This Feasibility Study is intended to outline the proposal for the new Bellozanne Sewage Treatment Works (STW) to replace the existing STW to meet the needs of the Island over the next 50 years.

This report identifies the preferred treatment process and appropriate level of treatment together with an evidence based policy approach for effluent quality. It also provides a summary of the Feasibility Design and confirms the funding requirements for implementation.

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EXECUTIVE SUMMARY

The Island has a continuing and ongoing need for disposing of its liquid waste. The existing Bellozanne Sewage Treatment Works (STW) is at the end of its original design life and now requires replacement despite ongoing capital maintenance. It is unable to meet modern day environmental standards due to design capacity limitations, including technology and loading. This report presents a plan for the replacement of the Bellozanne STW liquid stream and rehabilitation/ partial replacement of the effluent outfall into St Aubin's Bay to meet the needs of the Island for the next 50 years.

Studies have been completed to determine the best location and technology to meet the island's requirements over the next fifty years and the favoured solution is a conventional STW located at Bellozanne. A review of the projected population has been undertaken to assess the likely future population connected to the works. This review has provided the basis for the design flow and load to the works. In addition data collected at the works in recent years has been utilised to determine the likely minimum biological load and concentration and the turndown required of the plant during normal operation.

The new site layout is based on carbonaceous removal to achieve a BOD/ SS standard for a connected population equivalent of 118,000 for the design horizon of 2035. The overall site layout makes provision for potential future secondary treatment upgrades such that the requirement for an additional ammonia standard or even total nitrogen standard can be incorporated. Furthermore, the proposals would be able to cope with an increase in the flow and load by 20% for the ultimate scenario.

The land available for the construction of the new STW, while the existing STW continues to remain in service, is limited and this report discusses the demolition and site clearance required for the new works. This includes items such as the Energy from Waste Plant which is currently being demolished and funded separately and the Clinical Waste Incinerator that is programmed to be relocated under this project. The Public Waste Reception Facility and Scrapyard will also be relocated off site as part of the Bellozanne Waste Management Services site development to accommodate part of the new STW.

Based on the historical background of the site's industrial use, there is potential for ground contamination and therefore clean-up / site remediation will be required prior to the change in land use associated with any new construction.

With the proposed layout of the new STW, the vast majority of the services will be required to be relocated or re-routed while the existing facilities continue to remain in service and be phased to suit the construction activities. It is therefore imperative that various construction activities are programmed to minimise any disruption to the activities on the Waste Management Services site.

Once hillside excavation and stabilisation, demolition, relocation and diversion of services and site remediation activities have been completed, works can begin on the construction of the new STW which requires a phased construction to ensure that the existing works continues to remain in service at all times.

It should be noted that the proposed design of the new STW takes into account that an extension to the existing outfall will not be required on the basis of the assimilative capacity of the receiving waters resulting from evidence based policy to be agreed with the Regulator. The condition survey and data collection to model the catchment and existing outfall will continue in order to determine the capacity and suitability for its continued use as an effluent outfall. Any required rehabilitation works will be carried out and any hydraulic restrictions identified will be rectified to make the outfall viable for the long term needs.

The proposed works are programmed to start in July 2013 for completion by October 2018. The estimated total project cost, including the relocation of the Clinical Waste Incinerator, is £75m based on the 2012 prices. A total allowance of £18.3m is included in this estimate for Transport & Technical Services Costs, Professional Fees and Contingencies with varying level of contingencies on work elements at this stage to reflect the current level of understanding and the areas considered to have the most risk.

The Feasibility Study has included an assessment of what could be done to upgrade the new works in the future if a tighter discharge consent is applied. In order to achieve a Total Nitrogen Standard the estimated total project cost for the future secondary treatment upgrades, including engineering and contingencies, is £30.8m based on the 2012 prices.

1. Introduction

This report presents a plan for the replacement of the Bellozanne Sewage Treatment Works (STW) and rehabilitation/ partial replacement of the effluent outfall into St Aubin's Bay to meet the needs of the Island for the next 50 years.

Transport & Technical Services Department (TTS) are currently failing the discharge consent for the Bellozanne STW due to design capacity limitations. The existing STW is also nearing the end of its original design life and now requires replacement despite ongoing capital maintenance. These have been detailed in the following reports:

1. Liquid Waste Strategy (January 2009)
2. Bellozanne Master Plan for the STW (July 2009)
3. Liquid Waste Strategy (May 2010)
4. Bellozanne STW Operation Strategy (May 2011)

Development of these reports included engagement with key States stakeholders including Environmental Protection, Development Control, Building Control, Environmental Policy and Health Protection services. Options for the location of the new STW have previously been considered in these reports and Bellozanne has been agreed as the preferred location, as recommended.

The treated effluent from the Bellozanne STW is currently discharged via an outfall into St Aubin's Bay near the First Tower area at a distance of 0.5km from the seawall. The following report assessed the relative impacts of effluent on water quality in St Aubin's Bay and beyond from the discharge at existing and various potential locations.

5. Outfall Assessment (Desk) Study (April 2010)

TTS have been in discussion with the Department of the Environment (DoE) regarding the effluent quality from Bellozanne STW for many years. As part of the agreement with the Regulator TTS recently commissioned and completed a report to consider and recommend the most appropriate technology for the treatment of the Island's liquid waste. The following report concluded that a conventional STW was the most appropriate technology.

6. Bellozanne STW Best Available Technology Report (May 2012)

Following discussions in June 2012, TTS and the DoE have now agreed that the existing Bellozanne STW will never be able to achieve the required environmental standards for the current and future population until it is replaced in its entirety.

The master plan developed for the delivery and treatment of liquid and solid waste for the next 50 years generally sees liquid waste being received and treated at Bellozanne and solid waste being received and treated at La Collette. This splitting of existing waste activities has already begun with the new Energy from Waste (EFW) Plant being relocated to La Collette and the new sludge treatment facilities being constructed at Bellozanne. Future plans see the existing STW being replaced at Bellozanne to include land and service rationalisation, and clinical waste disposal and public solid waste reception sites moving to La Collette.

TTS have been in detailed discussions with the DoE regarding the appropriate level of treatment, the preferred treatment option and in particular, the future discharge consent. These discussions are now concluded and the Regulator has agreed on the treatment process on the basis that there is adequate capacity in the design together with expansion capability on the site should an enhanced treatment standard be required in the future.

With the identification of need for and location of the new STW at Bellozanne, concept work for the design and layout of the new STW, based on the agreed treatment process, has been completed. This work has demonstrated that it is possible to construct a new STW on the site with the required space for potential future expansion. However, in order to achieve this there is a requirement to relocate existing services to La Collette (clinical waste and public solid waste reception sites) and to increase the footprint of the site by excavating and stabilising the valley sides. This expansion has an associated risk of requiring land acquisitions of both States and private land if ground conditions dictate the valley sides cannot be made steep enough. The practicality of these earthworks is a key consideration and is discussed in detail later in this report. Another consideration is the challenge of phasing the works such that the existing STW remains operational during all stages of the construction.

The Business Case was submitted to identify and bid for funding for the replacement of the STW and its approval has been received (MD-T-2012-0067 dated 02 August 2012) to move forward in the process. The replacement of an asset of the size of the STW requires careful planning and project management and the remaining key stages to this process are listed below.

1. Complete a detailed feasibility study to confirm concept and funding request
2. Complete detailed design and gain Planning Approval for the new STW
3. Tender the works, appoint a suitable contractor and construct the new STW

Within the Business Case there is a requirement to move the concept design to a completed feasibility study. This feasibility study has been undertaken by Transport & Technical Services in order to:

1. prepare feasibility design and finalise the new STW layout to replace the existing STW;
2. identify requirements for land acquisitions of both States and private land for the long term needs of the Island;
3. determine any requirements for phasing the works such that the existing STW remains operational during all stages of the construction;
4. assess the suitability of the existing effluent outfall for its continued use;
5. determine the Planning and Environmental requirements for the proposed works;
6. provide recommendations to meet the needs of the Island for the next 50 years;
7. develop an implementation programme for the recommended works; and,
8. provide cost estimates for the recommended works to seek funding approval.

The outcome of the Feasibility Study is to have a better confidence level in terms of the scope and the cost estimates.

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2. Existing Sewage Treatment Works and Effluent Outfall

Up until the 1950s, untreated sewage was discharged directly onto beaches causing public health concerns. It was therefore decided that the system should be re-built to modern standards and that all sewage should be treated before being discharged to the sea. Bellozanne Valley was an obvious choice for locating the sewage treatment works as, at that time, it was a relatively remote area and was the natural centre of the Island's drainage system.

Bellozanne STW was commissioned in 1959 as a Conventional Activated Sludge Plant (ASP) and designed to provide full treatment to produce an effluent to Royal Commission standards (30mg/l SS and 20mg/l BOD_5) for a population of 57,000. Through the years it has been continually improved and upgraded to take into account the change in flows, increased environmental standards and modern process technology. The existing sludge digestion plant at the Bellozanne STW was constructed in 1987 and is currently in the process of being replaced. Presently, Bellozanne STW treats flows from an estimated connected resident population of 85,136[1] with an estimated summer peak of 101,795[2].

The Discharge Permit (DC2999/07/01), which comes under the Water Pollution (Jersey) Law, 2000, requires the annual average total nitrogen concentration to be less than 10mg/l and suspended solids less than 35mg/l (on a 95 percentile basis).

In the early 1990s, it became evident that the sewage treatment works needed to be upgraded to increase its capacity and replace some of the outdated/ inefficient equipment. TTS also made the decision to install Ultra-violet (UV) disinfection at the STW to reduce bacteria levels in the effluent; this was the first of its type in the British Isles. The UV plant installed in the early 1990's was upgraded in 2003 with self-cleaning and more energy efficient units with applied dose monitoring facilities.

Based on an initial water quality survey carried out by the Centre for Research into Environment and Health (CREH) on the Trophic Status of St Aubin's bay in 1997, it was noted that St. Aubin's bay displayed some evidence of eutrophication in the nearshore area and potential for eutrophication in the bay itself. The report also noted that the nutrient removal from the Bellozanne STW effluent would be a *prudent precautionary step*. However, the report identified the environmental status of the St Aubin's bay as inconclusive based on the limited survey and noted that the time constraints necessitated by the decision timescales for infrastructure investment at Bellozanne STW had not allowed a protracted, but possibly more prudent, data acquisition.

On the basis of the CREH Report on the Trophic Status of St Aubin's Bay (November 1997), it was agreed that the planned upgrade of the ASP should include a nutrient removal process that would decrease the amount of nitrogen entering St Aubin's Bay. Prior to the full implementation of the Water Pollution (Jersey) Law, 2000 on the 27 November 2000, the Public Services Committee issued a Discharge Certificate for Bellozanne STW in favour of itself which stipulated a stringent effluent quality for total nitrogen of no more than 10mg/l for a population equivalent of more than 100,000

[1] Based on Jersey 2011 Census population of 97,857 with 87% connected to the sewerage system

[2] Based on Population Forecast extrapolating from 2011 Census and 2010 Tourism Report

and no more than 15mg/l for a population equivalent of less than 100,000. The Discharge Certificate also contained a relaxed set of conditions until 31 December 2001 and was extended for additional periods of time during construction.

Due to the valley restricting available construction land, there was insufficient aeration volume to achieve nitrification/ denitrification using the **Conventional ASP** as established in 1959. Instead, a new technique proposed by Degremont was used in providing a fixed film media within the aeration zones for organisms to grow and permit full nitrification (**Enhanced ASP**). In addition, four new final settlement tanks were constructed for the enhanced process. At the time the new plant was installed, it was the only full scale example in Western Europe to use this new technique.

The final effluent from the works is discharged via an outfall into St Aubin's Bay near the First Tower area.

2.1 Existing Sewage Treatment Works

All flows to the existing Bellozanne STW as shown in Figures 2.1.1 & 2.1.2 (Appendix B) receive some form of treatment. Flow to full treatment (FFT) receives the following stages of treatment:

- **Preliminary Treatment**, comprising screening and aerated grit & grease removal.
    - *Two mechanically raked bar screens (duty/assist) are provided for the removal of coarse and settleable solids.*
    - *Three grit and grease removal channels are provided in an aerated tank. Grease removal tanks in the inlet works remove the fats, oils and grease coming to the works using dissolved air flotation.*

- **Primary Treatment**, comprising removal of settleable solids.
    - *Four circular primary settlement tanks are provided.*

- **Secondary Treatment**, comprising **Conventional ASP** which has been retrofitted since 2002 to form a high rate process (**Enhanced ASP**).
    - *There are three activated sludge lanes with anoxic and high rate aeration zones followed by twelve final settlement tanks which clarify the effluent from the aeration process prior to discharge to sea via the UV plant.*

The current FFT is approximately 600 l/s as a result of the capacity limitation of the ASP. Storm flows in the range 600 to 1100l/s receive preliminary and primary treatment. The existing inlet works and primary settlements tanks are designed treat this peak flow with the exception of the inlet screens. However, these additional capacities cannot be utilised due to the potential for premature overflow at the downstream of the primary settlement tanks.

All flows up to 1100l/s then combine to receive tertiary treatment in the form of ultraviolet disinfection by a proprietary system prior to discharge to St. Aubin's Bay.

Current key operational issues for the **Enhanced ASP** for compliance with the Discharge Permit (DC2000/07/01) are as follows:

- The **Enhanced ASP** has never been able to achieve consistent total nitrogen requirement with the total nitrogen level averaging 25.8 mg/l between 2003 and 2010 for the **Enhanced ASP**, primarily due to limited anoxic tank volume and the level of nitrification achieved in the aeration zone;
- Formation of microthrix and Nocardia filamentous organisms cause extensive foaming and solids carry over in the final effluent;
- Nature of the suspended solids in the final effluent from the **Enhanced ASP** produces lower kill from the UV disinfection process;
- The high rate process results in excessive energy costs (larger carbon footprint) and health & safety issues.
- This process has the side effect of creating 'sewage foam' in the plant;
- The requirement for recycling of mixed liquor for the denitrification process places a hydraulic restriction on the FFT to 600l/sec prior to storm overflow; and,
- Hydraulic distribution to the final settlement tanks is poor leading to uneven flow splitting and overloading of some tanks.

2.2 Existing Effluent Outfall

The effluent from the Bellozanne STW is currently discharged via an outfall into St Aubin's Bay near the First Tower area at a distance of 0.5km from the seawall as shown in Figure 2.2 (Appendix B). The outfall also receives flow further upstream from the stream and other drainage in the Bellozanne Valley.

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3. Population Projections

3.1 Overview of Approach

The industry standard for developing population forecasts is to adopt a component–based approach either using census–based or policy–based forecasts. This approach has been adopted in the UK water industry for both water and wastewater supply-demand planning and further details can be found in the Environment Agency's Water Resource Planning Guidelines (Environment Agency, 2011[3]) and UKWIR's Long Term/ Least Cost Planning for Wastewater Supply-Demand (07/RG/08/2)[4]

The population projections presented in this Section are based on the States of Jersey Statistical Unit's model developed in 2009; the details of which can be found in the report entitled Jersey's Population Model[5]. The review presented in this Section uses this as a basis of the revised forecast and makes adjustments for the following information which has recently become available:

- States of Jersey Census Report 2011[6]
- States of Jersey Island Plan 2011[7]
- States of Jersey Annual Tourism Report 2010[8]

Ideally updates to the forecast should be undertaken using a component – based approach such as those mentioned above.

In addition to the population projections completed in May 2012, a further review has been carried out based on "2012 Jersey population projections" by States of Jersey Statistics Unit dated September 2012 and summarised in Section 3.6. Further potential future scenarios have been reviewed summarised in Section 3.7.

3.2 Current Population

Population statistics from the 2011 Census, released by the States of Jersey Statistics Unit, indicate that the resident population is 97,857. In 2010 Tourism brought in 556,860 visitors with a further 128,380 business and education related visitors, resulting in a total of 685,240[9] visitors for the year.

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3 Environment Agency Water Resource Planning Guidelines. April 2011.

(http://publications.environment-agency.gov.uk/PDF/GEHO0411BTWD-E-E.pdf)

4 UKWIR Long Term/ Least Cost Planning for Wastewater Supply-Demand (07/RG/08/2) 2007.

5 Jersey Population Model April 2009. http://www.gov.je/Government/Pages/StatesReports.aspx?ReportID=235

6 States of Jersey 2011 Census

http://www.gov.je/GOVERNMENT/CENSUS/CENSUS2011/Pages/2011CensusResults.aspx

7 States of Jersey Island Plan 2011.

http://www.gov.je/PlanningBuilding/LawsRegs/IslandPlan/IslandPlan2011/Pages/index.aspx

8 Jersey 2010 Tourism Report http://www.statesassembly.gov.je/AssemblyReports/2011/38806-41867-2272011.pdf

9 Jersey 2010 Tourism Report http://www.statesassembly.gov.je/AssemblyReports/2011/38806-41867-2272011.pdf

With a registered bed stock of 11,900[10], there is an approximate maximum tourist population of 14,900[11], including children.

There is also a further increase in the seasonal population as a result of the influx of migrant workers and people 'visiting friends and relatives'. The Labour Market Report of 2011[12] identified an increase of 3,190 in total workforce during the summer. However, there is some uncertainty as to what proportion of this number are seasonal workers from outside the island as opposed to residents who are already included in the population statistics and data on numbers of migrant seasonal workers is not collected giving rise to this uncertainty[13]. By comparison, the 2010 Tourism Report identifies a seasonal increase of 1,503 in staff employed in the hospitality sector. When coupled with the observed figures for visiting friends and relatives (estimated as 2,746[14]) the additional population from this sector is estimated to be 4,249[15].

3.3 Projected Resident Population

Population forecasts for Jersey, developed by the States of Jersey Statistics Unit, use a range of modelled scenarios based on different rates of fertility, mortality and net migration. Different scenarios of net inward migration are modelled which show increases in the number of economically active household heads. Increases of 150, 250, 325, and 650 household heads (hh) correspond to net total population increases, including dependants, of 320, 540, 700 and 1400 respectively.

It should be noted in reading the following paragraphs that the early predictions from the Statistic Units used change in household heads as the descriptor for each profile while more recent releases refer to net population change. This explains how, over the course of the discussion below, the design growth profile changes from +250(hh) to +500(net) with little change in the result. The change that does occur relates to refinements to the profiles made by the Statistical Unit over time.

The projections below use the 2011 census data as a starting point but then apply the population growth figures issued by the States of Jersey Statistics Unit in 2009 to forecast the population at regular intervals up to 2065. The population projections obtained using this methodology are summarised in Table 3.1 below. States of Jersey had previously used the +150hh projection for planning purposes and so this line has been highlighted below.

10 ibid.

11 Based on Population Forecast extrapolating from 2011 Census and 2010 Tourism Report

12 Jersey Labour Market at December 2011, States of Jersey – http://www.gov.je/SiteCollectionDocuments/Government%20and%20administration/R%20LabourMarketDec2011%2020120328%20SU.pdf

13 States of Jersey Statistical Unit, *pers. comm.*, 25/04/2012.

14 Based on Population Forecast undertaken as part of this review.

15 ibid.

	2011	2015	2020	2035	2065
Net NIL	97,857	97,857	97,857	95,757	80,757
+150hh	97,857	99,457	101,157	104,857	103,457
+200hh	97,857	99,957	102,157	107,957	111,057
+250hh	97,857	100,457	103,257	111,057	118,757
+325hh	97,857	101,257	104,957	115,657	130,257
+650hh	97,857	104,557	111,857	135,457	179,957

Table 3.1 - Projected resident populations based on 2009 growth figures from the States of Jersey Statistics Unit

The Jersey Island Plan 2011 identifies 4,625 new properties to be built between 2010 and 2020. On the basis of an occupancy rate of 2.3 this represents a population increase of 10,638 by 2020. Therefore the +150hh profile was taken and the growth up to 2020 replaced with the housing increase figures assuming linear growth. This gave the resident population profile as shown in Table 3.2 below.

	2011	2015	2020	2035	2065
Island Plan 2011	97,857	102,112	107,431	111,131	109,731

Table 3.2 - Design resident populations including new build housing

3.4 Projected Population including Visitors

Tourist numbers are difficult to forecast as they can be affected by a wide range of variables, most notably the state of the economy. Previous indications from First Research[16] and recent discussions with the Statistics Unit[17] suggest no growth in the near future and, therefore, it is assumed that the tourist population will remain constant at 14,900 over the forecast horizon. Similarly, seasonal workers and visiting friends and relative numbers are assumed to remain constant at 4,249.

It should be noted that the 2011 Census was the first to include residents that were off-island at the time. This has given rise to a step change in the population of some 6,000 people from predictions based on the 2001 Census as reported in the 2008 population update report[18]. Given this step

[16] First Research, *pers. comm.* 8th June 2009.

[17] States of Jersey Statistical Unit, *pers. comm.*, 25/04/2012.

[18] Jersey's Resident Population 2008, States of Jersey. http://www.gov.je/SiteCollectionDocuments/Government%20and%20administration/R%202008PopulationUpdate%2020090529%20SU.pdf

increase and the new build housing from the Island Plan the latest projection is significantly higher than the 2009 population model[19] and it would therefore be prudent to take the higher value for the purposes of flow forecasting.

Therefore, the projected populations shown in Table 3.2 were uplifted to account for the tourists and workers and visiting friends and relatives by adding these numbers to the resident population for each year. This results in the maximum total population for Jersey as shown in Table 3.3.

	2011	2015	2020	2035	2065
Island Plan 2011	117,006	121,261	126,579	130,279	128,879

Table 3.3 – Estimated Maximum Island Population with new build housing

3.5 Population Connected to Bellozanne Treatment Works

As noted previously, it was assumed that approximately 87% of properties were connected to the sewerage system in 2008. A more detailed assessment of residential and commercial connectivity was undertaken by TTS in 2012 which suggested that the 'average' connectivity was 85.2%[20]. However, it was noted that there are still a number of properties where the connectivity is unknown. Therefore, erring on the side of caution this assessment assumes the sewer connectivity remains at 87%.

It is assumed a further 1400 properties will be connected by 2028, over and above the expected population growth. This is primarily as a result of properties converting from septic and tight tanks for environmental reasons. For the purposes of this calculation it was assumed that the same proportion of visitor accommodation is connected to the sewerage system.

This gave a connected population of 102,278 in 2011 and this figure was projected into the future by using the profiles established above. For comparison purposes all of the profiles are shown in Table 3.4.

[19] The Jersey Population Model, States of Jersey, 2009.

[20] Email dated 18th April 2012 from Steve Fisher of TTS.

	2011	2015	2020	2035	2065
Island Plan 2011	**102,278**	**107,152**	**113,238**	**118,125**	**116,725**
Net NIL	102,278	102,897	103,664	102,752	87,752
+150hh	102,278	104,497	106,964	111,852	110,452
+200hh	102,278	104,997	107,964	114,952	118,052
+250hh	102,278	105,497	109,064	**118,052**	125,752
+325hh	102,278	106,297	110,764	122,652	137,252
+650hh	102,278	109,597	117,664	142,452	186,952

Table 3.4 – May 2012 projected growth scenarios for population connected to Bellozanne STW

Based on the assumptions presented above, the average of the 2035 connected population results for the various Statistics Unit scenarios is 118,785. This was closest to the +250hh scenario and so the maximum population connected to Bellozanne STW in 2035 was taken as 118,000 in May/ June 2012. This projection was based on 2007 population growth data from Statistics Unit pending the new information which was to be released in September 2012.

The data released in September 2012 showed a step change in the Statistics Unit model compared with the 2007 output and so a further review was carried out.

3.6 Review of 2012 Jersey Population Projections

3.6.1 Updated Scenarios (September 2012)

Based on the 2012 population growth data from Statistics Unit, the connected population scenarios were reviewed as a sensitivity analysis and are presented in Table 3.4A.

	2010	2020	2035	2065
Net +200	101,136	105,236	109,336	109,536
Net +350		106,836	113,836	120,936
Net +500		108,436	**118,336**	132,236

Table 3.4A – September 2012 projected growth scenarios for population connected to Bellozanne STW

The population projections from May 2012 give an overall range of results that are consistent with the States of Jersey Statistics Unit projections of September 2012. The growth profile selected in May 2012 also falls well within what are considered to be the most likely outcomes using a Monte Carlo methodology.

The September 2012 profile was therefore based on a net +500 growth which gave a 2035 design population of 118,336. Note the change in descriptor from +250**hh** to **net** +500 as discussed above.

3.6.2 Additional Scenarios (January 2013)

The Statistics Unit released further population growth scenarios in January 2013. These considered higher growth than previous models following analysis of the most recent data. The latest population predictions under review are based on four different scenarios namely, net nil, net +350, net +700 and net +1000. The resident population figures from the potential Four Scenarios discussed by the Council of Ministers are as follows:

	2010	2020	2035	2065
Net + nil	97,100	99,000	99,200	90,400
Net +350		102,800	109,800	116,900
Net +700		106,600	120,300	143,300
Net +1000		109,900	129,400	166,000

Table 3.5 – Projected resident populations based on 2013 growth figures from the States of Jersey Statistics Unit

These were again converted to connected resident population using the same methodology as previously i.e. 87% connected plus 160head/year for new connections over twenty years.

The connected resident population was then converted to total population by adding 2009/10 figures for Tourists and Visiting Friends and Relatives. As noted above, the maximum visitor population at any time of 19,149 is not expected to change in the coming years. This is factored by 87% for connectivity as above, giving an additional 16,660 population in each case.

	2010	2020	2035	2065
Net + nil	101,136	104,390	106,164	95,308
Net +350		107,696	115,386	118,363
Net +700		111,002	124,521	141,331
Net +1000		113,873	132,438	161,080
Sept 2012 profile	101,136	108,436	**118,336**	132,236

Table 3.6 – Projected total connected populations based on 2013 growth figures from the States of Jersey Statistics Unit

The population projections issued after the original May 2012 prediction show that the original 118,000 design figure continues to sit in the middle of the Statistics Unit's various projection models. The selected forecast design connected population of 118,000 in 2035 is therefore still considered to be a reasonable design horizon. However, given the obvious uncertainty, it is considered advisable to provide flexibility in the design such that 118,000 is not an absolute limit but can be expanded with minimal cost in the future. The options regarding how this flexibility is incorporated into the design of Bellozanne STW are discussed further in Section 3.7.1.

It is normal to allow for some contingency, or headroom, to allow for uncertainty in other factors aside from population. These tend to be, amongst other things, climate change, creep[21] and future changes in law or planning policy. However, given that 118,000 is close to the average of the Statistical Unit forecasts (between 106,164 and 132,438) in 2035, and the flexibility of operation that will be designed in, it is considered unnecessary to allow for any further uncertainty.

3.7 Design Horizon

3.7.1 Bellozanne STW Design

The Bellozanne STW site appears to have space to accommodate future increases in the connected population of 118,000 by up to a further 20% for the ultimate scenario. The 2035 design population of 118,000 is still considered to be a reasonable horizon for the works. However, given that there is such a wide range of potential growth it is recommended that the design is completed using conservative parameters. This will mean that some structures are initially oversized but not to an extent that underloading causes performance issues at commissioning.

This oversizing can also be done in conjunction with, or replaced by, leaving space for additional ASP lane(s) and FSTs in case of a larger than expected increase over time. However, given that space in Bellozanne Valley is at such a premium this may not be practical. Similar issues are attached to the PSTs and inlet works but these elements are less sensitive to underloading and therefore oversizing is not a significant risk. These issues are covered in more details in Section 8.

In general terms, slightly oversizing some of the process units now will be a smaller cost than trying to extend later.

At the design stage a sensitivity analysis can be carried out to check what population the alternate parameters would accommodate. There may still be a need for some minor modifications e.g. an extra blower in later years but this would be a minimal cost and could probably be coincided with normal maintenance/replacement.

As for the population growth scenarios, the new Bellozanne STW will cover the Net +350 scenario up to 2065. The Bellozanne STW site will be able to accommodate Net +700 scenario up to 2065 with further expansion to Activated Sludge Plant capacity. However, it will be necessary to have another satellite STW to cater for further increase in population beyond 2045 for the scenario of Net +1000.

It should be noted that flows to the STW may, in reality, be limited by the capacity of the sewerage network to handle the increase in population.

[21] Creep is defined as the impermeable area from developments at the sub-property (e.g. paving over gardens) which delivers additional surface water load to a network.

3.7.2 Discharge Conditions

Since the 1997 CREH Report on the Trophic Status of St Aubin's bay, discussed in Section 2, TTS have carried out some further limited investigations of the water quality in St Aubin's Bay and no further evidence of eutrophication has been identified. The original study was inconclusive and the further study has not raised any further concerns that would suggest the Bay should be classed as sensitive. It is therefore recommended that an extension of the outfall is not included in the proposal pending further collection of evidence.

This evidence based approach is subject to agreement with the Regulator but is considered to be a reasonable and practical way forward. This is discussed in more detail in Section 6 below.

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4. Best Available Technology Review

A report entitled "Bellozanne STW Best Available Technology Report (May 2012)" was completed to consider and recommend the most appropriate technology for the treatment of the Island's liquid waste. This report reviewed the best available treatment options for Bellozanne STW and, having examined the advantages and disadvantages of each process, proposed a preferred way forward and layout for each of the consent standards that are under discussion. A summary of the outcome of this report is provided below.

The sewage treatment plant at Bellozanne currently does not achieve the required discharge consent and suffers from extensive biological foaming, a consequence of which is poor removal of suspended solids and lower bacteriological kill from the UV disinfection process.

In order to address the above issue, a review is being carried out so as to ascertain what the options are surrounding the required discharge consent from the works and to determine the process options available to achieve these consents. Clearly the process design and selected technologies are largely driven by the consent standard required.

Striking the correct balance between environmental and health benefits can be achieved by a more robust treatment process which will include establishing an evidence based policy based on the assimilative capacity of the receiving waters. The assimilative capacity is indicative of whether a given body of water is to be considered as 'sensitive'.

Sensitive waters are designated under the Urban Waste Water Treatment Directive where more stringent treatment is required to protect the aquatic environment of the receiving water. This can be enclosed bays with the risk of nutrient growth, or designated shell fisheries where much cleaner water is required in terms of bacteriological quality. For inland or freshwater conditions the nutrient to be controlled is phosphorus but in saline conditions this is nitrogen.

Populations from 15,000 to 100,000 are subject to a 15 mg/l Total Nitrogen (TN) limit in the event of the receiving water body being designated as sensitive. For populations above 100,000 the TN limit is 10 mg/l. Generally these limits are applied as annual averages.

Bellozanne has a population equivalent below the 100,000 value in winter but above in the summer months. Should the receiving water be designated as being sensitive, a TN standard of 10 mg/l in the summer months and 15 mg/l in the winter months could be applied. Denitrification is very temperature dependent and therefore the winter consent standard applied to the works will affect the size and cost of the treatment plant required.

For example, a plant designed to achieve a TN standard will be much larger and have much higher energy costs than a plant designed to achieve the Urban Waste Water Treatment Directive standard for a non-sensitive receiving water i.e. 25 mg/l BOD, 35 mg/l SS. Therefore a far more robust treatment process with lower energy usage and complexity can be considered.

The preferred treatment process regardless of consent standard required is the activated sludge process. This encompasses a variety of mechanisms and processes that use dissolve oxygen to promote the growth of biological floc. Activated sludge is a robust well known technology that is familiar to the operators at Bellozanne thus minimising any training that would be required. It also provides a high degree of flexibility of operation to allow for the variation in summer/ winter influent conditions. Should a carbonaceous removal (BOD standard) or nitrifying (ammonia standard) be required then a variation of the activated sludge process known as the Inclined Bubble Aeration (IBA) would be proposed. This provides simplicity of control whilst avoiding over aeration which is common on standard systems and thereby significantly reduces the overall energy consumption. The IBA system cycles between aerobic and anoxic periods and therefore a degree of denitrification is achieved. However to achieve a 15 mg/l or 10 mg/l TN standard a large anoxic zone and a large recycle from the aerobic to the anoxic zone are required. Due to these design requirements a more conventional layout based on separate anoxic and aerobic stages is more practicable.

The Bellozanne STW Best Available Technology Report (May 2012) concluded that a conventional STW was the most appropriate technology. The level of treatment required will depend upon the location of the effluent outfall discharge and the assimilative capacity of the receiving waters. Therefore, any proposed layout at this stage would make provisions for adequate capacity in the design together with expansion capability on the site should an enhanced treatment standard be required in the future.

5. Design Basis

5.1 Introduction

The purpose of this Section is to clearly define the basis upon which the process plant design and assessment has been undertaken.

5.1.1 Design Philosophy

The plant design has been undertaken such that consent compliance can still be achieved with any individual process unit taken out of service. For example a single primary settlement tank or a single final settlement tank can be taken out of service at the design load and the plant will still achieve effluent compliance. However it has not been designed such that an activated sludge lane and a final settlement tank can be taken out of service at the same time under design load conditions and still meet the required effluent quality. To do so leads to a plant sized for an uncommon condition that can be largely avoided via planned maintenance and which would be oversized under normal operating conditions leading in itself to potential process issues with the biological system.

Within the individual process units and ancillary works, equipment will be provided with sufficient redundancy to allow uninterrupted services in the event of any equipment failure or maintenance requirements. This will include duty/ standby or boxed spare equipment as appropriate.

5.1.2 Existing Plant

The current sewage treatment plant does not meet the consent standard required. The current works and problems encountered are described in the Operation Strategy Report of 14th May 2011 and this should be referred to for further details.

The majority of the new sewage treatment works (secondary treatment including selector zone, activated sludge tanks and final settlement tanks) will be built in a different area of the works to the existing plant such that it does not affect the operation of the existing works. There are no plans at present to incorporate the existing plant into the final scheme or to use existing structures within the works in their current operating intention.

5.2 Design Flows and Loads

5.2.1 Projected Design Load

The plant is designed based on the predicted load estimated for the year 2035 as detailed in the Grontmij report entitled 'Population, Flows and Loads Review' dated June 2012. The table below summarises this data along with that for year 2011.

Year	Maximum Population	BOD	SS	Ammonia	Total N	COD
		kg/d	kg/d	kg/d	kg/d	kg/d
2011	101,795	7,459	8,885	1,172	1,580	18,648
2035	118,000	8,431	10,100	1,302	1,774	21,078

Table 5.1 – Predicted loads to Bellozanne STW

These figures include the load associated with the return liquors and show the maximum daily load expected at the works. However it is also important to assess the likely minimum load to the works to ensure that there is sufficient turndown and flexibility in the design.

The sustained minimum biological load can be estimated from the resident population taken from the States of Jersey 2011 census of 97,857. The percentage of properties that were connected to the sewerage system in 2008 is approximately 87%. Therefore based on the census figure of 97,857 the connected resident population in year 2011 was 85,135. This population value can be used to calculate the minimum load expected at the works. Assuming a BOD of 60g/head/day gives a BOD into the works of 5,108 kg/d.

5.2.2 Actual Biological Load in Years 2010 to 2012

In addition to calculating the expected BOD load from the connected population, influent data has also been examined for the years 2010, 2011 and 2012 to give an indication of what the actual load seen at the works is and to determine the monthly variation in biological load. The table below gives the average daily load calculated for each month in which the biological load of the incoming raw sewage was analysed.

Crude Sewage						
	Average Daily BOD Load, kg/d			Number of Data Points		
Month/Year	**2010**	**2011**	**2012**	**2010**	**2011**	**2012**
January	2,670	3,345	3,546	10	11	11
February	3,434	3,908	4,519	10	10	9
March	2,195	3,006	4,014	12	1	9
April	2,850		4,216	9		10
May	3,595		3,898	9		3
June	3,477			11		
July	3,448			11		
August	3,589			10		
September	4,291			11		
October	4,316			11		
November	2,928			9		
December						

Table 5.2 – Observed monthly average loads to Bellozanne STW

This indicates a reasonably consistent BOD load throughout the year and that the minimum load to the plant is not surprisingly in the winter months due to factors such as the lower tourist population and higher rainfall leading to dilution of the influent and loss of solids over storm overflows. The minimum average daily BOD load detected is approximately 2,200 kg/d in March 2010 which is approximately 25% of the 2035 BOD maximum design load of 8,431 kg/d thus requiring the plant to have a 4:1 turndown under normal operation. However this is the average daily load and to assess the turndown of the works the minimum load seen over a sufficiently long period to affect the system biology needs to be examined. This is assumed to be between one and two sludge ages, i.e. approximately one week for a carbonaceous removal plant.

To affect the operation of the activated sludge plant the BOD load would need to be low for a prolonged period of several days. It therefore makes sense to examine the weekly average BOD load for each of the three years detailed to assess how the BOD load over a period of one week compares to the 2200 kg/d value determined above. This gives the following results: -

	2010	2011	2012
Average	3,379	3,562	4,087
Minimum	1,171	2,715	1,720
Maximum	6,081	4,724	6,530
90%ile	4,972	4,585	5,614
10%ile	1,846	2,719	2,229

Table 5.3 – Daily BOD Load (kg/d) Averaged Over a Week at Bellozanne STW

The minimum load of 1,171 kg/d observed in 2010 occurs in August and is low as a result of 2 data points that are substantially lower than the other samples taken over the summer months. It would appear therefore that there could be an error in the sampling or analysis of these points. If these two data points are ignored then the 10%ile for 2010 equates to 1,920 kg/d and the minimum is 1,400 kg/d.

The 10%ile is a useful value to use rather than the minimum as it smoothes out the troughs in the curve and makes an allowance for any spurious data points. This gives a minimum BOD in the range of 1,850 kg/d and there are only 4 weeks out of the 74 weeks for which data was analysed that have a value below this figure. In comparison there are 11 weeks that have an average daily BOD load below the 2,200 kg/d. However it should be noted that most of these occur during 2010 whereas in years 2011/2012 there is only one week analysed that falls below either of the daily minimum loads considered.

Since there have been upgrades to the sewerage network to address the saline intrusion issue and it will be this infiltration that is largely contributing to the dilution of the raw sewage and the low concentrations/ biological loads to the works it is reasonable to assess the realistic minimum concentration on the recent data to the works rather than that in 2010. A minimum BOD load of 2,200 kg/d to the works has therefore been used in the design and the plant turndown assessment. It should be noted that this does not mean that the plant cannot treat loads lower than this value but if prolonged periods of low loads are realised then activated sludge lanes will need to be taken out of service as the biological system is limited in the turndown achievable for a set process capacity.

5.2.3 Projected Design Flows

The projected flows to the Bellozanne STW are detailed in the Grontmij report 'Population, Flows and Loads Review' dated June 2012 and are summarised below: -

Year	Population	DWF (l/s)	Average (l/s)	FFT (l/s)	Formula A (l/s)
2011	101,795	307	368	720	1,969
2035	118,000	343	412	813	2,261

Table 5.4 – Daily predicted flows to Bellozanne STW

These flows are influent and do not include return liquors from the plant itself. The return liquors are estimated to be 17 l/s assuming that they are returned over a 16 to 17 hour period each day. This gives a total Flow to Full Treatment of 813 + 17 = 830 l/s.

5.2.4 Saline Infiltration and Biological Concentration

The Grontmij report 'Population, Flows and Loads Review' dated June 2012 states that the infiltration is high at 185 l/s due to illegal drainage connections, saline intrusion and water from streams entering the works. Planned works have been identified to reduce this significantly and the design infiltration rate has been set by TTS at 100 l/s for the future and this value utilised in the projected design flows.

The biological concentration of the sewage entering the works over the period 2007 to 2009 was as follows: -

	Average	Maximum	Minimum
BOD (mg/l)	148	457	25
SS (mg/l)	217	874	4
Total Nitrogen (mg/l)	55	143	15

Table 5.5 – Observed biological concentrations to Bellozanne STW 2007-2009

Severe saline intrusion can be a major problem to robust operation of a sewage treatment works as not only does it affect solids settlement but it also contributes to the flow coming into the works without increasing the load thereby resulting in low BOD concentrations. Low influent BOD concentrations result in low mixed liquor suspended solids (MLSS) concentrations within the activated sludge plant that in turn are more susceptible to changes in influent conditions. This means that the plant is far less robust and more difficult to operate effectively.

In order to get a better assessment of the duration of these low influent concentrations, further data has been obtained for years 2010, 2011 and 2012. These are obtained from daily composite samples and show the following outputs for the BOD concentration: -

	2010	2011	2012
Average (mg/l)	143	130	162
Maximum (mg/l)	369	256	411
Minimum (mg/l)	25	77	46
Number of data points	113	22	43
Number of data points with BOD less than 100 mg/l	42	6	10

Table 5.6 – Observed daily BOD concentrations in Bellozanne STW

These are in general agreement with the data obtained for years 2007 to 2009 and again indicate that there are prolonged periods where the biological concentration of the influent is very low due to the high infiltration. Any influent with a BOD concentration below 100 mg/l for a prolonged period can be difficult to treat reliably due to the low floc density and light sludge blanket that results. However the above data covers the influent daily concentrations whereas this would need to be realised for several days to a week before the system biology was affected.

If the data is used to average the BOD concentration into the works over a period of a week using the months January to March for year 2011 and January to May for year 2012, 5 out of 28 weeks have a concentration below 100 mg/l and these occur in the main in January.

The concentrations shown in Table 5.7 below are averaged over a period of a month and are well above the 100 mg/l value for the winter months in 2011 and 2012. This would suggest that the observed biological concentrations of below 100 mg/l do not persist for weeks on end even without the current refurbishment work on the sewerage network.

Crude Sewage Average Daily BOD Concentration per Calendar Month						
	BOD Concentration, mg/l			Number of Data Points		
Month/Year	**2010**	**2011**	**2012**	**2010**	**2011**	**2012**
January	88	118	134	10	11	11
February	122	145	181	10	10	9
March	77	118	181	12	1	10
April	127		165	9		10
May	160		135	9		3
June	164			11		
July	176			11		
August	161			10		
September	198			11		
October	183			11		
November	113			9		
December						

Table 5.7 – Observed monthly BOD concentrations in Bellozanne STW

The plant has therefore been designed on the assumption that there will not be any prolonged period (5 days or greater) where the BOD concentration into the works falls below 100 mg/l. This seems a reasonable assumption based on the recent data shown above and taking into account the planned improvements to the sewerage network.

5.2.5 Bellozanne STW Future Discharge Standard

The new sewage treatment works has been designed on the basis of an evidence based policy approach for effluent quality and that St Aubin's Bay is not deemed to be 'Sensitive' at some point in the future. This means that treatment of discharged effluent from the Bellozanne STW shall be in accordance with Article 4 of the UWWTD (91/271/EEC) and the discharge shall satisfy the requirements of Annex I.B as shown in Table 1 and summarised as follows:

- Biochemical Oxygen Demand (BOD_5) - 25 mg/l
- Chemical Oxygen Demand (COD) - 125 mg/l
- Total Suspended Solids - 35 mg/l

In addition UV disinfection will be included in the process and designed to achieve a measured applied UV dose of 30 $mW.s.cm^{-2}$ at 254 nm to minimise any potential health risk.

It should be noted that the process design of the plant does allow for future expansion of the secondary treatment stage such that an ammonia consent or total nitrogen consent can be achieved in the future if required.

5.3 Environmental Factors

5.3.1 Odour

In the UK there is a legal requirement regarding odour not to cause a nuisance. However, in law for sewage treatment works there are no national odour standards that must be met. The odour standard that is chosen for a planning consent for a sewage treatment works tends to be done on a case by case basis and is usually set as a percentile odour or hydrogen sulphide concentration at the boundary or nearest receptor.

The odour standards that have been applied at other sewage treatment works have ranged from 5ou/m^3 as a 98%ile at the nearest receptor to 1 ou/m^3 as a 98%ile or 5 ou/m^3 as a 99.5%ile at the treatment works boundary. An odour standard of 5ou/m^3 as a 98%ile at the nearest receptor has been adopted by at least one water utility in the UK for determining whether odour abatement measures are required at sewage treatment works.

TTS has an approach for assessing the impact of its green waste reception and composting facility. The approach involves taking odour concentration measurements using handheld olfactometry equipment around the facilities and creating a steady state model of the odour concentrations around the facilities for the particular day and meteorological conditions that the measurements were taken. It does not involve creating an atmospheric dispersion model of the odour emissions or running the model with meteorological data over a period of time to take into account the different meteorological conditions experienced during the year.

An odour monitoring protocol has been drawn up by TTS for Bellozanne STW to monitor the effectiveness of the proposed sludge treatment facilities. This protocol is based on using simplified olfactometry and portable analyser methods.

Best practice will be followed with appropriate mitigation measures implemented to control potential odour nuisance during both the construction and operation phases.

5.3.2 Noise & Vibration

There are potential effects associated with noise from the excavation works, construction activities and operation of the completed plant which may require mitigation measures to be employed. The effects of future decommissioning of the existing sewage treatment works also need to be assessed.

Activities associated with the excavation and construction works and the operation of the proposed facility are not considered likely to include significant sources of ground-borne vibration, and the separation distance between the Site and receptors is, in vibration terms, relatively large.

5.3.3 Planning

For the new STW, a maximum building height of 7.5m above existing ground levels is assumed. This height corresponds to the potential location of the Inlet Works and, therefore, represents the worst-case scenario. The ability to see the Inlet Works is, however, dependent on view location of the receptor. Other existing buildings are higher with a maximum height of 18m for the new sludge digesters but these will sit below the level of the adjoining hillside.

The Site is enclosed by strong visual barriers created by the topography of the steep valley, reinforced by the wooded vegetation along these slopes. Views of the new STW from the esplanade will be generally screened by virtue of the strong visual barriers created by built form and topography.

The proposed works would result in the replacement of part of the valley slope with new structures. The proposed units vary in size and scale and would create a continuation of the character of the existing built form, mirroring the height and grain of the Sludge Thickening & Dewatering Building. As the development is adjacent to the existing site, the effect it has on the rest of the valley character away from the immediate site is limited.

6. Effluent Disposal

6.1 Hydrographic Survey

The effluent from the Bellozanne STW is currently discharged via an outfall into St Aubin's Bay near the First Tower area at a distance of 0.5km from the seawall.

Various surveys of the St Aubin's Bay, including Oceanographic Survey, Bathymetric & Geophysical Survey and Sediment Sampling, were completed in June/ July 2012 in order to generate a model for future predictions of water quality and to gain a better confidence level in the construction cost estimates.

The hydrographic survey data have been used to verify the hydrographic model created by Marcon as part of the Outfall Assessment (Desk) Study (April 2010).

6.2 Water Quality

Based on the review of the data currently available, it has been identified that there is a gap in the water quality data for the St Aubin's Bay during winter months. A further water quality survey for baseline data collection for St Aubin's Bay and the surrounding area has been initiated to supplement the available data to create a robust model for predicting the water quality at the St Aubin's bay.

It is anticipated that an evidence-based policy will be established by the Regulator to determine the effluent quality requirements for discharge from the Bellozanne STW.

6.3 Outfall Modelling

Modelling (eutrophication and bacteriological) will be required to develop an appropriate evidence-based policy for the States of Jersey for disposal of effluent from the Bellozanne STW. It is anticipated that modelling of microbial and nutrient parameters to refine the model of the St Aubin's Bay will be continued and completed by April 2014.

The model will be able to predict faecal coliform concentrations throughout the entire model domain. The model will also be able to predict the future bathing water compliance of the beaches and historical data can be used to calibrate the model.

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7. New Bellozanne STW Process Design

7.1 Process Overview

The new treatment process as shown in Figure 7.1 - Process Flow Diagram (Appendix B) incorporates the following stages:

7.1.1 Preliminary Treatment

Preliminary treatment incorporates screens, FOG (fats, oils and greases) removal, grit removal and flow measurement/ control to limit the flow to full treatment to 3DWF. Two storm water tanks will be used as blind tanks and there will be no overflow to the outfall. The capacity of the First Tower Sewage Pumping Station will be restricted to the current level of 1,100 l/s and any excess flows will be handled by existing and/ or expanded storage facilities.

7.1.2 Primary Settlement Tanks (PSTs)

There are 4 No. primary settlement tanks proposed each with a diameter of 24.5m. These provide gravitational settlement of solids and typically will remove between 50 and 65% of the suspended solids in the influent.

7.1.3 Activated Sludge Plant (ASP)

The settled sewage flows by gravity to the selector zone stage of the activated sludge process from where it is distributed between three aeration lanes. The ASP provides the biological treatment required prior to the flow gravitating to the eight final settlement tanks. The ASP can be upgraded to meet possible future consent standards as detailed in the main text below.

7.1.4 Final Settlement Tanks (FSTs)

After the period of aeration in the activated sludge process the mixed liquor passes to the final settlement tanks. Two options have been examined, namely four tanks at 34m diameter or 8 tanks at 24.5m diameter. Here the activated sludge flocs settle to the bottom of the tank from where the sludge is continuously withdrawn and returned to the inlet of the aeration tank or removed from the system as SAS. The supernatant liquor passes over the outlet weir.

7.1.5 UV Disinfection

The effluent from the final settlement tanks passes to the UV disinfection plant where disinfection of the discharge occurs such that the consent standard relating to faecal coliforms is achieved on the basis of applied UV dosing.

7.2 Flow and Load to Works

7.2.1 Design Flow to Works

The projected flows to the Bellozanne STW are detailed in the Grontmij report 'Population, Flows and Loads Review' dated June 2012 and are summarised below: -

Year	Population	DWF l/s	Average l/s	FFT l/s	Formula A l/s
2011	101,795	307	368	720	1,969
2035	118,000	343	412	813	2,261

Table 7.1 – Daily predicted flows to Bellozanne STW

This flow does not include return liquors from the plant. The return liquors are estimated to be 17 l/s assuming that they are returned over a 16 to 17 hour period each day. This gives a total flow to the works of 813 + 17 = 830 l/s.

7.2.2 Design Load to Works

As detailed in Section 5.0, the load to the sewage treatment works is: -

Year	Population	BOD kg/d	SS kg/d	Ammonia kg/d	Total N kg/d	COD kg/d
2011	101795	7,459	8,885	1,172	1,580	18,648
2035	118000	8,431	10,100	1,302	1,774	21,078

Table 7.2 – Predicted loads to Bellozanne STW

These figures include the load associated with the return liquors and show the maximum daily load expected at the works. The process design has been carried out based upon the flows and loads detailed above but has also considered the possibility of a future increase in flow and load of up to 20% and this is detailed in Section 8.4. The turn down achievable by the biological plant has also been considered as discussed in Section 7.3.4.3.

7.3 Design of Treatment Process Units

The P&ID's for the new treatment processes are shown in Figures 7.2.1 to 7.2.13 (Appendix B) and are discussed in detail in this Section.

7.3.1 Balance Tank Requirement

The plant as designed is capable of accepting and treating the daily fluctuations in flow and load normally envisaged with a sewage treatment plant treating predominantly domestic waste. A balance tank is not required unless there is high saline intrusion or the possibility of high spikes in ammonia coming into the works.

High saline spikes will cause settlement problems due to density currents in the final settlement tanks (saline water is more dense than non-saline water) thus increasing the risk of solids carry over and will also affect the system biology. However, rehabilitation works are in place to reduce the saline intrusion currently experienced and therefore it will no longer be an issue. This being the case, balancing of the flow into the works will not be required to take out any saline peaks.

High spikes in ammonia will pass through the system and affect the ammonia level in the discharge. If high spikes are expected such as a cattle market on a small works then balancing of the flow to smooth out the ammonia load into the works would be recommended. However this is not the case at Bellozanne and in addition the selection of the activated sludge process means that, should an ammonia consent be required in the future, there is a high degree of buffering of any spikes in influent concentration due to the retention time achieved within the biological system.

Carbonaceous treatment is not affected by high peak loads into the works as the BOD is adsorbed onto the sludge floc. Balancing of the flow is not therefore required providing that the saline issue is addressed.

7.3.2 Inlet Works

The inlet works comprises mechanically raked screens, FOG (fats, oils and greases) removal, grit removal and flow measurement and control. The flow passed forward to the treatment works is 830l/s (3DWF plus balanced liquors). It is understood that the flow to the works will be pumped at a maximum rate of 1,100 l/s. Should this be the case then storm tanks are required to ensure that the Flow to Full Treatment (FFT) to the works is not exceeded.

The storm tanks ideally would be positioned adjacent to the inlet works such that any flow that exceeds the FFT can simply back up from the flow measuring device and overflow a weir to the storm tanks. This avoids the use of a gravity pipeline to transfer storm flows to the storm tank. The problem with a gravity pipeline is that it either has to completely drain, leading to a large headloss between structures or failing this, it would potentially have standing storm water for long periods when no storm flow occurred, which would in turn lead to odours and solids settlement.

Two storm water tanks are proposed and these will both act as blind tanks such that there will be no overflow to the outfall. The capacity of the First Tower Sewage Pumping Station feeding the works will be restricted to the current level at 1,100 l/s and any excess flows will be handled by existing and/ or expanded storage facilities.

Once a high level is detected in the onsite storm tanks the pumped flow will be limited to 830 l/s and any excess flow backs up from the First Tower Sewage PS and flows to the existing and/ or expanded storage facilities. The existing and/ or expanded storage facilities should be provided with mechanical desludging to facilitate the cleaning of these storage facilities on a regular basis.

7.3.3 Primary Settlement Tanks

The purpose of primary sedimentation is to remove by gravitational settling from water, suspended particulate matter that is denser than water. In addition to this primary settlement tanks help to equalise the strength of the sewage passing to the biological treatment stage, remove additional FOG from the flow and remove insoluble BOD load associated with the suspended solids thereby reducing the biological load on the activated sludge plant.

Radial flow tanks are proposed at Bellozanne. These are circular tanks with a sloping floor towards a central hopper. A rotating scraper directs the sludge towards the hopper from where it is extracted. The scraper mechanism can be driven from the centre with a fixed bridge arrangement or from the perimeter via a rotating bridge.

When there are a number of tanks, even distribution of the flow such that each tank receives a similar proportion of the flow at all incoming flowrates is important such that equal load distribution is achieved. To this end a distribution chamber is required that promotes smooth flow in the chamber thus avoiding gross turbulence and providing hydraulic symmetry. Poor performance of a distribution chamber can lead to serious problems in downstream units since at high flows some units may be under loaded whilst others are overloaded to the point of failure.

The inlet zone of the primary settlement tank is required to slow down the flow as quickly as possible and dissipate the kinetic energy. The flow is then diverted across the whole area of the tank such that settlement can take place.

The flow passes radially from the centre of the tank to the periphery where it overflows a weir and passes to the activated sludge tanks. The weir overflow rate should be designed such that streaming and disturbance due to upward draw to the outlet weir is avoided.

7.3.3.1 Design Criteria

1. Surface Loading

   Surface loading is the volume loading on the surface of a settlement area and is given by: -

   $$\text{The surface loading } (m^3/m^2/d) = \frac{\text{Maximum Flow } (m^3/d)}{\text{Tank Surface Area } (m^2)}$$

   The surface loading conventionally used for radial flow tanks in the UK is 45 $m^3/m^2/d$ at FFT. However higher loading rates can be utilised as long as the reduction in removal efficiency of SS and the associated insoluble BOD is taken into account.

2. Retention Period

   The retention period should be long enough to enable the required degree of settlement to take place. If however it is too long, septicity may occur with its consequent harmful effect on the biological treatment stage. Typical retention time required is 2 hours at FFT. This is not a critical parameter however but more of a guidance value.

3. Weir Overflow Rate

$$\text{The weir overflow rate } (m^3/md) = \frac{\text{Maximum Flow } (m^3/d)}{\text{Total length of outlet weirs (m)}}$$

7.3.3.2 Primary Settlement Tank Design

For the new works at Bellozanne, four primary settlement tanks (PSTs) are proposed. To ensure equal distribution to each of the four tanks the flow from the inlet works passes by gravity to a distribution chamber which feeds the four PSTs. This ensures good distribution of flow to each of the tanks. A tank can be taken out of service via stop logs on the weir or a weir penstock in the distribution chamber.

The critical parameter for the primary tanks is the surface loading and the tanks are designed such that the surface loading of 45 $m^3/m^2/d$ is not exceeded with all tanks in service. This gives four tanks each with a diameter of 24.5m if the 20% future contingency is taken into account. The actual surface loading realised is as shown in the table below: -

	Maximum Flow		**Minimum Flow**	**Average Flow**	
No. tanks in operation	4	3	4	4	3
Tank Surface Gross Loading (m^3/m^2.day)	38.4	51.1	16.1	19.3	25.8
Tank Surface Nett Loading (m^3/m^2.day)	39.3	52.3	16.5	19.8	26.4
Upflow velocity (m/h)	1.6	2.2	0.7	0.8	1.1

Note: Including inlet diffuser drum

Table 7.3 – PST Design Surface Loading Rates

A stilling well of 3.5m is installed at the inlet zone to disperse the influent flow and draw the flow path down into the tank to prevent any short circuiting.

The retention time achieved in the tanks is 1.8 hours at maximum flow assuming a sidewall depth of 2.5m, which is satisfactory albeit below the 2 hours at FFT.

7.3.3.3 Sludge Draw off and Pipework

To prevent blockages the desludging pipelines conveying the sludge by gravity should be as short as possible, contain a minimum number of bends and be at least 150mm in diameter.

7.3.3.4 Removal Efficiency

The data recorded for years 2010, 2011 and 2012 indicate that the average removal efficiency of the primary settlement tanks is within the range of 25% to 38% for BOD and 38% to 52% for SS.

The data recorded in 2011 is limited compared to the other two years and it is this year that contains the lower end of the range for SS. The data for both 2010 and 2012 shows an average SS removal of between 50 and 52%. This seems the more sensible value particularly when considering that design removal efficiencies are typically around 55 to 65% and normally primary settlement tanks will achieve the higher end of this range. For design purposes a range of 55% to 65% SS removal has therefore been used assuming that low performance has been linked to saline intrusion in the past and this will be addressed.

Normal design removal efficiencies for BOD are between 25 to 35% across the primary settlement tanks so this backs up that recorded by the data and is used in the design calculations. It is the BOD that affects the sizing of the secondary treatment stage and not the SS so this is the more critical parameter as far as the secondary treatment plant design is concerned.

The data recorded for years 2010 to 2012 indicates that the removal of ammonia was between 0 and 5%. Any removal of ammonia is highly unlikely and the design assumes that no removal of ammonia is achieved across the primary settlement tanks.

7.3.3.5 Sludge Quantity

The estimated primary sludge production using the above removal efficiencies is as follows: -

Year	SS Removal Rate	kg DS/d	Sludge Production, m3/d			
			1.5% DS	2% DS	3% DS	4% DS
2011	65%	5,775	385	290	195	145
	55%	4,887	325	245	165	120
2035	65%	6,565	440	330	220	165
	55%	5,555	370	280	185	140

Table 7.4 – Primary Sludge Production at varying dry solids (DS) content

7.3.4 Secondary Treatment – Activated Sludge Process

The secondary treatment stage is designed initially to achieve *carbonaceous removal* (Stage 1) only. However it is recognised that expansion may well occur in the future such that the plant will need upgrading to achieve either *nitrification* (Stage 2) or a *total nitrogen* (Stage 3) standard. To this end the secondary treatment stage has been designed such that it can be upgraded to achieve these requirements at a later date if required.

7.3.4.1 Settled Sewage Load to Secondary Treatment

The load to the secondary treatment stage is as follows: -

	2011		2035	
	Maximum	Minimum	Maximum	Minimum
Biochemical Oxygen Demand (BOD kg/d)	5,594	4,848	6,323	5,480
Total Suspended Solids Load (SS kg/d)	3,998	3,110	4,545	3,535
Ammonia Load (kg/d)	1,172	1,172	1,302	1,302
Total Nitrogen Load (TN kg/d)	1,580	1,580	1,774	1,774

Notes

- Maximum load is based on 25% removal of BOD and 55% removal of SS across the primary settlement tanks.
- Minimum Load is based on 35% removal of BOD and 65% removal of SS across the primary settlement tanks.

Table 7.5 – ASP Design Loading Rates

7.3.4.2 Design Criteria

(i) F/M Ratio

In order to treat the incoming sewage to the required level a particular quantity of biomass will be required. The relationship between the food source (F) and the biomass (M) is known as the F/M ratio, or sludge loading rate. The F/M ratio is calculated as follows:-

$$F/M = \frac{\text{Organic Loading}}{\text{Volume of Sludge}}$$

$$= \frac{\text{BOD Load (kg/d)}}{\text{Volume of Aeration Tank (m}^3\text{) x MLSS (kg/m}^3\text{)}}$$

The CIWEM Handbook of UK Wastewater Practice entitled 'Activated Sludge Treatment' lists the following parameters for activated sludge plants:-

Treatment Mode	Sludge Loading kg BOD/kg MLSS. d
High rate (for partial treatment)	> 1.0
Conventional (20 mg/l BOD)	0.25 - 0.50
Conventional (10 mg/l BOD)	0.15 - 0.25
Conventional with nitrification	0.15
Extended Aeration	0.05 - 0.15

Table 7.6 – Activated Sludge Loading

Note that the extent of treatment achieved will be dependent upon other parameters such as the sewage temperature.

(ii) Sludge Age (Solids Retention Time)

The sludge age is an estimate of the average time period that a bacterial cell present in the activated sludge floc, is retained in the aeration system. In other words it is the number of days during which the total mass of sludge wasted plus the total mass of sludge solids passed in the effluent is equal to the mass of sludge solids undergoing aeration in the aeration tanks assuming that the activated sludge plant is being operated under equilibrium conditions.

This relationship does not take into account the mass of sludge in the final settlement tanks.

$$\text{Sludge Age} = \frac{\text{Volume of Activated Sludge Tank } (m^3) \text{ x MLSS } (kg/m^3)}{\text{SAS } (kg/d) + \text{Solids in final effluent } (kg/d)}$$

In an activated sludge system the bacteria must have sufficient retention time within the system in order to reproduce and maintain a healthy population such that the level of treatment required can be achieved. Should the time period be too low, wash out will occur and treatment of the incoming wastewater will cease to occur.

The sludge age within the system will determine the extent of treatment achieved since fast growing heterotrophs required for BOD removal only require a short sludge age whereas slow growing autotrophs required for nitrification require a much longer sludge age. Typical sludge ages for treatment are given below:-

	Sludge Age
High Rate Process	1 - 2 days
Conventional System	4 - 6 days
Conventional with nitrification	12 - 14 days
Extended Aeration	up to 50 days

Table 7.7 –Sludge Age for different treatment modes

Sludge Age or Solids Retention Time (SRT) can be based on either the total system volume or more normally on aeration tank volume.

Based on aeration tank volume,

$$SRT = \frac{V \text{ x } MLVSS \text{ in tank}}{(Q_w X_w) + (QeXe)}$$

Q_w = waste sludge flowrate, m^3/d
X_w = concentration of VSS in the waste stream g/m^3
Q_e = treated effluent flowrate, m^3/d
X_e = concentration of VSS in the treated effluent, g/m^3
MLVSS = mixed liquor volatile suspended solids, kg/m^3

7.3.4.3 Secondary Treatment – ASP Design

(i) Selector Zone

The raw sewage passes to the selector zone where it is mixed with the RAS prior to being split between the activated sludge lanes. Plug flow activated sludge plants have a high length to breadth ratio which means that there is a high initial concentration of substrate relative to the micro-organisms (MLSS). Even then however, the aeration volume is such that the concentration is not necessarily high enough to act as a proper selector. In the system selected for Bellozanne, the reactor configuration is such that the system is nearer to a completely mixed system than a plug flow one. This means that the substrate and the recycled activated sludge (RAS) are dispersed immediately into the tank contents resulting in a low concentration of substrate.

It is generally accepted that a high initial concentration of substrate relative to the MLSS can improve the settleability of the sludge and therefore a 'separate reactor', known as a 'selector zone' is positioned on the front end of the IBA ditch. This selector tank is designed such that the distribution chamber is incorporated within the structure such that an even flow split is achieved between the activated sludge lanes. The purpose of this selector zone is to provide a high floc loading where the floc loading is given by:-

$$\text{Floc Load (mg COD / g MLSS)} = \frac{\text{Mass of available COD in unit time at initial mixing}}{\text{Mass of MLSS in unit time at initial mixing}}$$

where BOD can replace COD.

The floc loading should be high enough to give a competitive advantage to the floc-forming organisms rather than the filamentous ones. A floc loading in the range of 50-150 mg COD/g MLSS is recommended in CIWEM's handbook 'Activated Sludge Treatment'.

Based on BOD, the suggested initial floc loading should be between 2-6 kg BOD/kg MLSS. d.

At Bellozanne the selector zone is sized to give the minimum retention of 5 minutes at maximum flow plus RAS. As the maximum flow is 830 l/s and the RAS rate required is 2550 m^3/h, the volume required is 460 m^3 which in turn gives a floc loading of 3.5 kg BOD/kg MLSS.day under the initial conditions currently seen and 3.9 kg BOD/ kg MLSS.day under the ultimate design load in 2035. The mg COD/g MLSS achieved is 115 which is well within the 50 to 150 range recommended by the CIWEM handbook.

This is adequate to achieve the required floc loading rate over the range of loads expected.

(ii) Process Capacity – Carbonaceous BOD Removal

Carbonaceous removal is less onerous than nitrification and utilises far less energy as the aeration requirement is far less. An F/M ratio of 0.3 kg BOD/kg MLSS/day will achieve a good quality effluent with no nitrification occurring in the winter and limited nitrification in the summer, depending upon BOD load and temperature.

Using the maximum BOD load of the settled sewage of 6323 kg/d and an MLSS of 3 kg/m^3 gives a volume requirement of 7030 m^3. However the plant needs to be able to achieve consent with one lane out of service in order to provide the required degree of robustness. Thus if three activated sludge lanes are selected then clearly two lanes must be able to achieve the level of treatment required. However to simply put in three lanes at approximately 3500 m^3 each would mean that under normal operating conditions, with all three lanes in operation, the MLSS would be 2000 mg/l (assuming that the F/M of 0.3 is maintained) at the maximum design load. This would result in a very poor achievable turndown by the system. Turn down of the MLSS must therefore be considered and this should not go below 1800 to 2000 mg/l otherwise poor floc density will be experienced with resultant settlement issues. The minimum load into the plant for prolonged periods of time is 2200 kg BOD/d (see Section 5.2.2 Flows and Loads) and the required MLSS during this period should not fall below 2000 mg/l in order to maintain a sensible F/M ratio.

Due to the above constraints the system design is based on a MLSS of 3500 mg/l at the maximum design loading when one activated sludge tank is taken out of service. This is comfortably achievable by the final settlement tanks as detailed in Section 7.3.5 and means that the turn down achievable by the plant is greater when all three lanes are in service. Clearly should lower BOD loads be experienced into the works for prolonged periods for any reason then a single lane can be taken out of service.

Using the above gives rise to three lanes, each with a process capacity of 2820 m^3 and therefore a total capacity of 8460 m^3. This capacity also lends itself better to the future potential upgrade requirements needed for the change in effluent consent standard as detailed in Section 8.

The F/M ratio calculated for the maximum BOD load in the years 2011, 2035 and the minimum BOD load of 2200 kg/d are shown below: -

	2011		**2035**			
	Max Design BOD Load		**Max Design BOD Load**		**Min Design BOD Load**	
	Max	Min	Max	Min	Max	Min
MLSS in ASP (kg/m^3)	2.5	2.5	2.5	2.5	2.0	2.0
Actual F/M ratio (d^{-1})	0.26	0.23	0.3	0.26	0.1	0.08
SRT (d^{-1})	4.6	5.4	4.1	4.7	14	16

Table 7.8 – ASP F/M ratios in normal operation

	2011		**2035**			
	Max Design BOD Load		**Max Design BOD Load**		**Min Design BOD Load**	
	Max	Min	Max	Min	Max	Min
MLSS in ASP (kg/m^3)	3.5	3.5	3.5	3.5	2.0	2.0
Actual F/M ratio (d^{-1})	0.28	0.25	0.32	0.28	0.15	0.13
SRT (d^{-1})	4.3	5.0	3.8	4.4	9	11

Table 7.9 – ASP F/M ratios with one lane out of service

This provides good turndown for the BOD loads that are expected to occur for prolonged periods and flexibility of operation. With all 3 lanes in service the MLSS is maintained at between 2 and 2.5 kg/m^3 and this will ensure a good level of carbonaceous treatment. It should be noted that the F/M ratio of circa 0.1 calculated at the minimum BOD load will lead to nitrification occurring should it occur for a long period of time. This is not a major problem and as the low BOD loads are expected in the winter when the tourist population is low and rainfall higher the low influent temperatures will not encourage nitrification to occur. However these minimum loads do indicate very low concentrations of BOD entering the works which in turn could lead to low floc densities and a lower robustness of plant operation regardless of biological system selected. This is discussed in Section 5.2.4.

As discussed in Section 3 due to the fact that there is such a wide range in the potential population growth a fairly conservative approach has been taken to the process design whilst ensuring that the structures are not oversized to the extent that the process performance will be compromised by biological underloading. The secondary treatment plant has also been designed to allow for future upgrading should the discharge consent be modified and this has influenced the initial sizing of the activated sludge lanes. To this end as detailed in the table above, the proposed design is such that an operating MLSS of 2.5 kg/m^3 can achieve the required F/M ratio for carbonaceous removal providing that all three ASP tanks are in service. The MLSS can be run as high as 3.0 kg/m^3 under design conditions and with one final tank out of service and therefore a 20% increase in BOD load can be treated to the required carbonaceous standard providing that all ASP lanes are in service. The future proofing of the design and its capacity to treat the increased load is covered in more detail in Section 8.4.

7.3.4.4 Process Description

The process selected for the activated sludge plant is the Inclined Bubble Aeration Process as utilised within numerous plants within the UK. This process uses a racetrack type configuration and incorporates banana blade flow inducers within the lanes to maintain the solids in suspension such that the air to the system can be optimised based on the treatment process rather than providing the mixing energy as well. The layout of this configuration is shown in Figure 7.3 (Appendix B).

In a conventional activated sludge plant, the aeration device is used not only to transfer oxygen but also to keep the biological solids in suspension. This often involves a compromise between aeration efficiency and mixing requirements thus leading to periods of over aeration and higher energy use as well as possible process issues associated with high dissolved oxygen concentrations. This is avoided with the IBA process due to the high turn down available in terms of the aeration input thus leading to savings in running costs.

In addition where the works can see periods of very low BOD load, as is the case at Bellozanne, the aeration input can be switched on and off to suit the air demand. This prevents over aeration and the resulting process issues that can follow that are common on low loaded works with conventional aeration control systems.

The IBA system uses membrane diffusers capable of 100% air turndown without the risk of water ingress. Mixing is by the use of slow-speed propeller flow inducers designed to provide sufficient velocity in the ditch to prevent any solids settling.

One of the main advantages of the IBA system is its simplicity in terms of dissolved oxygen control. A DO probe providing 4 - 20mA signal to switch fixed-speed blowers on and off is all that is required. The DO within the process is normally controlled within a band of 1-3 mg/l.

A DO meter is positioned in each IBA ditch. The air is input to the system by three positive displacement blowers, one dedicated to each lane with a common standby. The dedicated blower to each ditch is controlled by its corresponding DO probe.

An MLSS meter is also positioned within each IBA stream. The flow inducers are guide rail mounted such that maintenance and replacement can occur without taking the activated sludge tank off line.

7.3.5 Secondary Treatment - Final Settlement Tanks

The final settlement tanks are utilised to separate the activated sludge flocs from the mixed liquor to give a well clarified, stable effluent, low in BOD and suspended solids. Eight final settlement tanks are proposed for the treatment works although a four tank system has also been investigated.

7.3.5.1 Process Design

Final settlement tanks are designed based upon surface loading and solids flux theory which takes into account sludge settleability, the desired underflow concentration and solids mass flow to the tank. Both design bases are discussed below.

7.3.5.2 Design Criteria

Surface loading is the concept of a volume loading on the surface of a settlement area and defined as follows:

$$\text{Surface Loading } (m^3/m^2d) = \frac{\text{Maximum flow } (m^3/d)}{\text{Tank Surface Area } (m^2)}$$

Final settlement tanks are normally designed such that the upflow velocity does not exceed 1.2m/h with all tanks in service or 1.7 m/h with one tank out of service. For effluents with the possibility of high salinity spikes the upflow velocity reduces to 1.0 m/h with all tanks in service and 1.3 m/h with a tank out of service.

7.3.5.3 Design Options

Two options using either four final settlement tanks or eight final settlement tanks have been examined which require a similar land take. The eight tank option will require more structures and equipment such as distribution chambers, RAS sumps and pumps but will have less surface effects due to wind.

(i) Four Tank Option

The tanks at Bellozanne have been sized at four tanks each with a diameter of 34m.

This gives the following surface loading with all tanks in service and one tank out of service.

	Maximum Flow		Average Flow		DWF	
No. tanks in operation	4	3	4	3	4	3
Tank Surface Gross Loading (m^3/m^2.day)	20.2	26.9	10.2	13.6	8.2	10.9
Tank Surface Nett Loading (m^3/m^2.day)	21.0	28.0	10.6	14.2	8.5	11.3
Upflow velocity (m/h)	0.87	1.17	0.44	0.60	0.35	0.47

Note: Including inlet diffuser drum

Table 7.10 – Four FST Design Surface Loading Rates

Using mass flux theory to determine the solids loading levels on the FSTs

The system is designed based on an MLSS of 3 kg/m^3 when all activated sludge lanes are in service and 3.5 kg/m^3 when one activated sludge lane is out of service.

Normally if one activated sludge lane is out of service all FSTs will be in operation. Similarly if one FST is out of service then all activated sludge lanes will be in service. This is the basis for the following calculations.

1. One FST Out Of Service: - Under this scenario, three FSTs are in operation and the design MLSS in the activated sludge tank is maintained at 3.0 kg/m^3. The maximum flowrate to each tank is a third of 830l/s plus the RAS (maximum rate of 1.5 x DWF) giving a total flow to each tank of 1614 m^3/h. At the design MLSS of 3 kg/m^3 the applied solids flux to the secondary clarifiers is therefore 5.4 kg/m^2h.

   Solids loading levels for clarifiers can be calculated from mass flux theory and for the design SSVI of 120 the maximum solids flux that can be handled by the final settlement tank is calculated at 5.92 kg/ m^2/h. Thus there is a 9% contingency on the maximum flux rate possible with the tank design at 34m diameter.

2. All FST's In Service but One ASP Out Of Service: - Under this scenario, under normal operation (see below) four FSTs are in operation and the design MLSS in the activated sludge tank is maintained at 3.5 kg/m^3. The maximum flowrate to each tank is a quarter of 830l/s plus the RAS (maximum rate of 1.5 x DWF) giving a total flow to each tank of 1,210 m^3/h. At the design MLSS of 3.5 kg/m^3 the applied solids flux to the secondary clarifiers is therefore 4.66 kg/m^2h.

   Solids loading levels for clarifiers can be calculated from mass flux theory and for the design SSVI of 120 the maximum solids flux that can be handled by the final settlement tank is calculated at 4.87 kg/ m^2/h. Thus there is a 4% contingency on the maximum flux rate possible with the tank design at 34m diameter.

Note that the difference between the maximum solids flux that can be handled by the system with tanks in and out of service, is due to the fact that the RAS flow remains the same as the RAS is being pumped from a central RAS chamber. The system will therefore automatically maintain the RAS rate in relation to the inflow. The underflow rate seen in the final settlement tank will therefore change depending upon the number of final settlement tanks in service and thereby the maximum solids flux that can be handled.

(ii) Eight Tank Option

Should eight tanks be utilised then a diameter of 24.5m is required based on the design flowrates.

Based on the design flowrate and a diameter of 24.5 m this gives the following surface loading with all tanks in service and one tank out of service.

	Maximum Flow		**Average Flow**		**DWF**	
No. tanks in operation	8	7	8	7	8	7
Tank Surface Gross Loading (m^3/m^2.day)	19	21.7	9.7	11.4	7.9	9.0
Tank Surface Nett Loading (m^3/m^2.day)	19.8	22.6	10.1	11.5	8.2	9.4
Upflow velocity (m/h)	0.83	0.94	0.42	0.48	0.34	0.39

Note: Including inlet diffuser drum

Table 7.11 – Eight FST Design Surface Loading Rates

Using mass flux theory to determine the solids loading levels on the FSTs

The system is designed based on an MLSS of 3 kg/m^3 when all activated sludge lanes are in service and 3.5 kg/m^3 when one activated sludge lane is out of service.

It is assumed that if one activated sludge lane is out of service all FSTs will be in operation. Similarly if one FST is out of service then all activated sludge lanes will be in service. This is the basis for the following calculations.

1 One FST Out Of Service: - Under this scenario, seven FSTs are in operation and the design MLSS in the activated sludge tank is maintained at 3.0 kg/m^3. The maximum flowrate to each tank is a seventh of 830 l/s plus the RAS (maximum rate of 1.5 x DWF) giving a total flow to each tank of 692 m^3/h. At the design MLSS of 3 kg/m^3 the applied solids flux to the secondary clarifiers is therefore 4.4 kg/m^{2}h.

Solids loading levels for clarifiers can be calculated from mass flux theory and for the design SSVI of 120 the maximum solids flux that can be handled by a final settlement tank is calculated at 5.2 kg/ m^2/h. Thus there is a 15% contingency on the maximum flux rate possible with the tank design at 24.5m diameter.

2. All FST's In Service but One ASP Out Of Service: - Under this scenario, under normal operation (see below) eight FSTs are in operation and the design MLSS in the activated sludge tank is maintained at 3.5 kg/m^3. The maximum flowrate to each tank is an eighth of 830l/s plus the RAS (maximum rate of 1.5 x DWF) giving a total flow to each tank of 605 m^3/h. At the design MLSS of 3.5 kg/m^3 the applied solids flux to the secondary clarifiers is therefore 4.5 kg/m^2h.

   Solids loading levels for clarifiers can be calculated from mass flux theory and for the design SSVI of 120 the maximum solids flux that can be handled by the final settlement tank is calculated at 4.75 kg/ m^2/h. Thus there is a 5% contingency on the maximum flux rate possible with the tank design at 24.5m diameter.

3. It should be noted that due to the build sequence initially there may only be seven final settlement tanks. It is envisaged that these may need to operate with only two out of the three activated sludge lanes operating in which case they will need to operate at 3.5 kg/m^3. In this situation the maximum solids flux that can be handled by the final settlement tank is 4.75 kg/m^2h as calculated in 1 above but the applied load increases to 4.89 kg/m^2h and thus the system cannot operate with an SSVI of 120 ml/g. If a slightly better settling sludge is realised at 115 ml/g then the maximum solids flux that can be handled by the final settlement tank is 4.91 kg/m^2h thus giving a design contingency of 0%. Whilst this appears tight on the basis of these calculations it should be recognised that this is using the 2035 design flows and loads to the works. In reality the initial biological load to the works will be lower with the result that a lower MLSS would be acceptable. The SSVI of a well operating works is also below the 115 ml/g value. It is therefore considered that the use of 7 FSTs if required for a period time is acceptable.

4. Should a tank be taken out of service such that six final settlement tanks are utilised then all three activated sludge lanes must be in service in order to treat the design load into the works. At the operating MLSS of 3 kg/m^3, the maximum solids flux is 5.77 kg/m^2h at 120 ml/g SSVI and this gives an 11% contingency over the applied flux of 5.1 kg/m^2h. Six tanks cannot achieve the required MLSS should an activated sludge lane be out of service.

7.3.6 Tertiary Treatment

The current practice of UV will be continued with the new Bellozanne STW on the basis of health and environmental considerations. All flows received at the new Bellozanne STW will receive full treatment prior to receiving tertiary treatment in the form of UV disinfection before being discharged into St. Aubin's Bay to safeguard the bacteriological quality for bathing waters and shellfish beds.

The UV plant will be designed on a duty/ standby basis to treat up to 1,100 l/s with future increased flows in mind.

8. Future Bellozanne STW Process Design

8.1 Future Consents

The initial design is based on carbonaceous removal to achieve a BOD/ SS standard. In the future this may be altered such that an additional ammonia standard or even total nitrogen standard is required. The plant has therefore been designed with this potential future secondary treatment upgrade requirement in mind.

8.2 Ammonia Standard (Nitrification)

8.2.1 General

Nitrification is the biological oxidation of ammonia to nitrate, with nitrite formation as an intermediate. The micro-organisms involved are the autotrophic species Nitrosomonas and Nitrobacter which carry out the reaction in two steps:-

$$2NH_4^+ + 3O_2 = 2\ NO_2^- + 4H^+ + \text{new cells}$$
$$2\ NO_2^- + O_2 = 2\ NO_3^- + \text{new cells}$$

The extent of nitrification that occurs during treatment is dependent on the extent to which nitrifying organisms are present in the mixed liquor which in turn is dependent upon the sludge age, i.e. a minimum sludge age is calculated and the systems process requirements checked on this basis.

The nitrification process is sensitive to dissolved oxygen (DO) concentration, pH and temperature. A minimum DO concentration of between 1.0 and 2.0 mg is normally necessary.

It can be seen that the nitrification reaction causes a depression of pH because of the hydrogen ions liberated. If the pH depression is sufficient, it can reduce nitrification reaction and therefore chemical dosing may be required. A residual alkalinity as calcium carbonate of approximately 30-50mg/l should remain in the wastewater to ensure a stable pH.

The optimum temperature of nitrification is 25°C, although it may occur from 5 ° C to 45 ° C. As temperature decreases, reaction rate decreases. The system proposed is designed based on a minimum winter temperature of 12° C.

In order to achieve full nitrification an F/M ratio of 0.1 is required with a sludge age of at least 12 days.

8.2.2 Bellozanne STW Design

For a plant requiring an ammonia consent the IBA system can still be utilised to achieve the required consent concentration, however, due to the longer sludge age and lower F/M ratio required, the process capacity requirement increases. The layout of the additional activated sludge lanes required for nitrification is shown in Figure 8.1 (Appendix B).

In order to maintain ease of operation and flexibility in design the initial concept is to maintain activated sludge tanks of the same dimensions as utilised on the carbonaceous removal only design. This gives rise to 7 activated sludge lanes each with a process capacity of 2820 m^3 and this in turn gives rise to the following F/M loadings: -

	2011		**2035**	
	Max Design BOD Load		**Max Design BOD Load**	
	Max	Min	Max	Min
MLSS in ASP (kg/m^3)	3.0	3.0	3.0	3.0
Actual F/M ratio (d^{-1})	0.09	0.08	0.1	0.1
SRT (d^{-1})	17	18	15	16

Table 8.1 – Future Nitrifying ASP F/M ratios in normal operation

	2011		**2035**	
	Max Design BOD Load		**Max Design BOD Load**	
	Max	Min	Max	Min
MLSS in ASP (kg/m^3)	3.5	3.5	3.5	3.5
Actual F/M ratio (d^{-1})	0.09	0.08	0.1	0.1
SRT (d^{-1})	17	18	15	16

Table 8.2 – Future Nitrifying ASP F/M ratios with one lane out of service

Thus the F/M ratio achieved is sufficient that good nitrification will be achieved throughout the year.

8.3 Total Nitrogen Standard

8.3.1 General

Nitrogen entering a biological treatment system in the organic or ammonia form can be either removed or transferred to another form. Removal of nitrogen is obtained by assimilation and by conversion to nitrogen gas through nitrification and denitrification. Transformation of ammonia and organic nitrogen to the oxidised form of nitrate is accomplished through biological nitrification.

8.3.2 Nitrogen Removal by Assimilation

Since nitrogen is an essential constituent of microbial cells, any net growth of biomass that is removed from the waste stream will cause some nitrogen removal. The amount of nitrogen that can be removed by this mechanism is limited by the amount of growth which in turn depends up on the carbonaceous organic content of the wastewater and the systems operating conditions. Typically the nitrogen content of the biological suspended solids is around 8% and this value is used in the calculations.

8.3.3 Sludge Age

For the treatment of settled sewage (with no inhibitory wastewater present) in hard water areas, the relationship between sludge age and temperature is such that at 10°C the sludge age should be 12.5 days and at 20°C it should be about 6.6 days to achieve full nitrification. Allowing for the fact that the BOD reduction within the anoxic zone is reduced such that 1 mg/l of NO_3-N will satisfy a BOD of 2.83 mg/l, the actual BOD to the aerobic zone can be calculated at 6,000 kg BOD/d. Assuming an overall F/M ratio requirement of 0.08, an overall volume of 21,000 m^3 is required. This equates to a sludge age during the winter of 13.8 days which is satisfactory.

8.3.4 Denitrification

Denitrification is the biological conversion of nitrate-nitrogen to more reduced forms such as N_2, N_2O and NO. The process is brought about by a variety of facultative heterotrophic organisms which use organic carbon both for energy and as a carbon source. Molecular oxygen is utilised by these bacteria under aerobic conditions but under anoxic conditions, the fixed oxygen present in the NO_3 is used in the following reaction:-

$$6\ NO_3 + \text{organic carbon} = 5CO_2 + 3N_2 + 2\ H_2O + 6\ OH$$

It can be seen that this reaction releases hydroxide ions (OH) which increase the alkalinity of the wastewater thus acting to buffer the pH of the nitrification process.

To obtain reliable denitrification it is necessary to bring the required facultative bacteria into contact with nitrate under anoxic conditions with an adequate concentration of readily biodegradable organic substrate.

The rate of denitrification is dependent upon the temperature and DO concentration as follows:-

$$R_{DN(T)} = R_{DN(20)} \times 1.07^{(T-20)} \times (1-DO)$$

where:-

$R_{DN(T)}$ =	Rate of denitrification at temperature T, mg NO3-N/g MLSS d
$R_{DN(20)}$ =	Rate of denitrification at 20 C, mg NO3-N/g MLSS d
T=	Sewage Treatment, °C
DO=	Dissolved Oxygen level within anoxic reactor, mg/l

For most applications it can be assumed that the DO level within the anoxic zone is zero. The denitrification rate is dependent upon both the concentration and biodegradability of the carbon source. For design purposes the following can be used:

Low rate : $R_{DN(20)}$ = 60mg NO3-N/g MLSS d

Average rate : $R_{DN(20)} = 80mg\ NO3\text{-}N/g\ MLSS\ d$

In order to determine the aerobic and anoxic volume required, a nitrogen balance is carried out. The anoxic zone is then obtained by the following equation:-.

$$\text{Volume} = \frac{N_{DN}}{MLVSS\ x \frac{R_{DN(T)}}{1000}}$$

where:-

N_{DN} = N used for denitrification, kg N/d
MLVSS = mixed liquor volatile suspended solids, kg/m^3
$R_{DN(T)}$ = rate of denitrification at temperature T.

8.3.5 Bellozanne STW Future Modifications

A temperature range of 12 to 25 degrees Celsius is used for the design. This gives an estimated sludge yield of 0.4 to 0.7 and an associated sludge production of 2,530 to 4,400 kg DS/d. Assuming that the percentage volatile suspended solids in the sewage is 70% and the nitrogen content per kg VSS is 10%, the nitrogen used in the sludge production ranges from 180 to 352 kg VSS/d. This means that the nitrogen used for denitrification is calculated at 1,350 kg N/d which results in an anoxic volume requirement of 14,700 m^3. In order to maintain the required F/M ratio an aerobic volume of 13,500m^3 is required which gives a total activated sludge tank capacity of 28,200 m^3. This would require 10 activated sludge lanes should the capacity of each lane be the same as for the initial carbonaceous removal design.

The total nitrogen standard requires a different tank layout than proposed for the other two consent options. For Carbonaceous with or without nitrification the IBA system as described above provides a reliable, robust and flexible method of treatment. However due to the high recycle rates required within the total nitrogen system and the need for true anoxic conditions; a plug flow type arrangement is better suited for this standard. For this reason the IBA lanes are designed such that the flow enters and departs from the same end of the tank such that they can be easily converted to a U shaped, plug flow plant in the future should it be required. The additional tanks required would obviously be designed in the plug flow configuration with separate anoxic and aerobic zones within the activated sludge lane. The layout required of the activated sludge lanes is shown in Figure 8.2 (Appendix B).

8.4 Future Proofing

The plant has been designed as detailed above to allow for future expansion should the consent standard required of the effluent become more stringent. However, this Section looks at how the works would cope or what expansion to the works would be required should the flow and load to the works increase by 20%.

The primary settlement tanks are based on a surface loading rate of 45 $m^3/m^2/d$ which is common UK practice and will deliver a removal of between 50 to 65% Suspended Solids (SS) and an associated insoluble BOD removal of between 25 and 35%. Using this and the design Flow to Full

Treatment of 830 l/s results in a requirement for four primary tanks each of 23m diameter. If this loading rate was still required with the additional 20% future expansion, then either a fifth primary settlement tank of 23m diameter would be required or the four new primary settlement tanks could be initially built at 24.5m diameter which in effect gives the required loading should the future increase in flow and load be realised. This gives an initial loading on the primary settlement tanks of approximately 38 $m^3/m^2/d$, which is satisfactory. The proposed layout is based on the larger 24.5m diameter tank.

The alternative would be to leave the primary settlement tanks at 23m diameter and accept a higher hydraulic loading on them at the higher flows. For the four 23m diameter primary settlement tanks the hydraulic loading would increase to 53 $m^3/m^2/d$ which is still within acceptable design limits. The SS and thereby the insoluble BOD removal efficiency will reduce at the higher flowrates such that the BOD removal achieved will be nearer the lower end of the expected design range. However, the activated sludge plant is designed for this range so it would not affect the plant capacity although there may be a slight increase in operating cost as the average BOD to the secondary treatment would be higher.

The proposed capacity provided by the three activated sludge lanes can treat the 20% increase in capacity providing that all lanes are operational. This is achieved by increasing the operating MLSS concentration which would still be well within that achievable by the proposed final settlement tanks. However should one activated sludge lane be taken out of service then the F/M ratio would increase to 0.38 at an MLSS of 3.5 kg/m^3 (the design value for the FSTs). This is too high to reliably achieve the carbonaceous removal required to achieve the effluent consent. Approximately an extra 15 to 20% in activated sludge tank capacity is therefore needed. Therefore a single additional lane would comfortably provide the necessary treatment capacity to achieve the required carbonaceous removal.

It is not advisable to build this additional capacity into the initial plant sizing as it would lead to an oversized plant for the initial design loads which would in turn lead to low MLSS requirement and in turn settleability issues due to the low floc density realised. This could lead to solids loss from the final settlement tanks when high flows to the plant occur and a resulting deterioration in effluent quality.

It is therefore proposed that a single additional aeration lane be constructed if and when the flows and loads are identified as being consistently above the design values at some point in the future. Normal monitoring of the works should recognise the trend for increasing MLSS over time which would trigger the need for the fourth lane to be built to allow maintenance to take place. This trigger may also be linked to early indicators such as future census results projecting populations in excess of 118,000.

Assuming that the additional activated sludge lane is of the same capacity, the loading seen is as follows: -

	All 4 Lanes in Service		1 lane Out Of Service	
	Max	Min	Max	Min
MLSS in ASP (kg/m^3)	2.5	2.5	3	3
Actual F/M ratio (d^{-1})	0.27	0.25	0.3	0.28
SRT (d^{-1})	4.6	5	14	15

Table 8.3 – Future ASP F/M ratios with 20% extra load

The final settlement tanks are designed based upon a surface loading of not greater than 24 $m^3/m^2/d$ and solids flux basis suitable to achieve a MLSS of 3.5 kg/m^3 with all final settlement tanks in service and 3.0 kg/m^3 with a single final settlement tank out of service. This means that should the final settlement tanks be designed with the future increase in flow of 20% in mind then either 4 tanks of 34m diameter or 8 tanks of 24.5m diameter would be required. The 8 tanks of 24.5m diameter are therefore suitable for the possible future increase in flow and load. It is proposed to incorporate 8 tanks of 24.5m diameter as part of the initial development, although the comments in section 7.3.5 regarding initially building seven tanks would still apply.

It should be noted that an increase in flow and load in the future will give rise to a larger quantity of sludge produced. A 20% increase in loads will result in a pro-rata increase in sludge make approximately and the impact on the pasteurisation and anaerobic digestion plant will need to be assessed.

The sludge plant has been designed based on 6,175 kg DS/d of primary sludge and 4,400 kg DS/d of SAS. The calculations predict a primary sludge load of 6,175 kg DS/d at the maximum SS removal rate across the primary tank and a SAS production of 4,460 kg DS/d at this level of primary tank performance. These are within about 5% of the original design values which is reasonable as they are maximum values. However an increase of 20% would give approximately 7880 kg DS/d of primary sludge and 5350 kg DS/d of SAS. This is clearly greater than the current design basis and whilst there is spare capacity within the sludge plant design, should the extra 20% load become a reality then the effect on the sludge plant would need to be examined further.

9. New Bellozanne STW

9.1 Layout Development

In developing the design of the plant the main considerations for the layout of the new works are: -

- Process design and performance
- Hydraulic constraints
- Access and maintenance
- Traffic movements
- Site specific constraints

The initial thoughts for Bellozanne STW centred around the eastern valley with the inlet works at the head, primary settlement tanks adjacent to the inlet works with the activated sludge plant to the south east of the new sludge works and the final settlement tanks between the ASP and the valley road. The layout has now developed over a number of revisions and the agreed layout, taking into account the main considerations noted above, is shown in Figures 9.1 & 9.2 (Appendix B). The cross sectional views are shown in Figure 9.3.1 and the longitudinal hydraulic profile is shown in Figure 9.3.2 (Appendix B).

The new site layout is based on carbonaceous removal to achieve a BOD/ SS standard for a connected population equivalent of 118,000 for the design horizon. The site layout has been developed to ensure that the existing STW continues to remain in service until the process units are replaced.

The overall site layout makes provision for potential future secondary treatment upgrade requirements such that an additional ammonia standard or even total nitrogen standard can be incorporated.

9.1.1 Inlet rising mains

These have been located within the excavated area around the southern edge of the site. This has been done to enable these to be installed during the enabling works as space is limited during the construction of the works.

9.1.2 Inlet works

The choices with the location of the inlet works are as follows.

1. To place this building near to the road so that access for screenings and grit removal skips from site is optimized would require interstage pumping. There are also advantages should a reception facility for imports be installed near the inlet works as access for tanker deliveries would be good without the need for them to enter the main works.

2. The alternative would be to use the levels within the site to enable the works to operate under gravity for the complete process thus eliminating any risk associated with pump failure

and overflows of sewage. However tanker access to the inlet works, which would be positioned at the north east end of the site, is less than ideal.

Although the overall pumping head would be similar it was decided that as the existing First Tower pumping system can deliver flows to a raised inlet works, a fully gravity driven system at the Bellozanne site would be preferred. The inlet works has been placed near to the new sludge works with an appropriate roadway and vehicle turning area. This location allows the storm tanks to be located adjacent to the inlet works with a simple weir arrangement for the separation of storm flows and a short pumped return for storm water.

The inlet works and the storm tanks will be constructed once the Accommodation Works are completed.

9.1.3 Primary Settlement Tanks

Radial tanks have been chosen for this layout although rectangular type units were also considered. The land take between the two options is not significantly different and the radial flow tanks will lend themselves to easier sludge removal while rectangular tanks have inherent difficulties with scraper performance. The tanks are located near to the inlet works and storm tanks thus minimising the length of large bore pipework runs. The tanks are also conveniently located to allow the primary sludge to be easily pumped to the new sludge tanks and minimise sludge delivery pipework.

The primary settlement tanks will be constructed and commissioned together with the preliminary treatment units. Settled sewage will then be transferred to the existing ASP prior to decommissioning of the existing inlet works and primary settlement tanks.

9.1.4 Activated Sludge Plant

The initial treatment stage of the new ASP has been located on the site of the existing primary tanks to enable the existing ASP to remain operational during the phased construction of the new plant. This allows the amount of temporary works that will be required during construction to be minimised. The new air blowers are located in between the treatment Stages 1 and 2 ASPs with the new power generation building alongside. This is to minimise the lengths of large bore pipes and large cables as the blowers are expected to be the largest power user on the site.

A temporary selector tank will be required as the permanent tank will be located on the site of the old ASP for flow distribution reasons. The later treatment stages of ASP can then be constructed as and when required without materially affecting the operation of the works due to the access from the north side of the site. Access for maintenance is from the internal road system.

9.1.5 Final Settlement Tanks

These have been located in the south west of the site near to the re located UV building and outfall. This again has been done to minimise the large bore pipework runs. The tanks have been arranged to ensure good flow distribution and access for maintenance. The current layout does require a new access onto the valley road to be created although the alignment of the road is not changed. The new entrance location will have better visibility than the existing site entrance.

Radial flow tanks are utilised as sludge withdrawal and recycle, which is vital to the operation of the biological process, is unreliable in rectangular tanks.

Some of the final settlement tanks can be constructed concurrently with the primary settlement tanks providing cost efficiencies but the commissioning will take place as part of the new activated sludge plant.

9.1.6 Tertiary Treatment

The existing UV disinfection will continue to remain in use until the commissioning of the whole of the new STW and then be replaced by a new UV plant located adjacent to the Mechanical & Electrical Workshop.

Provision will be made in the site layout for any further tertiary treatment requirements to the upstream of the UV disinfection.

9.2 Land Requirements

Based on the selected technology and the availability of land within the Waste Management Services site, additional land purchase for the New Bellozanne STW is not anticipated at this stage based on the anticipated geotechnical conditions (see 9.3.1 below).

9.3 Demolition, site clearance and rock removal and stabilisation

In view of the constraints of the existing site, it will be necessary to enlarge the existing operational site area to accommodate the new works. This Section considers the project requirements in terms of demolition of existing structures, site clearance and the excavation and support of the existing slopes around the perimeter of the site to enlarge the area to accommodate the new works.

Demolition of all the above ground buildings is necessary, with the exception of the existing Sludge Thickening & Dewatering Building.

Site clearance will require the re-location of several activities that the TTS operate from the Bellozanne Waste Management Services site, including the public waste reception facility and scrap yard. It will also require the detailed, temporary and permanent provision of services to the various sections of the plant that will remain operational whilst the new plant is being constructed. These factors are given detailed consideration in Section 9.4.

9.3.1 Constraints

The existing operational site is insufficient to accommodate the new STW. In order to increase the site footprint, the existing wooded slopes surrounding various sections of the site will need to be excavated back and stabilised. The new STW will be sited within the existing site boundaries without the need for any land purchase or re-alignment of the main Bellozanne Valley Road although a new site entrance is required.

An ecological perimeter buffer zone of a minimum of ten metres in width will be maintained around the site boundary, which will consist of undisturbed woodland.

In order to locate the proposed works within the existing boundaries of the site and maintain the ecological buffer zone, the excavated profile of the slopes must be close to the vertical. It will not be possible to utilise slopes at 45° with benches at six metre vertical intervals with respect to slope stability. Detailed site investigation works and geotechnical mapping of the existing slopes will be carried out as part of the detailed design.

9.3.2 Assumptions

Although site investigation works and geotechnical mapping of the existing slopes has not been carried out to date, the geological units comprising the site are well understood from the recent sludge digestion project and do not vary significantly on a local scale. This feasibility study is therefore based upon conditions that are likely to be encountered but makes no allowance for conditions that cannot be reasonably foreseen at this stage in the absence of specific exploratory work and mapping.

It is noted that the proposed STW is constructed broadly on three level platforms at 20.000, 25.000 and 30.000 AOD. A cut and fill exercise has not been carried out in respect of this element of construction.

A volumetric assessment of the material cut from the slopes in the construction of the stabilisation works has been made. Again, this has not been considered in the overall cut and fill balance for the works. It is assumed that material arising from the excavation and slope stabilisation works will be exported from the site, as this phase of the works is likely to be undertaken in advance of the construction of the new STW. It is not considered to be advantageous to store excavated material from the stabilisation works on site, even if it may be a component of the overall fill balance.

In the absence of bespoke site investigation data and geotechnical mapping, the current profile of the existing slopes, failures of the slopes that have occurred historically and an appraisal of exposed rock faces has been the primary method for the assessment of stabilisation measures for this feasibility study.

9.3.3 Ground Conditions, Methodology and Stabilisation Categories

The geology of the site comprises the Jersey Shale Formation. This is a thickly and thinly laminated interbedded sequence of siltstone, sandstone and mudstone. A sedimentary rock, the bedding planes dominate discontinuity, but other discontinuity sets exist which leads to the entire structure of the rock being intensely fractured. Whilst the compressive strength of the rock is moderate, up to 50MPa, the degree of fracturing enhances the excavatability of the rock. However, it also contributes to instability of excavated faces, with toppling, planar failure and wedge failure all being potential modes of instability.

Whilst the dip and dip direction and the orientation of the major joint sets do not vary significantly across the site and faulting occurs in an east to west direction, the alignment of the proposed excavated faces varies significantly. As a result, the likely mode of failure, and hence the stabilisation measures that are required change along the length of the excavated face. There is evidence of loss of fault material between exposures of competent rock on the site. The heavily wooded nature of the majority of the proposed excavation prevents a proper assessment of the frequency of this failure mode at this stage, but an allowance has been made.

In order to provide the required footprint for the new STW, yet minimise the land take, the excavated volume and the area of support provided, the profile of the slopes would be vertical. This is not considered to be practical, and by reducing the angle of the slopes from the vertical to 10° from the vertical, the risk of failure of toppling, planar or wedge reduces significantly and the aesthetic appearance is improved.

The excavated profile of the newly developed slopes varies between 10m and 18m. It is therefore considered that the stabilisation methodology should be based upon a top down approach, so that slopes are stabilised as soon as possible after they are exposed.

It is noted that the upper platform of the site, occupied by the proposed inlet works and offices is of the order of five metres below the existing ground level. Whilst stabilisation of the slopes surrounding this area can be carried out in advance of the main works, when this area is excavated to its final level, the additional area of slope that is exposed will need to be stabilised. The top down method of construction allows this to be carried out without the need for re-working of the previous slope stabilisation works.

The final design of the stabilisation works will be based upon a full suite of site investigations and geotechnical mapping. The installed support measures will be based upon observation of the exposed geology. For the purposes of this feasibility study, this approach is obviously not possible. For this reason, a more conceptual approach has been adopted based upon the installation of three categories of systematic support. The three categories of stabilisation measures that are considered to be required for the development of the excavation are as follows (See Figures 9.4 and 9.5 (Appendix B) for details):

1. Category 1, the most robust category, consisting of reinforced concrete panels with double corrosion protected ground anchors on a 4m grid horizontally and vertically. Weepholes at 6m centres will be drilled at FGL +1m.

2. Category 2, consisting of double corrosion protected ground anchors on a 4m grid horizontally and vertically with 1m by 1m concrete plinths, shotcrete, mesh and dowels at 2m centres vertically and horizontally. Weepholes at 6m centres will be drilled at FGL +1m.

3. Category 3, consisting of double corrosion protected ground anchors on a 4m grid horizontally and vertically with 1m by 1m concrete plinths and mesh, locally pinned with dowels at 2m centres vertically and horizontally.

This arrangement of systematic ground anchors with 1m by 1m concrete plinths at the anchor heads form "virtual buttresses" in this fractured yet mechanically (reasonably) intact rock, without the need for the reinforced concrete buttress. Additionally, this method allows top down construction.

The extent and category of stabilisation required has been determined by assessment of the rock faces as well as the verticality of the cut faces and the type of support (if any) that has been provided in the past. It is to be noted that the slopes at the southern end of the site stand almost vertically without support, although failures associated with faulting are apparent.

9.3.4 Demolition

Buildings for demolition have been divided into two categories, temporary and permanent structures. The temporary structures comprise the portakabins, kiosks and other insubstantial structures on the site. The permanent structures for demolition comprise the building housing the standby generator sets, Clinical Waste Incinerator, Weigh Bridge Building and Old Power Station Building. Demolition of the Old Energy from Waste Plant is currently underway and does not form part of this feasibility study.

9.4 Site Services

This Section identifies the current services on the Bellozanne STW site and the requirements for service diversions to facilitate the construction of the new STW upgrade.

9.4.1 Existing Services

The existing services based on the drawings currently available are shown in Figures 9.7.1 and 9.7.2 (Appendix B) and summarised below.

9.4.1.1 General

The old Energy from Waste plant on site is currently being demolished. All other buildings and structures will be demolished and removed in a sequence to suit the phasing of construction of the new STW. The Public Waste Reception Facility and Scrapyard will also be relocated off site as part of the Bellozanne Waste Management Services site development to accommodate part of the new STW. The Clinical Waste Incinerator currently in operation at the site is programmed to be relocated under this project.

9.4.1.2 Foul and Surface Water Sewers

Site Foul Drainage

The existing foul sewer on site picks up both sewage from toilets and potentially contaminated water from process areas and discharges to one of two pumping stations on the site and then discharge to the inlet works. There are also some sewers that are connected to the foul sewer on site; from the houses on the hillside above the old EFW; and, from the coal yard pumping station. The St John's gravity main discharges directly to the inlet works.

Surface Water Drainage

Surface water from the site is collected in the surface water drainage system and discharged into the final effluent culvert. In addition, the surface water runoff from the valley and the natural brook from the valley are routed into the site surface water drainage system from three locations at present.

9.4.1.3 HV Electrical Supply (JEC and TTS)

The HV electricity supply to the site is from the JEC. In addition, there are a number of HV cables by JEC that run through the site and up the hillside to supply other areas outside the site. TTS uses a number of transformers to transform this HV from 11kV to 3.3kV and distributes around the existing site via a ringmain system. This ringmain allows additional flexibility and security to the site's infrastructure.

9.4.1.4 LV Electrical system

The 3.3kV HV electrical supply is transformed down to 415V which then feeds a number of MCCs. From these MCCs, each individual item of equipment is fed. Record drawings are available on the feed from each MCC.

9.4.1.5 Effluent Wash Water System.

The current effluent wash water system draws effluent from the final settlement tanks and pumps it to wash water storage tanks located near MCC7 after filtration and UV disinfection. This wash water is then pumped to a ringmain system and distributed around the existing STW and the up the road to the Sludge Thickening & Dewatering Facilities.

9.4.1.6 Potable Water

The existing potable water main comes through the main gate and then follows the road up to the Clinical Waste Incineration plant and is considered to be suitable for the new STW.

9.4.1.7 Telecoms and Communications

Currently the Weighbridge Building acts as a hub for all the site IT and JT cables. Whilst the JT cables are perhaps less significant as most of the site phones are now VOIP and rely on the IT system. The fibre optic network on the other hand uses the Weighbridge Building as a server room and hub for the Bellozanne STW network.

9.4.1.8 Process Lines

There are a number of process pipelines which currently run between the Sludge Thickening & Dewatering Facilities to the existing inlet works and sludge storage tanks. These lines are likely to become redundant with the completion of the Centralised Sludge Treatment Facilities – Phase 2.

9.4.2 Proposed Services

9.4.2.1 General

The proposed modifications and diversions of the existing services will require careful planning, investigation and execution so that the existing works can remain operational during the accommodation works and construction phases of the project. These are summarised below.

Prior to the demolition of the building housing the standby generator sets, it will be necessary to either relocate the generator sets to a suitable location or replace them with new generator sets that are more suitable for the future requirements.

9.4.2.2 Foul and Surface Water Sewers

Site Foul Drainage

As the new inlet works will be higher up the valley, the coal yard pumping station and St John's Gravity Main will need to be rerouted to the new inlet works. The sewer from the houses and the site sewage system will need a new pumping station which will pump to the new inlet works.

Surface Water Drainage

Special design consideration will be required for the diversion of the surface water drainage especially during construction of the inlet and primary tanks as these are being built over the existing surface water sewers. It is anticipated that the brook will use a new culvert system running to the south of the new inlet works, two of the primary settlement tanks and the final settlement tanks before discharging into the outfall as before.

9.4.2.3 HV Electrical Supply (JEC and TTS)

The existing HV electrical supplies (JEC supply and TTS ringmain) will need to be diverted around the new STW so as not to interfere with the construction. The JEC supply should be redirected to tie into a suitable location on the southern site boundary. The JEC have indicated that their cable must be laid in a cable culvert and not in cable ducts. This culvert will also be utilised for the TTS ringmain.

Further consideration will be given to the modification of the TTS ringmain design to incorporate the existing transformers and MCCs required for the new facility. Currently transformer T2 and its switch gear are sited at the old power station building and will need to be relocated to facilitate the demolition of the building prior to commencement of construction activities for the new STW. A further transformer, T8, adjacent to the existing primary tanks and ASP will also need to be relocated/replaced to tie in with the construction of the temporary flow diversion chamber prior to the existing ASP.

9.4.2.4 LV Electrical system

Based on the record drawings that are available for each MCC, routing of the cables will require further consideration to assess the impact on the construction of the new STW. Any disused equipment together with the associated local cables will be removed prior to construction. However, each area would need to be checked that all cables are dead before commencing construction in the area.

9.4.2.5 Effluent Wash Water System.

There are a number of areas where the wash water pipework will interfere with the construction of the new STW. The existing wash water pipework is mostly underground and old with a long history of failures. It is therefore necessary to have a new final effluent ringmain system for the new plant with the draw-off from downstream of the new UV plant with a preferred option of having the pipework mostly above ground. A new wash water pumping station and main will be required to pump wash water to new storage tanks as the existing storage tanks are considered to be too remote from the new STW. Temporary diversions of the existing system will be carried out during the accommodation and construction works such that the system can remain operational until the new system is complete.

9.4.2.6 Potable Water

The incoming potable water main will be diverted to avoid conflicts with the new STW process units and extended to the new administration facilities.

9.4.2.7 Telecoms and Communications

Prior to the demolition of the Weighbridge Building it will be necessary to divert the fibre optic network as well as the IT server to a new location.

9.4.2.8 Process Lines

The primary concerns in relation to the proposed process pipelines are the new rising main extensions and the feed pipes from the new PSTs to the existing ASP in the first phase of construction.

There are a number of congested service corridors identified which require careful consideration of how the new services will be phased during construction, particularly in the area between the existing inlet works and the Sludge Thickening & Dewatering Building.

The diversion of the twin rising mains from the First Tower Sewage Pumping Station will be along the southern hillside on completion of the hillside excavation towards the new Inlet Works. The connection to the final effluent outfall will be made with the new discharge chamber constructed on the existing outfall.

It is imperative that the exact locations of all the HV cables in the affected area are identified as early as possible to ensure that the new service corridors are available for the start of construction.

9.4.2.9 Summary

A summary of the issues to be considered as part of the design of the new STW is given on the following table.

Service Description	Issues	Temporary Supplies
Standby Generators	Prior to the demolition of the building housing the standby generator sets, it will be necessary to either relocate the generator sets or replace them with new generator sets for the future requirements in a suitable location.	
Surface Water Drainage	The current surface water sewers will be significantly impacted by the new STW. Consideration of routing of new sewers/culvert will be required.	A temporary surface water sewer will be required to take surface water from the site and the valley during construction.
Foul Sewers	Existing foul sewers on site will be impacted significantly. It will ultimately require a pumping station to pump to the new inlet works.	Temporary foul sewer diversion will be required during construction.
St John Sewer	This will require rerouting to the new inlet works.	
Coal Yard Pumping Station	This will require rerouting to the new inlet works.	
JEC HV and Site Ringmain	The current JEC cables will need to be rerouted prior to construction. Consideration will also need to be given to repositioning T2 and its switchgear as it is currently located at the old power station which is to be demolished. The design of the HV cable trench will also need to consider the routing of the new and rerouted pipelines.	
LV Power Supply	Essential supplies to the existing plant will need to be considered.	Temporary power supplies to various MCCs may be required.

Service Description	Issues	Temporary Supplies
Wash Water System	The current wash water ringmain is likely to be in the way of the new pipelines.	Temporary lines may be required to maintain current supplies until changeover to new STW.
Potable Water	Potable water main will require rerouting and extending to new administration facilities.	Temporary supplies may be required to maintain supplies to existing areas.
Fibre Optic Cables / Computers/ Telephones	Existing computers / fibre optic cables need to be relocated. Hub at weighbridge will need to be relocated.	Temporary cables and/or a server room may be required to maintain service to existing areas until decommissioning.

Table 9.1 – Issues with Services

9.5 Planning/EIA

It is recognised by TTS that the Bellozanne STW renewal constitutes Environmental Impact Assessment (EIA) development. As such, an Environmental Impact Statement will be prepared and submitted alongside the planning application for the project. Such an approach is in accordance with the requirements of Article 13 of the Planning and Building (Jersey) Law 2002 (Jersey Law) and the Planning and Building (Environmental Impact) (Jersey) Order 2006 (the EIA Order 2006).

As a result of the above requirement and the potential for the project to result in significant environmental effects, a high level review has been undertaken of readily available existing environmental data and reports in relation to the site, to identify likely environmental constraints which should be fed into the STW design.

The proposed development of the Bellozanne STW) site has the potential to result in a number of environmental impacts during the construction and/or operational phases. These are considered individually below:

9.5.1 Site Management

Suitable controls and management procedures will need to be in place to ensure that the operation of the STW at Bellozanne is undertaken in accordance with current environmental legislature and best practice requirements. An environmental aspects and impacts assessment will be undertaken to ensure that all hazards are identified and mitigated against. In the event of a breach of these mitigation measures it is possible that a detrimental environmental incident could occur. Sewage, sludge, wastes, treatment chemicals and fuel are all stored on site and may have the potential to act as sources of pollution in the event of handling or storage failures. Such failures may result in emissions to ground and in the event that the areas are not appropriately bunded/ intercepted, may result in pollution of controlled waters or underlying soils. However, appropriate control measures such as secondary containment will be implemented to lower the risk of such events occurring.

9.5.2 Biodiversity

The mature vegetation surrounding the site is classified as a Biodiversity Action Plan (BAP) habitat[22] and contains a variety of protected species (such as bats and red squirrels). Aside from the removal of this habitat through the excavation works, there is also potential for works in adjacent areas to disturb these protected species and habitats. Where applicable, controls and mitigation measures will be required, for example, avoiding bird nesting season, works on / near hedgerows and on or near trees subject to Tree Preservation Orders. These issues will be reviewed as part of the Environmental Impact Statement.

9.5.3 Population

The southern portion of the Bellozanne STW is located adjacent to residential housing which is within close proximity of nearby residents and thereby requires mitigation measures to control potential impacts during both the construction and operation phases. However the new sewage treatment facilities are proposed to be positioned at the centre of the existing development and therefore do not encroach further toward existing residential development. Early and ongoing engagement with local residents will be required to minimise potential disruption and ensure that where some disruption cannot be avoided, it is well communicated to reduce the impact on local residents.

9.5.4 Human Health

No study of potential human health impacts associated with the redevelopment of the Bellozanne STW has been undertaken at this stage. However, since the work involves the replacement and improvement of an existing facility, it is considered unlikely that there will be a net detrimental impact on human health.

9.5.5 Soil/ Geology/ Contaminated Land

The majority of the proposed works is within the existing STW footprint; however the new STW requires the use of additional land obtained from the re-profiling of an adjacent hillside. Due to the historic use of the area for open cast mining and the existing Energy from Waste Plant (decommissioned), Clinical Waste Incinerator and STW, there is significant potential for contaminated soils to be present. An assessment of potential risks associated with historic land use, existing land quality, ground conditions and proposed land use will be carried out in the next stage of the feasibility study.

9.5.6 Water

There are no natural surface water features on the STW site itself although two ponds lie to the east of the site boundary. Due to the close proximity of nearby ponds situated above the level of the site, mitigation measures will be required to control potential impacts during both the construction

[22] Mixed woodland is a designated BAP habitat, in accordance with Biodiversity: a strategy for Jersey (see particularly pages 5-6 and Table 3.1)

and operation phases. The design of the facility should include appropriate mitigation measures against potential water pollution incidents associated with spillages, vandalism and storage failure such as interceptors, spill kits and drain covers No assessment of the groundwater regime at the STW site has been undertaken at this stage.

9.5.7 Air

Appropriate controls and management techniques will be required to minimise potential impacts to air quality. In the event of a failure of these systems (e.g. gas storage area, flare and boilers) there is the potential for point source and fugitive emissions to air with a resulting potential reduction in local air quality and potential nuisance associated with odour.

9.5.8 Climatic Factors

The operation of the proposed STW requires less energy due to more efficient design and the application of new technologies this reduces the indirect environmental impacts associated with the depletion of natural resources and release of Greenhouse Gas (GHG) to the atmosphere.

The construction and operation of the proposed STW will unavoidably continue to include many activities which all involve the generation of GHG either directly or indirectly such as the operation of the waste gas burner, transportation of materials and wastes and purchase of parts for maintenance.

9.5.9 Material Assets

By maximising the design life of some elements of the existing STW, the project is able to reduce both the volume of waste generated and the amount of virgin materials required for construction of new infrastructure. For example, phasing the construction and utilising the existing aeration tanks will reduce the hillside excavation for the new STW reduces the requirement for construction of the new STW with associated resources, energy input and waste generation.

The existing STW will be replaced in advance of reaching the end of their operational life. This will act to avoid catastrophic environmental incidents related to equipment failure due to reaching the end of its life.

9.5.10 Waste

Construction will involve the generation of significant volumes of waste material due to the removal of part of the hillside. This will involve the excavation of rock and its transportation off-site with associated GHG emissions. It is understood that there are plans for excavated material from the hillside to be recycled on and off the existing site. Any head deposits encountered are likely to comprise shallow layers of silty, sandy gravel and it is likely that <50% will be reusable as general fill in the 'as dug' condition. The rock identified on site is an argillaceous rock which is likely to be suitable in its ‘as dug’ condition as general fill. However, the rock is unlikely to be suitable as structural fill as it could be prone to softening. Given the fractured nature of the rock it is likely that some processing, in the form of crushing, will be required prior to reuse.

A Site Waste Management Plan will be required to ensure that construction and demolition waste generated on site is managed to reduce volumes disposed to landfill by encouraging recovery/re-use and ensuring high recycling rates.

9.5.11 Nuisance

The new STW is generally located within the existing STW site footprint and therefore does not have residential neighbours within immediate proximity. However, there are a number of potentially sensitive receptors around the site including:

1. Local residents adjacent to and overlooking the site (for example properties along Bellozanne Valley and West Hill);
2. Visitors to the area (such as guests at Westhill Country Hotel); and
3. Road users particularly along Bellozanne Valley.

Given the potential nuisance to these third parties, best practice should be followed with appropriate mitigation measures implemented to control potential nuisance during both the construction and operation phases. Deliveries of materials for construction, storage of materials and removal of waste for disposal have the potential for mud on local roads, dust creation and litter.

Operation of the new STW will involve the ongoing transportation of sludge for land disposal or to the EfW plant at La Collette for disposal when land disposal is not available. The increased capacity at the STW is designed to allow for increases in future population growth. The predicted increase in the Jersey population will result in increased sewage and associated sludge production which will result in increased journeys for its disposal. An increase in number of vehicle movements may have an adverse impact on the noise levels to which local residents are subjected.

9.5.12 Cultural

To date, no detailed review of local records has been undertaken to understand local cultural and heritage sites and therefore full consideration of this impact cannot be considered at this time. However archaeological records have been used to consider the small area of the works that will take place on what is effectively a greenfield site. These excavations have some potential for interaction with archaeology such as hogues and monolithic remains, prehistoric landscape and prehistoric sites of interest although the risk is considered low.

9.5.13 Landscape and Visual

The STW location is such that the majority of the visual impact of any additional works will be masked due to valley topography, however high rise residencies to the south of the site may be subject to some negative visual impact. The general appearance of the site will not change as the new assets are visually broadly similar to the existing, however, the area of the site will be increasing.

10. Effluent Outfall

The effluent from the Bellozanne STW is currently discharged via a 42" diameter outfall into St Aubin's Bay near the First Tower area at a distance of 0.5km from the seawall. The outfall also receives flow downstream of the STW from the stream in the Bellozanne Valley and other surface water drainage in the area.

10.1 Existing Outfall

The existing outfall is more than 50 years old and the condition is currently under investigation. Preliminary findings are that the condition is variable and some sections will require replacement or rehabilitation. The size of the outfall is recorded as varying between 900 and 1200mm although the outfall pipe across the beach has been measured at 42" diameter. The maximum total flow through the outfall is unknown and is the subject of a flow measurement study at the time of writing. The flow contribution from the stream and drainage sources is unknown although the effluent from the STW is measured. . There are reports from the STW operators of the existing UV plant being flooded due to lack of capacity in the outfall although it is unclear if this has been due to excessive stream/drainage flow, high tides or a temporary blockage. The ongoing flow measurement study and subsequent network modelling is intended to determine the capacity of the existing outfall and its suitability for reuse.

It is anticipated that rehabilitation of the existing outfall will be required as a minimum and any hydraulic restrictions identified will be rectified.

10.2 Future Outfall

Two main drivers have been considered for the purposes of the future outfall as follows:

- Assimilative capacity of St Aubin's Bay
- Aesthetics in terms of exposed effluent outfall pipe

The investigations[23] carried out to date have considered a number of options for extending the outfall, up to a maximum of 1.5km from the seawall, based on the possibility of St Aubin's Bay being classed as sensitive water. It is now understood that an extension to the existing outfall will not be required based on the assimilative capacity of the receiving waters. This is subject to the evidence based policy being agreed with the Regulator as discussed earlier.

With regard to the aesthetic considerations, the latest estimate to replace the outfall with a buried pipe to a point beyond the low water mark is £4m. This is a significant capital expenditure and considered unnecessary in the current economic climate if it is only required on the basis of aesthetics.

Should the condition survey show that the exposed outfall requires repair then this will, of course, be carried out as part of the proposed scheme. The extent of any repairs identified may mean that a

[23] Bellozanne STW – Long Sea Outfall Feasibility Report, October 2012

like for like replacement across the beach is the most cost-effective solution and the option of carrying out some form of extension at the same time will be reconsidered at that time.

11. Project Programme

An outline programme for the replacement of Bellozanne STW is attached to this Section as Figure 11.1 (Appendix B) and is summarised below. The land available for the construction of the new STW, while keeping the existing STW in service, is limited. The programme is therefore driven by sequencing work to minimise disruption to operations rather than minimising the overall duration.

In order to increase the footprint of the site, the existing wooded slopes surrounding sections of the STW will need to be excavated back and stabilised in advance of any new construction works. This excavation will allow the new STW to be sited within the existing boundaries and avoid the need for any land purchase or re-alignment of the main Bellozanne Valley Road.

The Energy from Waste Plant on the site is currently being demolished under a separately funded project. All other buildings and structures will be demolished and removed to suit the phasing of the project and this work is therefore included in the project. The Clinical Waste Incinerator currently in operation at the site is programmed to be relocated to La Collette by the end of 2014 under this project.

The Public Waste Reception Facility and Scrapyard will also be relocated off site to accommodate the relocated Inlet Works and Administration Building. These relocation works are being funded separately and do not form part of this project. The old scrap yard at the end of Bellozanne valley has been closed and a temporary operation has been set up adjacent to the old EFW. The scrap operation will stay in this location until it moves to its new location at La Collette in 2104. The new Public Waste Reception Facility is planned to go on the old scrap yard site. It is proposed to complete the design and approvals of these schemes in 2013 and commence construction in 2014 when funding becomes available (subject to the 2013 budget debate).

Based on the historical background of the site with industrial use, there is potential for ground contamination and therefore clean-up / site remediation will be required prior to the change in the land use associated with any new construction.

The Accommodation Works to facilitate the start of construction of the new STW will be initiated in conjunction with the site remediation. This will include the hillside excavation/ stabilisation, demolition of redundant buildings and structures and diversion of services,

The proposed layout of the new STW has been developed to ensure that the existing facilities continue to remain in service throughout the construction period. To achieve this final plant locations unfortunately dictate that the vast majority of the existing services will have to be relocated or re-routed. These diversions require careful phasing to suit both the construction activities and ongoing operations. On receipt of approval for funding it will be necessary to relocate/ replace the standby generator sets and divert a number of underground services including the HV electrical supply cables (JEC & TTS), communication cables, sewers and surface water drainage pipework so that the Accommodation Works (demolition, hillside excavation, further service diversions and site remediation) can proceed.

Once all of the above-noted activities have been completed, works can begin on the construction of the new STW. As the existing works must remain in service at all times, the construction and commissioning of the new STW will be completed in two distinct phases in order to create the required space and to decommission the existing STW in stages without compromising the treatment capacity. However, it should be noted that some of the process units can be constructed under either Phase 1 or Phase 2 based on the funding availability although some of these elements will not be of use until Phase 2 is complete

Phase 1 of construction of the new STW, following the Accommodation Works, can be summarised as follows:

Phase 1

- Inlet Works, including screening and grit/ FOG removal
- Storm Tanks
- Primary Settlement Tanks
- Sludge Storage Tanks (replacement of existing)
- Administration Building

On successful commissioning of Phase 1, the existing Inlet Works, Sludge Storage Tanks and Primary Settlement Tanks and associated works will be demolished to make way for the remaining STW assets to be constructed under Phase 2 as follows:

Phase 2

- Activated Sludge Plant and associated works
- Final Settlement Tanks (if not constructed under Phase 1)
- UV Disinfection Plant (if not constructed under Phase 1)

The FSTs, if constructed in Phase 1, will only be commissioned with the ASP in Phase 2. However if funding allows, there is an opportunity to build some of the tanks in Phase 1 and shorten the overall Phase 2 programme. If Phase 1 and Phase 2 are both let to the same contractor, the construction of the FSTs could bridge between the two phases without impacting the ongoing operation of the site.

If the UV plant is constructed in Phase 1 it may be possible to provide a temporary connection to the existing outfall such that it will be of beneficial use. This would certainly be preferable to the plant sitting unused while Phase 2 is built and could run in conjunction with the existing UV plant for additional security during commissioning and turning of flows.

The condition survey of the existing outfall is currently underway together with the data collection and modelling of the catchment to determine the capacity and suitability for continued use of the existing outfall. It is anticipated that rehabilitation of the existing outfall will be required for its continued use and any hydraulic restrictions identified will be rectified.

The detailed design and construction of the proposed works are expected in 2014/ 2015.

12. Cost Estimates

12.1 Proposed Scheme

The estimated total project cost, including the relocation of the Clinical Waste Incinerator, is £75m based on 2012 prices. These estimates are based on 2012 costs and are derived from the feasibility design work. A confidence level of plus or minus 15% can be expected at this stage.

A total allowance of £18.3m is included in this estimate for Transport & Technical Services Costs, Professional Fees and Contingencies. Varying levels of contingencies on work elements have been included at this stage to reflect the current level of understanding and the areas considered to have the most risk.

The estimate summary with a further breakdown of these costs is presented in Table 12.1.

Cost Estimate Summary	Direct Cost Estimate	Contingency		Total
		%	£	
Feasibility, EIA, Tender Documents & Design Development	£1,700,000	15.0%	£255,000	**£1,955,000**
Site Remediation	£1,000,000	25.0%	£250,000	**£1,250,000**
Initial Service Diversions and Temporary Works	£3,000,000	25.0%	£750,000	**£3,750,000**
Accommodation Works (demolition, hillside excavation & stabilisation and service diversions)	£9,800,000	25.0%	£2,450,000	**£12,250,000**
STW Phase 1 Construction (Inlet Works, Storm Tanks, PSTs, Part FSTs, Sludge Storage replacement & Admin. Building)	£18,450,000	20.0%	£3,690,000	**£22,140,000**
STW Phase 1 Commissioning	£200,000	20.0%	£40,000	**£240,000**
STW Phase 2 Construction (ASP, Part FSTs & UV)	£12,500,000	20.0%	£2,500,000	**£15,000,000**
STW Phase 2 Commissioning	£300,000	20.0%	£60,000	**£360,000**
Effluent Outfall (Rehabilitation/ Replacement)	£2,750,000	30.0%	£825,000	**£3,575,000**
CWI Replacement	£7,000,000	0.0%	£0	**£7,000,000**
TTS Costs	£2,055,000	0.0%	£25,000	**£2,080,000**
Professional Fees	£4,800,000	12.5%	£600,000	**£5,400,000**
Total	**£63,555,000**	**£11,445,000**		**£75,000,000**

Table 12.1 - Cost Estimate Summary

It should be noted that the proposed design of the new STW takes into account that an extension to the existing outfall will not be required on the basis of the assimilative capacity of the receiving waters resulting from the evidence based policy to be agreed with the Regulator.

A cashflow profile is provided in Table 12.2 together with a graphical form in Figure 12.1.

	Quarterly Amount	Cumulative
1st Quarter 2013	£222,000	£222,000
2nd Quarter 2013	£228,000	£450,000
3rd Quarter 2013	£1,232,500	£1,682,500
4th Quarter 2013	£1,848,750	£3,531,250
1st Quarter 2014	£3,724,052	£7,255,302
2nd Quarter 2014	£3,930,857	£11,186,159
3rd Quarter 2014	£4,681,968	£15,868,127
4th Quarter 2014	£4,835,815	£20,703,942
1st Quarter 2015	£4,233,432	£24,937,374
2nd Quarter 2015	£5,207,615	£30,144,989
3rd Quarter 2015	£4,928,203	£35,073,192
4th Quarter 2015	£4,851,280	£39,924,473
1st Quarter 2016	£4,697,434	£44,621,907
2nd Quarter 2016	£4,202,434	£48,824,341
3rd Quarter 2016	£4,202,434	£53,026,775
4th Quarter 2016	£2,131,846	£55,158,621
1st Quarter 2017	£1,879,885	£57,038,506
2nd Quarter 2017	£3,446,552	£60,485,057
3rd Quarter 2017	£3,446,552	£63,931,609
4th Quarter 2017	£3,446,552	£67,378,161
1st Quarter 2018	£3,446,552	£70,824,713
2nd Quarter 2018	£2,673,218	£73,497,931
3rd Quarter 2018	£1,126,552	£74,624,483
4th Quarter 2018	£375,517	**£75,000,000**

Table 12.2 – Cashflow Profile

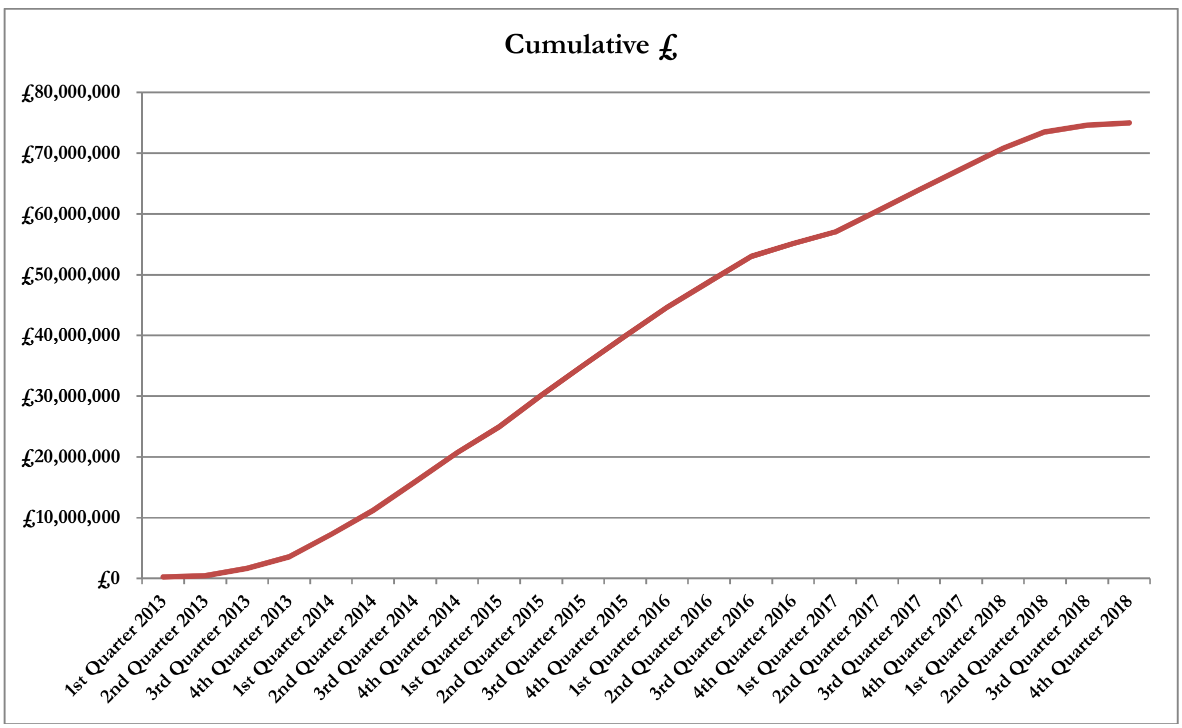

Figure 12.1 – Cashflow Chart

12.2 Future Upgrades

Based on the feasibility design presented, the new Bellozanne STW is designed and constructed as a conventional activated sludge plant based on carbonaceous removal to achieve a BOD/ SS standard. The design includes the capacity for extending the works in the future with a secondary treatment upgrade if consent conditions change. Stage 2 would provide nitrification using additional tanks and aeration capacity for an additional £16.5m, based on 2012 prices. Should a total nitrogen standard be applied then further tanks would be required together with modification to the existing units under Stage 3 for an additional £14.3m, based on 2012 prices.

Stage 2 may be constructed in isolation but Stage 3 must be constructed in conjunction with or following completion of Stage 2. The estimated total project cost for these future secondary treatment upgrades, including engineering and contingency, is therefore £30.8m based on the 2012 prices. If both stages were constructed under a single contract there may be some economies of scale to be applied but these have not been assessed at this stage. A summary of these future cost estimates are presented in Table 12.3.

Cost Estimate Summary Future Stages 2 & 3	**Direct Cost Estimate**	**Engineering & Contingency**		**Total**
		%	£	
New STW Stage 2 Demolition and Construction	£13,200,000	25.0%	£3,300,000	£16,500,000
New STW Stage 3 Demolition and Construction	£11,440,000	25.0%	£2,860,000	£14,300,000
Total for Future Stages	**£24,640,000**	**£6,160,000**		**£30,800,000**

Table 12.3 – Future Cost Estimates

12.3 Net Present Value

A net present value for the New STW, excluding the Centralised Sludge Treatment Facilities, has been calculated using a discount rate of 6% over a period of 20 years. A summary of this is given in Table 12.4, together with the predicted operational costs and other assumptions.

Including the initial capital expenditure of £68m for the new STW, (£75m less £7m for the cost of relocating the existing Clinical Waste Incinerator), the NPV for the new STW is approximately £98.4m over the assessment period.

Initial Capital cost of Works	£68,000,000
Spares	1,246,992
Plant - 5 Year Life	1,790,556
Plant - 10 Year Life	2,088,479
Plant - 15 Year Life	104,316
Plant - 20 Year Life	155,902
Electricity consumed	6,619,732
Manpower & Admin	7,756,291
Supplies & Services	6,234,961
Premises & Maintenance	4,364,472
Total	**£98,361,702**

Table 12.4 – Summary of Net Present Values

The NPVs have been calculated with some information from the existing STW based on the following assumptions:

Basis of Power Consumption		
Power Consumed (Average)	4,423,800	kWhr/annum
Power Produced (Average)	0	mWhr/annum
Cost of Power - In	0.12	kWhr
Cost of Power - Out	0.12	kWhr

Basis of Replacement Plant		
Software	£30,000	5 Years
Instruments	£100,000	5 Years
Mixers	£400,000	10 Years
Diffusers	£500,000	5 Years
Blowers	£500,000	10 Years
PC Pumps	£250,000	5 Years
Screens Inlet	£500,000	10 Years
Control Panels	£500,000	20 Years
UV plant	£500,000	10 Years
Odour Control	£500,000	10 Years
Glass Coated Tanks	£250,000	15 Years

Basis of Other Costs		
Manpower	£600,000	As Existing
Administration Costs	£22,000	As Existing
Supplies and Services	£500,000	As Existing
Premises & Maintenance	£350,000	Excl. Power & M&E Maintenance
Annual Spares	£100,000	1% of £10m M&E Equipment

Table 12.5 – Base data for Net Present Values

Revenue operating costs can be accommodated from the TTS Department's existing revenue budget. The replacement of the STW will allow more automation of the STW which will allow further de-manning and future potential revenue savings.

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13. Risk Assessment

Potential risks or residual risks have been reviewed and outlined in this Section.

Effluent Quality - General

It has been assumed that an extension to the existing outfall will not be required on the basis of the assimilative capacity of the receiving waters resulting from an evidence based policy to be agreed with the Regulator. Further data collection and water quality modelling are currently underway to demonstrate that the location of the outfall is suitable for the long term needs. However, there is always the possibility that the Regulator will insist on a more stringent standard than the carbonaceous removal plant initially proposed will accommodate.

Effluent Quality - UV Treatment

The consent standard for UV disinfection is currently based on the applied UV dose. Should this change to an actual faecal coliform standard, but depending upon the actual standard applied, it is likely that the suspended solids concentration and particle distribution, even for a fully compliant effluent, would be too high to guarantee the FC kill required. Tertiary treatment via sand filters prior to the UV plant would then be required to ensure effluent compliance.

Saline Infiltration into Sewerage Network

Saline sewage, due to the high sulphate content, will lead to high H_2S production and potential problems with confined spaces and corrosion leading to possible equipment failure. Odour control systems will need to be designed accordingly. In addition severe saline intrusion contributes to the flow coming into the works but not the load resulting in low BOD concentrations. At high tides and during periods of high rainfall the saline intrusion can mean that the capacity of the sewerage system is exceeded thus leading to overflow at CSOs and further reduction of the load entering the works. These factors mean that the normal relationship between flow and load seen in the STW is no longer true.

Low biological concentrations (loads) at the STW result in low MLSS concentrations within the activated sludge plant that are more susceptible to changes in influent conditions. This means that the plant is far less robust and more difficult to operate effectively. As well as being affected by the low concentration and load into the works, high changes in salinity in the influent will affect the biology of the system and the settlement of the solids. This can obviously affect the effluent quality and cause consent failure. In addition the process commissioning period required for an activated sludge plant with high saline intrusion will be considerably longer than for a system without saline intrusion.

Proposed Secondary Treatment Technology

Technology identified could possibly be seen as a risk by the Regulator. The existing plant utilises an advanced technology that replaced the old activated sludge plant but has made the process unstable and unable to produce a consistent quality effluent. It is feasible therefore that the question as to why the process is moving back to a conventional system may be raised. However the conventional activated sludge process selected is utilised throughout the world and gives a level of flexibility and robustness not achieved by the current process selected. It is also familiar to the operators, easier to control and has a lower operating cost due to the lower aeration requirement. The numerous advantages gained therefore mean that this is considered a low risk.

Location of outfall

The current proposal does not include monies for the relocation or extension of the existing outfall. Political pressure or an aesthetic driver may require the existing outfall to be relocated or extended as it is currently visible at low tide. Discussions have been held with the Regulator and the existing location is currently considered as acceptable.

Land Availability

Any major increases in the sizes of the proposed process units will affect the land take and may require additional hillside excavation which in turn may require additional land to be purchased. It may also require the re alignment of the Bellozanne Valley Road.

The slope stabilisation has been assumed to be able to achieve a rock face of approximately 10^{o} from the vertical. If this proves to be impractical and a shallower gradient is required then additional hillside excavation will be necessary.

Programme – Relocation of the Clinical Waste Incinerator and other Services

Delays in the relocation of the Clinical Waste Incinerator, Scrapyard, Public Waste Reception Facility or the ongoing demolition of the old Energy from Waste plant will directly affect the Accommodation Works for the new STW and hence delay the predicted project start and completion dates. Careful monitoring of programme and progress of these projects will be required to ensure the new STW project stays on programme overall.

Programme – Relocation of Existing Services

There are critical services on site that are required to be relocated prior to any demolition and/ or excavation activities and could therefore affect the programme. Discovery of unknown buried services will also extend the period required for the diversion of the existing services and hence will directly affect the predicted project completion date. Early location of services through the use of trial holes will reduce the risk of the discovery of unknown services which will require investigation as to their purpose and status before further action can be carried out and the service diverted or removed.

Office Relocation

The site IT servers are located in the existing Weigh Bridge Building and will require relocation prior to any demolition activities. When these are relocated additional labour may be required to monitor and operate the works.

Lack of Full Funding

Should full funding for the new STW not be available then an alternative approach would be required to ensure that the works at Bellozanne can reliably treat the incoming sewage to consistent standards. This would involve modifications to the existing works whilst maintaining operation of the plant such that the incoming sewage is treated prior to discharge. Careful planning and re-commissioning strategies would be required. The construction would consist of a number of stages and the commissioning, testing and proving periods between stages would add considerably to the overall project duration.

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14. Conclusion & Recommendations

Transport & Technical Services Department (TTS) are currently failing the discharge consent for the Bellozanne STW due to design capacity limitations. The existing STW is also nearing the end of its original design life and now requires replacement despite ongoing capital maintenance.

This report presents a plan for the replacement of the Bellozanne Sewage Treatment Works (STW) and rehabilitation/ partial replacement of the effluent outfall into St Aubin's Bay to meet the needs of the Island for the next 50 years.

The land available for the construction of the new STW, while the existing STW continues to remain in service, is limited. It is therefore imperative that various construction activities are programmed to minimise any disruption to the activities on the Waste Management Services site.

In order to increase the footprint of the site, the existing wooded slopes surrounding various sections of the site will need to be excavated back and stabilised as required in advance of any new construction works. However, the new STW will be sited within the existing site boundaries without the need for any land purchase or re-alignment of the main Bellozanne Valley Road.

The Energy from Waste Plant is currently being demolished and is funded separately. All other buildings and structures will be demolished and removed to suit the phasing of the project and therefore included in the project.

The Clinical Waste Incinerator currently in operation at the site is programmed to be relocated under this project.

The Public Waste Reception Facility and Scrapyard will also be relocated off site as part of the Bellozanne Waste Management Services site development to accommodate part of the new STW. These works are being funded separately and do not form part of this project. The old scrap yard at the end of Bellozanne Valley has been closed and a temporary operation has been set up adjacent to the old EFW. The scrap operation will stay in this location until it moves to its new location at La Collette in 2014. The new Public Waste Reception Facility is planned to go on the old scrap yard site.

Based on the historical industrial use of the site, there is potential for ground contamination and therefore clean-up / site remediation will be required prior to the change in the land use associated with any new construction.

The Accommodation Works to facilitate the construction of the new STW will be initiated to include the hillside excavation/ stabilisation, demolition of building structures and diversion of services in conjunction with the site remediation.

The proposed layout of the new STW means that the vast majority of the existing services will require relocation or diversion while the existing facilities remain in service. These works will be phased to suit the construction activities.

Once all of the above-noted advance works have been completed, construction of the new STW can begin. As the existing works continue to remain in service at all times, the construction and commissioning of the new STW will be in two phases in order to create the required space and to decommission the existing STW without compromising the ongoing treatment capacity. However, it should be noted that some of the process units can be constructed under either Phase 1 or Phase 2 based on the funding availability.

Based on the feasibility design presented it is recommended that the new Bellozanne STW is designed and constructed as a conventional activated sludge plant based on carbonaceous removal to achieve a BOD/ SS standard. The discharge standard may be altered in the future such that an additional ammonia standard or even a total nitrogen standard is required. Therefore it is recommended that the Bellozanne STW be designed with the potential future secondary treatment upgrade requirements in mind.

In view of the uncertainty associated with the wide range of population growth scenarios under consideration, it is recommended to make provisions in the design for the future connected population to be up to 20% higher than the proposed 2035 design population of 118,000 such that only minor modification to the works will be required. While the design horizon is considered to be reasonable it is based on a net population growth of +500 per annum and the Statistics Unit are currently considering scenarios with growth between nil and +1000. It is recommended that the design is completed on conservative parameters to achieve this flexibility rather than allowing for a higher population per se.

It is proposed to continue with the condition survey and data collection of the outfall catchment. This will be used to model the catchment and existing outfall to determine the existing capacity and its suitability for continued use as the effluent outfall. It is recommended that any required rehabilitation works are carried out and any hydraulic restrictions identified are rectified to make the outfall viable for the long term needs. It is understood that an extension of the existing outfall will not be required on the basis of the assimilative capacity of the receiving waters and it is not recommended to replace or extend the outfall on purely aesthetic grounds. This is subject to agreement with the Regulator.

The proposed works are programmed to start in July 2013 for completion by October 2018. The estimated total project cost, including the relocation of the Clinical Waste Incinerator, is £75m. This estimate is based on 2012 costs and the feasibility design work. A confidence level of plus or minus 15% can be expected at this stage.

APPENDIX A – LIST OF SUPPLEMENTARY REPORTS

1. Liquid Waste Strategy (January 2009)
2. Bellozanne Master Plan for the STW (July 2009)
3. Liquid Waste Strategy (May 2010)
4. Bellozanne STW Operation Strategy (May 2011)
5. Outfall Assessment (Desk) Study (April 2010)
6. Bellozanne STW Best Available Technology Report (May 2012)
7. Bellozanne STW – Long Sea Outfall Feasibility Report, October 2012

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APPENDIX B – FIGURES

LIST OF FIGURES

SK13-01	Figure 2.1.1 - Existing STW Layout 1 of 2
SK13-02	Figure 2.1.2 - Existing STW Layout 2 of 2
SK12-00	Figure 2.2 - Existing Effluent Outfall
1001-00	Figure 7.1 – Process Flow Diagram
1011-01	Figure 7.2.1.1 - P&ID Inlet Works 1 of 2
1011-02	Figure 7.2.1.2 - P&ID Inlet Works 2 of 2
1012-00	Figure 7.2.2 - P&ID Grit/Screenings Removal
1013-00	Figure 7.2.3 - P&ID Primary Settlement
1014-00	Figure 7.2.4 - P&ID Primary Settlement Desludge Chambers
1015-00	Figure 7.2.5 - P&ID Activated Sludge Plant
1016-00	Figure 7.2.6 - P&ID Final Settlement Tank Nos 1,2,5&6
1017-00	Figure 7.2.7 - P&ID Final Settlement Tank Nos 3,4,7&8
1018-00	Figure 7.2.8 - P&ID RAS/SAS Pump Station 'A'
1019-00	Figure 7.2.9 - P&ID RAS/SAS Pump Station 'B'
1020-00	Figure 7.2.10 - P&ID UV Disinfection
1021-00	Figure 7.2.11 - P&ID Final Effluent Sampling & Outfall
1022-00	Figure 7.2.12 - P&ID Storm Tanks & Return Pump Station
1023-00	Figure 7.2.13 - P&ID Scum Lift Pump Stations
SK14-00	Figure 7.3 - Stage 1 Treatment Process (BOD Removal)
SK15-00	Figure 8.1 - Stage 2 Treatment Process (BOD Removal & Nitrification)
SK16-00	Figure 8.2 - Stage 3 Treatment Process (BOD, Nitrification and TN Standard
SK10-01	Figure 9.1 - New STW Layout 1 of 2
SK10-02	Figure 9.2 - New STW Layout 2 of 2
SK10-03	Figure 9.3.1 - Site Cross Sections
1005-00	Figure 9.3.2 - Hydraulic Profile (Treatment Stages 1 to 3)
	Figure 9.4 - Hillside Stabilisation 1 of 2
	Figure 9.5 - Hillside Stabilisation 2 of 2
SK11-01	Figure 9.7.1 - Existing Site Services Layout 1 of 2
SK11-02	Figure 9.7.2 - Existing Site Services Layout 2 of 2
	Figure 11.1 – New STW Programme