 duty / 1 standby), 1 submersible pump, related pipes, valves, control equipment. One building would be added next to sludge holding tank 22, so as to take advantage of the existing access road area. As strain presses are closed machines, no significant odours are expected to arise. However the screening skip needs deodorization: either by connecting the complete building to the treatment or only a separate isolation room.

No additional equipment are necessary to treat the imported sludge, provided following data are deemed acceptable assuming that daily load of 2400 kg TSS/d is added to the maximum sludge quantity (1.27 DL).

1 Thickened sludge storage
Storage time is 3.51 days, considering 400 + 1900 = 2300 m3 capacity.

2 Sludge Digestion:
The required retention time of 14 days is still met but the safety coefficient for Flare capacity

3 Dewatering:
Using 3 Belt Press, the unit solid load 195 kg/m * h is lower than the maximum design value 210 kg/m * h. Polymer preparation and dosing facilities also allow dewatering the additional load.
New storage time in the silo is now 0.7 day

4 Drying:
The total amount of water to evaporate 2999 kg/h is lower than the maximum capacity of dryers. All conveyors and ancillaries accept the increased quantities.

Note: The maximum contribution of imported sludge to the return flows is 323 m^3/d, which will have no detrimental effect on process neither on hydraulics.

3.14 Sludge Liquor Return Pumps:

The liquors coming from the sludge line will be pumped in front of the Primary Tanks since they contain significant TSS.

Liquors are coming from:
- Primary Thickening
- Secondary Thickening
- Dewatering
- Drying

3.15 Flows and loads at maximum capacity

PRIMARY THICKENING

- TSS load = 516 kg/day (95% of efficiency => 5% of 10318 kg/day)
- Volume of water = average 868 m3/day

SECONDARY THICKENING

- TSS load = 1182 kg/day (95% of efficiency => 5% of 23634 kg/day)
- Volume of water = average 3323 m^3/day (daily inlet flow – daily extraction flow + belt wash-water)

DEWATERING

- TSS load = 1093 kg/day (95% of efficiency => 5% of 21861 kg/day)
- Volume of water = average 1818 m^3/day (daily inlet flow – daily extraction flow + belt washwater)

DRYING

- TSS load = 24 kg/day (TSS from the final effluent used for cooling)
- Volume of water = 1498 m^3/day (final effluent cooling water + condensate)

OTHER FLOWS (hosedown, overflows, hydraulic seals, ...)
- Volume of water : 575 m^3/d
- TSS load: 585 kg/d

3.13.5 Return liquors sump and pumps:

Total amount of liquors to return in front of the Primary Tanks when the sludge treatment is running at maximum capacity is:

Average volume of water = 8264 m^3/d
Peak volume =8640 m^3/d
TSS load = 3400 kg/d
BOD load = 1360 kg/d
COD load = 6800 kg/d

It is proposed to provide 3 pumps of 180 m^3/h capacity (2 duties / 1 standby) instead of the 2 pumps of 70 m^3/h which were indicated in the tender.

A volume of 100 m^3 would be enough to prevent too many startings of the pumps.

4 Control Odour System

Will be installed three different Control Odour Systems to treat the polluted air flows coming from three different zones of the plant:
- Preliminary Treatment – System to be installed in the Screen & Grit Removal House
- Sludge Treatment – System to be installed in the Sludge Building
- Primary Settlement Tanks - System to be installed outdoor near the Primary Tanks flow distribution chamber

In these zones air is contaminated by several polluting compounds, including H_2S, mercaptans, NH_3, volatile fatty acids. In order to reduce the volume of air to be treated, the mentioned areas are either covered or placed inside a building.
Here below are listed the different areas to be deodorized within each of the above mentioned zones:

1) Preliminary Treatment
- screening units;
- grit removal unit;
- skip area;

- pre-aeration tank;

2) Sludge Treatment
- Picket fence thickeners
- Thickened sludge holding & storage tanks,
- Digesters' inlet & outlet chambers,
- Digested sludge holding & storage tanks;
- Sludge liquor return tank;
- SBR sludge holding tank;
- Gravity Belt Thickeners;
- Belt Presses;
- Dewatered and dried sludge storage silos;

3) Primary Settlement Tanks

In each zone air flows are driven out of the covered tanks and buildings by dedicated fans and are conveyed to the wet scrubbers wherein the abatement of the polluting agents occur. Each odour treatment system will be three stages wet scrubbers (three treatment towers).
In the first stage a sulphuric acid solution is used to remove NH_3.
In the second stage a sodium hypochlorite solution is used to eliminate H_2S and CH_3SH.
In the final stage a caustic soda solution is used to eliminate any volatile fatty acid.
The various dilute scrubbing solutions will be continuously recycled to the scrubbers by means of centrifugal pumps. Each scrubber system will be fully automatic.

4.1 Performance Guarantee

The odour scrubbing equipment will be designed so that, during operation of the Plant, the treated air discharged from the ventilation stack will not increase the short term average TON (as measured using the procedure developed by CEN TC 264 Working Group 2) by more than 5 TON, at any receptor position outside the site and anywhere on the boundary of the Site.
The maximum allowable odour emission rate, E (OU/s), in the stack will be converted to a short-term hydrogen sulphide concentration, C_s (ppb), in the stack gas using the following formula:

$$C_s = C_t\,(E/UK) \qquad (1)$$

Where
C_t = the threshold concentration of hydrogen sulphide, which will be 0,5 ppb;
U = the flow rate of the air from the stack (m^3/s);
K = the ratio of the total TON of the stack air to the TON contributed by H_2S in the stack air, usually K = 5.
The short-term concentrations of hydrogen sulphide in the stack gas will be automatically and continuously monitored and periodically recorded. The upper 98-percentile value of these readings will be less than the value of C_s calculated from the formula (1).
Within the treatment buildings the H_2S concentration in the air at a height of 1 to 2 m above the floor level have to be:

- less than 50 ppb during normal operation in any building other then sludge cake enclosure;
- less then 200 ppb at any location during emergency operation;
- less then 400 ppb at any location of sludge cake enclosure.

Within personnel areas the H_2S concentration has to be always less then 2 ppb.
Considering the odour modelling submitted at Tender stage, the maximum 1 hour concentration at Site boundary and at the closest resident due to combined emission from the three sources are the following:

	Max 1-hour Odour Conc. At Boundary (OU/m^3)	Max 1-hour Odour Conc. nearest Resident (OU/m
Hydrogen Sulphide	448.84	139.93
Ammonia	0.0495	0.0126
Methyl Mercaptan	312.14	75.65
Total Odour	761.029	215.59

Scrubbers will allow obtaining odour concentrations less than 5 OU/m^3 above ambient levels at the boundary and at the nearest resident.

5 Mechanical Specification:

Detailed mechanical data sheets for all equipment are included in the design report related to each particular section.

6 Explanation of main abbreviations used in this document

DWF	Dry Weather Flow
ADF	Average Daily Flow
ADFP	Average Daily Flow Peak
WWF	Wet Weather Flow
BOD	Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
TSS	Total Suspended Solid
DL	Design Load
VSS	Volatile Suspended Solid
ISS	Inert Suspended Solid (TSS=VSS+ISS)
MLSS	Mixed Liquor Suspended Solids
SVI	Sludge Volume Index
DSVI	Decanted Sludge Volume Index
SBR	Sequencing Batch Reactor
TWL	Top Water Level
BWL	Bottom Water Level
AOR	Actual Oxygen Demand