N O T I C E   O F   M E E T I N G

W A T E R ,   W A S T E   A N D   S E W E R   A D V I S O R Y   C O M M I T T E E   M E E T I N G

A Water, Waste and Sewer Advisory Committee Meeting of Byron Shire Council will be held as follows:

Venue	**Council Chamber, Station Street, Mullumbimby**
Date	**Thursday, 1 March 2018**
Time	**11.30am**

Phil Holloway
Director Infrastructure Services

I2018/257
Distributed 22/02/18

CONFLICT OF INTERESTS

What is a "Conflict of Interests" - A conflict of interests can be of two types:

Pecuniary - an interest that a person has in a matter because of a reasonable likelihood or expectation of appreciable financial gain or loss to the person or another person with whom the person is associated.

Non-pecuniary – a private or personal interest that a Council official has that does not amount to a pecuniary interest as defined in the Local Government Act (eg. A friendship, membership of an association, society or trade union or involvement or interest in an activity and may include an interest of a financial nature).

Remoteness – a person does not have a pecuniary interest in a matter if the interest is so remote or insignificant that it could not reasonably be regarded as likely to influence any decision the person might make in relation to a matter or if the interest is of a kind specified in Section 448 of the Local Government Act.

Who has a Pecuniary Interest? - a person has a pecuniary interest in a matter if the pecuniary interest is the interest of the person, or another person with whom the person is associated (see below).

Relatives, Partners - a person is taken to have a pecuniary interest in a matter if:

- The person's spouse or de facto partner or a relative of the person has a pecuniary interest in the matter, or
- The person, or a nominee, partners or employer of the person, is a member of a company or other body that has a pecuniary interest in the matter.

N.B. "Relative", in relation to a person means any of the following:

(a) the parent, grandparent, brother, sister, uncle, aunt, nephew, niece, lineal descends or adopted child of the person or of the person's spouse;

(b) the spouse or de facto partners of the person or of a person referred to in paragraph (a)

No Interest in the Matter - however, a person is not taken to have a pecuniary interest in a matter:

- If the person is unaware of the relevant pecuniary interest of the spouse, de facto partner, relative or company or other body, or
- Just because the person is a member of, or is employed by, the Council.
- Just because the person is a member of, or a delegate of the Council to, a company or other body that has a pecuniary interest in the matter provided that the person has no beneficial interest in any shares of the company or body.

Disclosure and participation in meetings

- A Councillor or a member of a Council Committee who has a pecuniary interest in any matter with which the Council is concerned and who is present at a meeting of the Council or Committee at which the matter is being considered must disclose the nature of the interest to the meeting as soon as practicable.
- The Councillor or member must not be present at, or in sight of, the meeting of the Council or Committee:
  (a) at any time during which the matter is being considered or discussed by the Council or Committee, or
  (b) at any time during which the Council or Committee is voting on any question in relation to the matter.

No Knowledge - a person does not breach this Clause if the person did not know and could not reasonably be expected to have known that the matter under consideration at the meeting was a matter in which he or she had a pecuniary interest.

Participation in Meetings Despite Pecuniary Interest (S 452 Act)

A Councillor is not prevented from taking part in the consideration or discussion of, or from voting on, any of the matters/questions detailed in Section 452 of the Local Government Act.

Non-pecuniary Interests - Must be disclosed in meetings.

There are a broad range of options available for managing conflicts & the option chosen will depend on an assessment of the circumstances of the matter, the nature of the interest and the significance of the issue being dealt with. Non-pecuniary conflicts of interests must be dealt with in at least one of the following ways:

- It may be appropriate that no action be taken where the potential for conflict is minimal. However, Councillors should consider providing an explanation of why they consider a conflict does not exist.
- Limit involvement if practical (eg. Participate in discussion but not in decision making or vice-versa). Care needs to be taken when exercising this option.
- Remove the source of the conflict (eg. Relinquishing or divesting the personal interest that creates the conflict)
- Have no involvement by absenting yourself from and not taking part in any debate or voting on the issue as if the provisions in S451 of the Local Government Act apply (particularly if you have a significant non-pecuniary interest)

RECORDING OF VOTING ON PLANNING MATTERS

Clause 375A of the Local Government Act 1993 – Recording of voting on planning matters

(1) In this section, **planning decision** means a decision made in the exercise of a function of a council under the Environmental Planning and Assessment Act 1979:
  (a) including a decision relating to a development application, an environmental planning instrument, a development control plan or a development contribution plan under that Act, but
  (b) not including the making of an order under Division 2A of Part 6 of that Act.

(2) The general manager is required to keep a register containing, for each planning decision made at a meeting of the council or a council committee, the names of the councillors who supported the decision and the names of any councillors who opposed (or are taken to have opposed) the decision.

(3) For the purpose of maintaining the register, a division is required to be called whenever a motion for a planning decision is put at a meeting of the council or a council committee.

(4) Each decision recorded in the register is to be described in the register or identified in a manner that enables the description to be obtained from another publicly available document, and is to include the information required by the regulations.

(5) This section extends to a meeting that is closed to the public.

BUSINESS OF MEETING

1. APOLOGIES

2. DECLARATIONS OF INTEREST – PECUNIARY AND NON-PECUNIARY

3. ADOPTION OF MINUTES FROM PREVIOUS MEETINGS

3.1 Water, Waste and Sewer Advisory Committee Meeting held on 10 October 2017
3.2 Extraordinary Water, Waste and Sewer Advisory Committee Meeting held on 21 December 2017

4. STAFF REPORTS

Infrastructure Services

STAFF REPORTS - INFRASTRUCTURE SERVICES

Report No. 4.1	**Ocean Shores Sewage Transfer Risk Assessment**
Directorate:	Infrastructure Services
Report Author:	Peter Rees, Manager Utilities
File No:	I2017/1821
Theme:	Community Infrastructure Sewerage Services

Summary:

At Council's Ordinary Meeting of 22 June 2017 it was resolved to investigate operational risks cited in the GHD Study and report back to Council via the Committee on the Ocean Shores Transfer to Brunswick Valley STP Option 4.

The risk assessment for Option 4 was finalised by GHD in December 2017 and identified that the most significant risks are mitigated by the construction of a 30ML wet weather storage.

The storage concept is not new and also aligns with the Mullumbimby Inflow and Infiltration resolution 18-054; Council Resolution 10-840 in 2010 and original strategies conceptualised by Council in the early 2000's.

The Option 4 transfer of sewage from Ocean Shores to Brunswick Valley STP also represents a significant financial saving to Council in the longer term.

RECOMMENDATION:

1. **That the Risk Assessment Report be noted.**

2. **That Council proceed to detailed design phase for Option 4 of an upgraded plant including the wet weather storage at Brunswick Valley STP.**

Attachments:

1 24.2009.36.1 GHD Report - Ocean Shores transfer to Brunswick Valley STP Process Risk Assessment - December 2017, E2018/13028 , page 7⇩

Report

Background - At Council's Ordinary Meeting of 22 June 2017:

17-250 ***Resolved*** *that Council adopt the following Committee Recommendation:*

Report No. 4.4 Ocean Shores to Brunswick Valley STP Transfer Feasibility Study
File No: I2017/678

Committee Recommendation 4.4.1

1. *That Council notes the report about the Ocean Shores to Brunswick Valley STP Transfer Feasibility Study, including risks associated with Option 4 in Section 13 recommendations.*

2. *That Council investigate operational risks cited in the Study and report back to Council via the Committee on the Transfer option, taking into account Council resolution 17-177.*

(Richardson/Hunter)

17-177 Resolved *that Council adopt the following Committee Recommendations:*

Report No. 4.2 Inflow and Rainfall - Brunswick Valley STP, March 2017
File No: I2017/366

Committee Recommendation 4.2.1

1. *That Council note that the Water, Waste and Sewer Advisory Committee was provided with daily inflow and rainfall figures for March 2017 for the Brunswick Valley STP.*

2. *That the Committee be provided with a report on the need to replace the original sewer network in 'old' Mullumbimby (as it was in the 1960s, when the sewer network was built) and to consider including allocations in its future business plans for sewer management.*

3. *That the report, in part 2 of the recommendation above, consider options and how well they protect or enhance the environment.*

(Hunter/Cameron)

A recent Feasibility Study (GHD, 2016) proposed the transfer of raw wastewater flows and loads from the existing Ocean Shores catchments to the Brunswick Valley Sewage Treatment Plant (BVSTP), followed by the decommissioning of the older existing Ocean Shores STP.

The Feasibility Study report (GHD, 2016) identified that the transfer of wastewater to BVSTP poses some risks relating to plant process and/or hydraulic capacity.

The risk assessment for Option 4 was finalised by GHD in December 2017, see Attachment 1. The most significant risks are mitigated by the construction of 30ML wet weather storage.

The above also leads into the subsequent Council Resolution 10-840 resolved in part:

That Council endorses the recommendations on page 5 of the Mullumbimby Sewerage System Inflow and Infiltration Integrated Strategy Final Report June 2010 (#1002349) as per below:

(1) complete the work on the gravity system in catchments 4001 and 4003b
(2) continue with inspection of private assets
(3) Implement routine investigation and repairs for both public and private infrastructure in
Council's operational and maintenance activities; and
(4) investigate further the potential implementation of constructed wetlands and a bulk effluent storage dam at Council's Brunswick Valley STP site to capture and also improve the integrated effluent management outcomes.

Due to the then financial status of the sewer fund after the completion of an $80 Million capital programme during that period, work on investigation and implementation of constructed wetlands and bulk effluent storage dam were put on hold.

A design for wet weather storage was already carried out as part of the 2003 concept to build the Brunswick Valley STP. The storage was sized at 60ML (called the effluent storage dam).

The most logical process now is to move to detailed design for an upgraded plant including the wet weather storage at Brunswick Valley STP; and commence formal negotiations with the EPA and DPI regarding environmental licence conditions and required work approvals necessary.

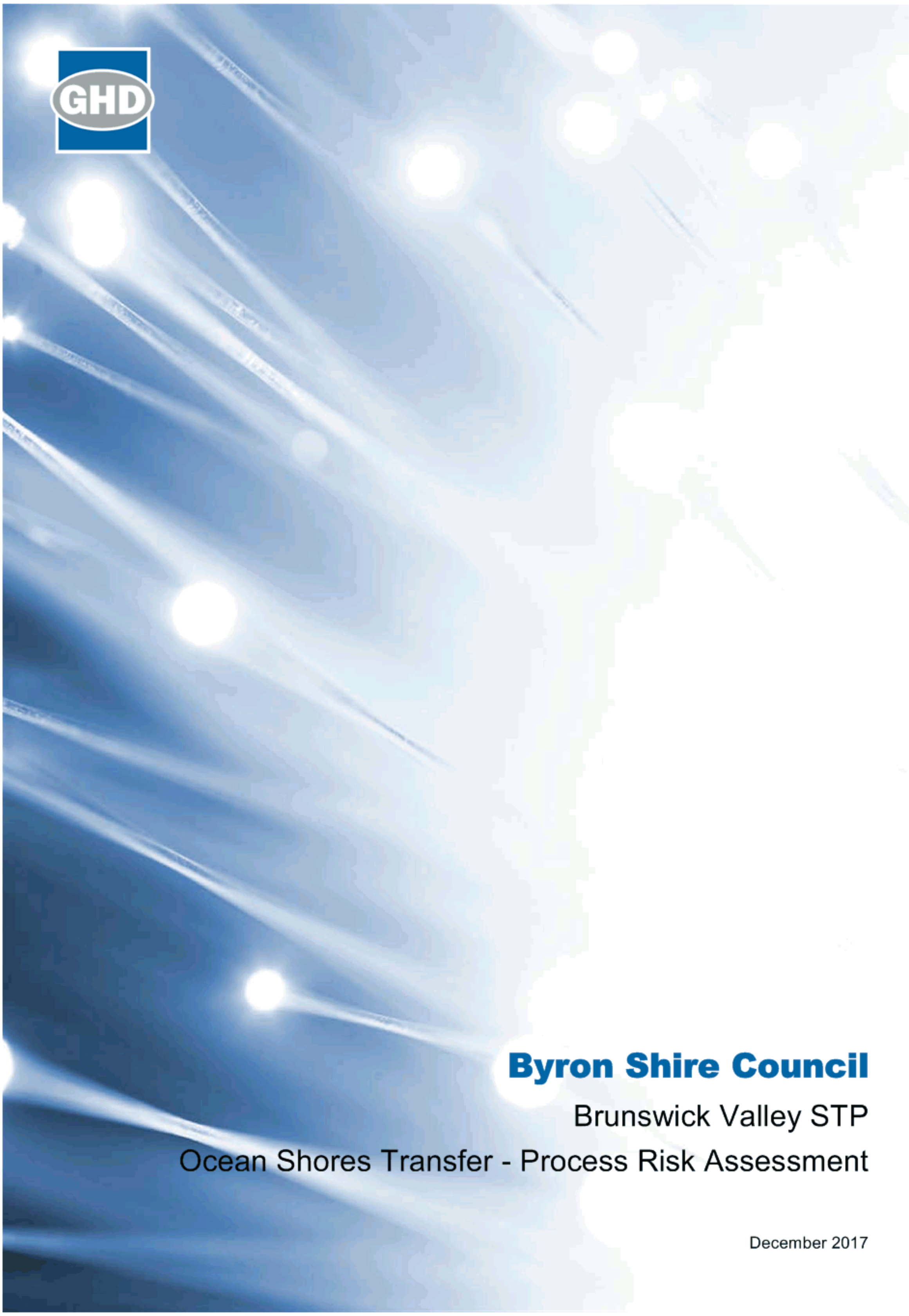
Financial Implications

The capital cost of Option 4 (in 2015 dollars), is estimated to be $10.6 Million. This is a significant saving to the community in both initial capital and whole of life costs.

An upgrade of Ocean Shores STP alone has a capital cost of $29 Million.

Statutory and Policy Compliance Implications

Council is continuing to liaise with NSW Environment Protection Authority (EPA) regarding compliance with licence conditions at both Ocean Shores and Brunswick Valley STPs. Consultation with the Department of Primary Industries will also be required in relation to Section 60 of the Local Government Act.



Byron Shire Council

Brunswick Valley STP

Ocean Shores Transfer - Process Risk Assessment

December 2017

WATER | ENERGY & RESOURCES | ENVIRONMENT | PROPERTY & BUILDINGS | TRANSPORTATION

Table of contents

Appendices

Disclaimer

GHD has prepared this report for Byron Shire Council and it *may only be used and relied on by Byron Shire Council for the purpose agreed between GHD and the Byron Shire Council, as set out in Sections 1.2 and 1.3 of this report.*

GHD otherwise disclaims responsibility to any person other than Byron Shire Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer sections 1.3 and 1.4). GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Byron Shire Council and others who provided information to GHD (including Government authorities)], which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report caused by errors or omissions in that information.

1. Introduction

1.1 Background

A recent Feasibility Study (GHD, 2016) proposed the transfer of raw wastewater flows and loads from the existing Ocean Shores catchments to the Brunswick Valley Sewage Treatment Plant (BVSTP), followed by the decommissioning of the older existing Ocean Shores STP.

The Feasibility Study report (GHD, 2016) identified that the transfer of wastewater to BVSTP poses some risks relating to plant process and/or hydraulic capacity. That report recommended that these risks be further assessed and appropriate risk mitigation strategies managed by Byron Shire Council in order for the wastewater transfer to be feasible. One of the risks is the relatively high wet weather flow associated with the combined catchments of Mullumbimby (M), Brunswick Heads (BH) and Ocean Shores (OS). Other risks include potential process constraints related to: clarifier capacity; oxidation ditch (bioreactor) aeration capacity and solids inventory; aerobic digester and associated biosolids holding/ dewatering capacity; and chemical dosing requirements, notably alum for supplementary phosphorus (P) removal.

1.2 Purpose of this report

The purpose of this report is to summarise the outcomes of an assessment of process risks at Brunswick Valley STP associated with the proposed transfer of wastewater from the Ocean Shores STP (and closure of the latter). The additional flows and loads due to this transfer will take the projected future combined loads, together with those from the existing catchments of Mullumbimby and Brunswick Heads, to near or marginally over the nominal design capacity of BVSTP, potentially within approximately ten to fifteen years of the transfer (assuming transfer occurs by ca. 2020). In this study, the risks of operating the BVSTP treatment plant during this period (i.e. near or marginally over its nominal design capacity) were assessed in terms of hydraulic and process constraints, together with possible mitigation measures. The constraints were simulated using the methodology documented, together with results, in a series of technical memoranda that are presented as appendices to this report. The main body of the report summaries the main outcomes of the study.

1.3 Scope and limitations

This study presents the results of a risk assessment based partly in the results of associated hydraulic and/or process simulations of the Brunswick Valley STP. The risk assessment is intended to form part of a Safety in Design framework for the future planning of transfer of wastewater from Ocean Shores catchments to the BVSTP, and possible future upgrades of that sewage treatment plant. The risk assessment and simulations undertaken here are at concept level and will need to be checked and repeated, if necessary, in order to form the basis of any future design of proposed works.

This study is based on the following limitations and information:

- Population and/or flow and load projections, as well as related information, as defined in the Ocean Shores to Brunswick Valley STP Transfer Feasibility Study (GHD, 2016) report dated November 2016 (Rev 0)
- BVSTP design information obtained from the plant designers' report (Fulton Hogan/ Cardno (2010), dated August 2010 (Version 9)
- Limited BVSTP current wastewater and operating data, as described in Section 1.4
- Concept-level simulations of flow (hydraulics) and process, as described in this report using commercial software (MS-Excel™, Palisade @RISK™ and BioWin™).

1.4 Assumptions

This study is subject to the following assumptions:

- Information obtained from previous reports by GHD and others (refer to Section 1.3 above)
- Limited plant operating data as follows:
  - Flow data including wet weather events in March-April 2017 and June 2017 for Brunswick Valley STP inlet, and pump stations SPS 5009 and 5004 (both in Ocean Shores), as supplied by Byron Shire Council
  - Activated sludge settleability data for BVSTP in October 2017
  - One complete set of 24-hour time series samples of raw wastewater at BVSTP on 31 October 2017.

This study is based on the underlying assumption that the capacity upgrade of BVSTP, to meet minimum requirements as defined in the Feasibility Study (GHD, 2016 – refer to Section 1.3), precedes the transfer of wastewater from Ocean Shores. The risks associated with transferring wastewater from Ocean Shores *without upgrading BVSTP* (i.e. due to insufficient hydraulic and process capacity) are assumed to be known and outside the scope of this study.

2. Hydraulic constraints

Refer to Appendix A for a more detailed technical description of modelling, results and recommendations in relation to the risks relating to hydraulic (flow) constraints.

2.1 Key constraints

2.1.1 Peak pumping capacity

The existing BVSTP has a design hydraulic limit of 314 L/s. This matches the peak pumping capacity for the two sewage pump stations (SPS 4000 and 2000) delivering flow from the existing catchments of Mullumbimby and Brunswick Heads connected to the plant.

With the proposed transfer of wastewater from Ocean Shores (involving flows pumped from SPS 5009 and 5004 to be connected to BVSTP via a new rising main pipeline), the combined peak pumping capacity to the plant is expected to exceed 500 L/s (a nominal value 546 L/s was adopted for this study).

2.1.2 Wet weather storage and proposed plant upgrade strategy

The option with the lowest whole-of-life cost, as recommended in the Feasibility Study (GHD, 2016) for the transfer of flows from Ocean Shores, proposed a limited initial upgrade of BVSTP to accommodate the increased peak pumping capacity. The proposed initial upgrade made provision for a new flow division structure (equipped with appropriate screening) to divert flows in excess of the plant capacity to a new storage facility, named a 'storm dam' with a provisional working volume of 20 ML. The storm dam would be equipped with a pump station to return stored wastewater for treatment at times of lower flow when treatment capacity permitted. A constructed wetland (which would normally receive treated effluent from the treatment process) was also proposed to be located downstream of the storm dam. The intent would be that any surplus flows that needed to be 'surcharged' from the storm dam under extreme conditions (i.e. when the dam is full, such as after sustained high flow periods in wet weather) would be directed via the wetland as an environmental 'buffer; prior to discharge to the Brunswick River. Furthermore, the initial upgrade was proposed to include modifications to the existing inlet works to maximise its hydraulic capacity (e.g. by redirecting internal plant recycles). Capital expenditure for other additional treatment process infrastructure (notably bioreactor, aeration and clarifiers etc.) was proposed to be deferred for a number of years until required, based on dry weather flow and load projections.

2.1.3 Maximum treatment capacity and wet weather storage size

Apart from peak combined pumping capacity, the nomination of maximum sustained treatment capacity strongly influences the size (volume) of the proposed storm dam. Inappropriate selection of the nominated maximum sustained flow that the treatment process can sustain poses risks, due to inappropriate selection of the storm dam size during design. A lower nominated flow rate for maximum sustained treatment capacity will lead to a larger storm dam design capacity requirement, or a risk of more frequent and larger volumes of surplus flow discharges to the wetland/ river (see above). Conversely, a higher (more conservative) assumption of maximum sustained treatment capacity will lead to a smaller storm dam design capacity. However, if the actual sustained treatment capacity is less than the assumed value in this scenario, then more frequent and larger volumes of surplus flow discharges to the wetland/ river (see above) are expected.

2.2 Summary of approach

The Feasibility Study (GHD, 2016) examined historical data records for a four-year period (2012-2015 inclusive) based on combined M+BH catchment pump station totalised flow data at daily time-steps. These data were normalised to a ratio relative to adopted average dry weather flows (i.e. "times ADWF") for the base dataset. The required storm dam capacity was then identified by pro-rating the flows to expected future BVSTP ADWF scenarios after the transfer from Ocean Shores (i.e. combined M+BH+OS flows).

The purpose of the work summarised in this section was to test the validity of the storm dam capacity selection from the abovementioned approach in the Feasibility Study (i.e. nominally 20 ML working volume), using a more detailed assessment. For the detailed assessment here, short time-step data for flow records were taken from SCADA (i.e. 'real time' records) for the relevant catchment pump stations. Two actual recent wet weather events were modelled, namely:

- A series of smaller wet weather events, followed by a large event associated with the Tropical Cyclone Debbie in March-April 2017; and
- A shorter but relatively intense storm event in June 2017

These periods were selected because they included peak flow rates that matched the expected (design) maximum pumping rates for the pump stations in consideration.

A water balance model was built that considered a range of scenarios considered to reasonably reflect likely design or operational limits, as follows:

- Treatment process maximum sustained flow range: 176 to 251 L/s (or 4 to 5.7 times design ADWF of the existing treatment process)
- Return flow rate from storm dam (when operating): 22 to 44 L/s (0.5 to 1 times design ADWF of the existing treatment process)
- Threshold of plant inflows below which return pumping from storm dam is operated: 88 L/s (2 times design ADWF of the existing treatment process).

2.3 Summary of results

The water balance modelling results can be summarised as follows (see Appendix A for details):

- A storm dam with a **30 ML** capacity would be sufficient to store surplus flows (without surcharging), including rainfall capture, with the conservative assumption that the sustainable maximum process capacity of the treatment process is 176 L/s (4x design ADWF), which is 70% of the original design capacity. For this scenario, the required storm dam return pump maximum rate is 44 L/s (when operating).
- A storm dam volume capacity of approximately **20 ML** (16.7 to 22 ML) will provide sufficient storage for the following cases:
  - Smaller wet weather events, typified by the June 2017 event and assuming that treatment process sustained capacity is limited to 176 L/s (see above) and the storm dam return pump maximum rate is 44 L/s (when operating); or
  - Larger wet weather events, typified by the Mar-Apr 2017 event, provided that treatment process sustained capacity is at least 224 L/s (5.1 times design ADWF i.e. close to the design sustained capacity of 5.7x design ADWF) and the storm dam return pump maximum rate is 30 L/s (when operating).

- Smaller storm dam (or alternative wet weather storage facility) capacities, in the range approximately 7 to 24 ML, would be feasible if one (or more) of the following design assumptions is made:
  - Rainfall capture in the storage facility is excluded (e.g. by covering the storm dam; or by design of an alternative facility such a one or more roofed tanks with appropriate ventilation etc.).
  - Some level of frequency and volume of surplus settled/ diluted (but nominally untreated) wastewater is permitted to be discharged to the proposed wetland and Brunswick River under wet weather conditions, in terms of a new environmental licence to be negotiated with the EPA. Indicatively, assuming a 20 ML storm dam capacity was provided and treatment process capacity is limited to 176 L/s (at 44 L/s max. return pump rate), during a major wet weather event typified by the March-April 2017 series, modelling suggests that at least 6.8 ML of dilute raw wastewater (potentially further diluted by approximately 4.3 ML of captured rainwater for a total of approximately 11 ML) will be surcharged to the Brunswick River, via the constructed proposed wetland.
  - Higher sustained treatment process capacities, close to the original design capacity of 255 L/s, are achievable for the duration of the peak wet weather events (typically up to five days).

3. Process treatment constraints

3.1 Clarifier constraints

Refer to Appendix B for a more detailed technical description of modelling, results and recommendations around risks relating to the clarifiers.

3.1.1 Key constraints

BVSTP has two secondary circular clarifiers (23 m diameter each) that follow the oxidation ditch activated sludge bioreactor. The design of these clarifiers (Fulton Hogan/ Cardno, 2010) was relatively 'aggressive', both in terms of peak overflow rate and peak surface loading rate, compared to other (more conservative) designs targeting low effluent solids and nutrient concentrations (e.g. Byron STP).

The Feasibility Study for the OS-BVSTP transfer (GHD, 2016) identified that the process capacity of the BVSTP clarifiers is sometimes constrained during high wet weather events by high sludge blanket levels.

The design peak (sustained) flow rate for full treatment in the BVSTP clarifiers under wet weather conditions is 255 L/s (or 5.8 times design ADWF) at 90%ile settleability (SSVI 60 mL/g) and peak MLSS (4,900 mg/L). BSC operator experience[1] suggests that the operating capacity limit of the clarifiers lies somewhere in the range ~150 to 180 L/s at an MLSS of ~4,400 to ~5,000 mg/L.

As described in Section 2.1 above, the sustained treatment capacity of the clarifiers in wet weather affects the choice of size for the proposed storm dam, particularly during the period after the transfer of flow from Ocean Shores and prior to the (deferred) augmentation of the treatment process capacity (projected indicatively to be required no later than 2035-36) (GHD, 2016). The associated risks are either solids loss from the bioreactor and treatment process (leading to potential licence exceedances), due to wet weather flows exceeding actual clarifier capacity, or surplus wet weather flows surcharging from the proposed storm dam during peak weather events and discharging ultimately to the Brunswick River.

To mitigate these risks, by means of modelling, an assessment of the capacity of the existing BVSTP clarifiers to accept sustained high flows via the treatment process was undertaken.

3.1.2 Summary of approach

A combined probability modelling approach that incorporated modified flux theory was used to assess clarifier capacity. The key uncertainties in the model inputs were sludge settleability, mixed liquor suspended solids and flow rate.

An uncertainty distribution for sludge settleability was set up to reflect reasonable agreement with current sludge settleability at the treatment plant. Since only limited recent settleability index data was available, the distribution was also set up in a way that reflected the range in settleability index values roughly midway between the original BVSTP design assumptions and those of the earlier Byron STP (BSTP) design. That is, this study assumed a clarifier capacity basis for sludge settleability that is somewhat more conservative than the BVSTP design (in 2010) but somewhat less conservative than that of the BSTP design (in 2005).

Similarly, an uncertainty distribution for MLSS was set up to reflect recent current MLSS at BVSTP (which, for operational reasons, has tended to be at the higher end of the recommended

[1] GHD (2016) report Appendix D and confirmed in GHD (D de Haas) discussions with BSC (Ray Collins), 22 Nov. 2017

range, relative to the design values) as well as the design value (MLSS 4,900 mg/L, 90%ile adopted).

Uncertainty distributions in flow rate were modelled to reflect actual recent flows during two recent wet weather events (March- April 2017; and June 2017 – refer to Section 2.2 above).

3.1.3 Summary of results

The existing two secondary clarifiers were found to have a nominal operating limit of 176 L/s (4 times design ADWF of the existing process), with a residual risk of the probability of clarifier process 'failure' (i.e. gross solids loss) of approximately 3.4% or less during peak wet weather events typified by those in March-April or June 2017, as modelled. That is, if peak flow to the treatment process and clarifiers is limited to 176 L/s, there is less than a 1:30 probability (approximately) that the clarifiers would theoretically fail, in terms of solids separation requirements, during a peak weather event. That risk could be marginally further reduced (to approximately 1.2% to 2.8% probability or <1:35 to <1:83) by increasing the maximum capacity of the Return Activated Sludge (RAS) pumps from 150 L/s to 200 L/s (4 no. pumps operating i.e. both pumps in both clarifiers).

At a peak flow limit of 176 L/s to treatment and the clarifiers, the storm dam volume required was modelled to be 30 ML, including surface rainfall capture (refer to Section 2.3). Limiting the peak (sustained) flow directed to the treatment process and clarifiers to <176 L/s was not modelled. It was not deemed to be feasible, unless a larger storm dam capacity is warranted and considered both feasible and affordable.

A peak (sustained) flow limit directed to the treatment process and clarifiers of 176 L/s was considered to be reasonable as a risk mitigation strategy, and in line with operator experience for BVSTP. Further risk mitigation strategies were considered appropriate and feasible to leave negligible residual risk of clarifier process 'failure' under wet weather conditions. These strategies could include, for example, early detection of rising clarifier sludge blankets using additional instrumentation, coupled with automated switch off aeration in the oxidation ditch (using existing control systems) to allow temporary sludge storage in the bioreactor.

3.2 Bioreactor, aeration and solids constraints

Refer to Appendix C for a more detailed technical description of modelling, results and recommendations around risks relating to the oxidation ditch (bioreactor) and ancillary processes.

3.2.1 Key constraints

Apart from clarifier capacity (which is related – refer to Section 3.1), the key constraint on the other main treatment process units are related to plant loading, both in terms of mass and concentration of pollutants required to be removed. In this study, the process performance (i.e. capacity constraints) were examined at projected plant loadings for the year 2035/36 for the combined catchments of Mullumbimby, Brunswick Heads and Ocean Shores.

The projected loadings at a nominal date of 2035/36 was chosen (refer to OS-BVSTP Transfer Feasibility Study GHD, 2016 – Section 8.3 and Section 13) as a 'worst case', on the basis that the proposed process capacity augmentation could be deferred to no later than this date[2]. The previous study (GHD, 2016) identified that by 2035/36 the existing plant would be operating nominally at approximately 114% of this design load and higher process loadings would likely

[2] The initial plant upgrade was proposed to be limited to dealing mainly with hydraulic and flow-related constraints (e.g. wet weather storage capacity in a proposed storm, new diversion structure and inlet works modifications etc. – refer to Section 2.1). The upgrade of other treatment process capacity units would be deferred until required, based on projected population loadings.

be infeasible. For the augmented process capacity to be operational no later than 2035/36 (as proposed), planning, design and implementation would need to commence earlier (indicatively 2032/33).

Based on the above, the main objectives identified were to check the capacities of the following existing reactors or items of equipment at BVSTP using projected loadings in ca. 2035/36:

- Oxidation ditch aeration system (diffusers and blowers);
- Oxidation ditch solids inventory (MLSS);
- Aerobic digester aeration system (diffusers and blowers);
- Aerobic digester solids inventory (MLSS), volatile solids (VSS) destruction;
- Aerobic digester operation (DO, mode, decanting of supernatant etc.); and
- Alum dosing requirements.

3.2.2 Summary of approach

A BioWin™ process model was set up for the BVSTP treatment process, including the oxidation ditch with clarifiers as a continuous-flow process, and aerobic digester as an intermittently aerated and intermittently decanted sequencing batch reactor, as designed. Dynamic simulations were carried out using an actual diurnal flow profile derived from BVSTP flow records (in dry weather) and adopted diurnal concentration profiles for key loading parameters (COD, TKN, TP etc.). The design wastewater characterisation was adopted (Fulton Hogan/ Cardno 2010). These assumptions were checked against limited recent (31 October 2017) actual wastewater characterisation data for BVSTP, and key differences for a model run using the latest data (compared with runs using the adopted wastewater characterisation) were noted. It should be noted the BioWin™ process model was not fully validated.

3.2.3 Summary of results

The results and outcomes of the process modelling can be summarised as set out below. Refer to Appendix C for further details.

Oxidation ditch aeration system (diffusers and blowers)

- Adequate capacity with modelled nominally 'Clean' diffusers
- Inadequate/ marginal with diffuser condition modelled as nominally 'Dirty/ Partially fouled'
- Risk mitigation strategy
  - Good maintenance (inspection, cleaning, replacement) of diffusers, including regular in-situ chemical (acid vapour) cleaning. Good maintenance of existing (dual duty) blowers.
  - Process capacity augmentation (*Note 1*)

Note 1: If the catchment growth stagnates (population numbers served become stable), by 2035/36 then the need for capacity augmentation (second, new oxidation ditch) should be reviewed. A possible alternative to relieving the (peak) aeration capacity will be dry weather flow balancing (new tank with mixers and pumps required). However, dry weather flow balancing will not provide relief for the solids inventory constraints (see below).

Oxidation ditch solids inventory (MLSS)

- Adequate capacity if the plant is operated as designed, under average loading conditions.

- Design 90%ile MLSS 4,900 mg/L expected to be exceeded during peak (summer) month loading at 18 d Sludge Retention Time (SRT or sludge age). Increased sludge wasting (15d SRT) during peak summer months likely to be required.
- Risk mitigation strategy:
  – Adequate sludge wasting to ensure true sludge age (SRT) of 18 days average (15 days in peak month). Average MLSS ~4,650 to ~5,300 mg/L.
  – Process capacity augmentation (*Note 1 – see above*)

Aerobic digester aeration system (diffusers and blowers)

- Adequate capacity if the plant is operated as designed.
- Average TSS concentration in the aerobic digester should not be allowed to exceed 10,000 mg/L to avoid significant decrease in oxygen transfer efficiency, compared with typical design (modelled) assumptions.
- Risk mitigation strategy:
- Good maintenance (inspection and cleaning) of diffusers, including regular in-situ chemical (acid vapour) cleaning.
- Good maintenance of existing (single) duty blower.
- *See also:*
  – *Aerobic digester solids inventory* (below)
  – Process capacity augmentation (*Note 1- see above*

Aerobic digester solids inventory (MLSS), volatile solids (VSS) destruction

- Adequate if the plant is operated as designed (Note 2)
- Aerobic digester to be operated as intermittently aerated/ intermittently decanted batch reactor to include gravity sludge thickening by decanting supernatant via draw-off valves (existing).
- Automation of the existing manual supernatant draw-off valves recommended.
  – Installation of alternative decanter system in the existing digester recommended if the existing supernatant draw-off system is found to be operationally inadequate.
- Risk mitigation strategy:
  – Thicken waste activated sludge in digester by operation of the (existing) supernatant draw off pipes/ valves (to be tested; currently not in operation).
  – Sufficient operation of the dewatering equipment (belt filter press), to prevent solids recycle to mainstream process via decanted supernatant (maintain TSS in digester below ~10,000 mg/L). Indicative BFP operating times required: 6.7 h every two days, on average.
  – Installation of second (standby) belt press recommended (available space in existing sludge dewatering building).

Note 2: Sludge stabilisation (SOUR of digested sludge) not simulated in this study – requires additional modelling.

Alum dosing

- Adequate, if the plant is operated as designed, even with increased average alum dose rates (up to ~60 mg/L dry solid alum, compared with a 50%ile design value of 20 mg/L)

- Risk mitigation strategy:
  - Good maintenance of existing alum storage and dosing system.

4. Risk assessment

The main constraints and associated risks discussed in Sections 2 and 3 of this report are assessed in the Safety in Design risk matrix captured in Appendix D. Also included in the matrix are a number of other risks identified from a site visit and discussions with BSC personnel during this study.

It is recommended that all the risks and suggested mitigation measures captured in this study (summarised in Appendix D) be carried forward for consideration into future stages of planning and/or implementation of this project.

5. Recommendations

Based on the work outlined in this report, the following recommendations are made to mitigate identified hydraulic and/or process risks associated with the transfer of wastewater flows and loads from Ocean Shores to Brunswick Valley STP:

1. Unless otherwise determined by further modelling or alternative means prior to project implementation, adopt a peak (sustained) wet weather flow treatment capacity of **176 L/s** for the existing BVSTP treatment process, being limited by clarifier solids separation capacity. Refer also to Recommendation (6) below.
2. Build and install a storm dam (or alternative wet weather storage facility) with associated infrastructure prior to the transfer of wastewater from Ocean Shores. Subject to Recommendation (1) above, and unless otherwise determined by further modelling or alternative means prior to project implementation, adopt a storm dam capacity requirement of **30 ML** (active volume, including rainfall capture). The associated nominal design value for the return pump rate from the storm dam is **44 L/s.**
3. Subject to Recommendations (1) and (2) above, an alternative wet weather storage structure may have a smaller working volume, provided it is covered to exclude surface rainfall capture. **The minimum active storage volume recommended in line with the above is 24 ML, excluding rainfall capture and other contingencies**. Special design considerations will apply for covered structures to store raw wastewater (e.g. confined spaces; ventilation requirements; hazardous area classification; cleaning and maintenance requirements). These considerations lie outside the scope of this study but must be taken into account during subsequent stages of this project, and prior to implementation.
4. Operate the BVSTP in accordance with its original design, to ensure *inter alia* that:

a. MLSS concentration in the oxidation ditch (and associated anaerobic bioreactors) does not routinely exceed 4,900 mg/L (except for brief periods during peak holiday season loading)

b. A high MLSS inventory in the oxidation ditch is combatted by incrementally wasting more activated sludge to the aerobic digester. A nominal sludge age of around 15 days (i.e. lower than design value of 18 days) might be required in future years during peak holiday season loading.

c. Higher sludge wasting requirements are met by operation of the aerobic digester in accordance with its design, including:

   – Operation of the supernatant draw-off valves (to be automated automation, if necessary) for thickening purposes;
   – The (thickened) MLSS concentration in the aerobic digester does not exceed 10,000 mg/L
   – Adequate withdrawal of thickened/ digested sludge for dewatering to meet the above requirements.

5. Implement engineering controls to ensure high standards of maintenance are maintained at BVSTP, particularly for the following key items of process equipment:

   – Aeration diffusers (in oxidation ditch and aerobic digester), including routine inspection, physical cleaning, and regular in-situ chemical (e.g. acid vapour) cleaning and testing (e.g. back pressure);
   – Blowers (for oxidation ditch and aerobic digester); and

- RAS pumps.

6. Investigate options for upgrading the existing RAS pumps (whilst avoiding or minimising associated pipework modifications) to increase combined pumping capacity from 150 to 200 L/s (total RAS for 2 no. operating clarifiers). This will provide an additional, marginal improvement in existing clarifier capacity for dealing with peak wet weather flows in the range indicatively 176 to 224 L/s, subject to good sludge settleability performance.
7. Install a second (standby) gravity drainage deck-belt filter press in the available space within the existing dewatering building. This will improve reliability of dewatering to provide good operation in respect of solids inventory management for the plant at higher future loadings (see above).
8. Give attention to all items identified in the Safety in Design risk matrix (Appendix D) for future stages of planning and/or implementation of this project.

These recommendations should be read in conjunction with the scope and limitations outlined in Section 1.

Appendices

Appendix A – Technical memorandum: Water Balance modelling

Memorandum

16 November 2017

To	Byron Shire Council		
Copy to	Dean Baulch		
From	David De Haas	Tel	+61 7 3316 3715
Project	OS-BVSTP Transfer Process Risk Assessment		
Subject	Water Balance Modelling	Job no.	4131098

Dean

As recently discussed, this memo provides a summary of the outcomes from the water balance modelling that we conducted for Brunswick Valley STP storm dam capacity assessment.

1 Background

A recent Feasibility Study (GHD, 2016) proposed the transfer of raw wastewater flows and loads from the existing Ocean Shores catchments to the Brunswick Valley Sewage Treatment Plant (BVSTP), followed by the decommissioning of the older existing Ocean Shores STP.

The transfer of wastewater to BVSTP poses some process risks relating to plant capacity, which would need to be assessed and managed in order for the wastewater transfer to be feasible. One of the risks is the relatively high wet weather flow associated with the combined catchments of Mullumbimby (M), Brunswick Heads (BH) and Ocean Shores (OS). The peak wet weather flow (PWWF) from these combined catchments is expected to exceed the maximum design hydraulic and sustainable (process) maximum flow capacity of the existing BVSTP. Therefore, a storm dam was proposed in the Feasibility Study (GHD, 2016). The purpose of the storm dam will be to store flows in excess of the treatment process capacity during wet weather events. Typically, the contents of the storm dam would be pumped back to BVSTP for treatment once the flows entering the plant have subsided. However, in extreme cases, the storm dam might fill completely and surcharge. The surcharged flows will discharge to a constructed wetland proposed on the BVSTP site, and from there to the Brunswick River. Since the surcharged flows will bypass full treatment (only partial treatment or 'buffering' will occur in the wetland), some residual environmental and public health risk is posed to BSC, due to the environmental value and recreational use of the river.

The choice of storm dam capacity is a key consideration in mitigating the risks associated with managing wet weather flows at BVSTP. Assessment of the required storm dam capacity is dependent on a number assumptions relating to flow rates (into and out of the dam) - i.e. assumptions around the timing, frequency and magnitude of the high flow events.

The Feasibility Study (GHD, 2016) examined historical data records for a four-year period (2012-2015 inclusive) based on combined M+BH catchment pump station totalised flow data at daily time-steps. These data were normalised to a ratio relative to adopted average dry weather flows (i.e. "times ADWF") for the base dataset. The required storm dam capacity was then identified by pro-rating the

Memorandum

flows to expected future BVSTP ADWF scenarios after the transfer from Ocean Shores (i.e. combined M+BH+OS flows).

The purpose of the work summarised in this memo was to test the validity of the storm dam size selection from the abovementioned approach in the Feasibility Study, using a more detailed assessment. For the detailed assessment here, we used short time-step data for flow records taken from SCADA (i.e. 'real time' records) for the relevant catchment pump stations. Two actual recent wet weather events (in 2017) were selected as these included peak flow rates that matched the expected (design) maximum pumping rates for the pump stations in consideration.

2 Methodology

2.1 Flow data

Actual flow data (SCADA) records were supplied by BSC [1] for the following pump stations:

- SPS 2000 (BH) and SPS 4000 (M) combined flow meter: FIT5000 located at BVSTP inlet
- SPS 5009 and SPS 5004: both located in the Ocean Shores catchment (these are the two pump stations that will transfer flows to BVSTP after the proposed transfer pipeline has been built and commissioned).

The two wet weather flow events selected were:

- Extended event from late March- early April 2017 (tropical depression associated with the wake of Cyclone 'Debbie')
- Storm event on 12-13 June 2017

The flow data records supplied were at very short time steps (1 to 10 seconds). The datasets were too large to feasibly manipulate in a spreadsheet environment to perform the water balance calculations. For the purposes of this assessment, we judged that a 15-min data interval would be feasible and provide sufficient resolution. Therefore, the original flow data time series was manipulated (by means of integration [2]) to provide flow volumes (and hence flow rates) at 15-min time intervals.

The adopted flow rates (15 min data interval averages) are shown below as follows:

- Figure 1 for the March-April 2017 event (separate catchments)
- Figure 2 for the March-April 2017 event, combined catchments
- Figure 3 for the June 2017 event (separate catchments)
- Figure 4 for the June 2017 event, combined catchments

[1] Fleet Edwards (BSC) email to David de Haas (GHD) dated 16 & 22/8/2017.

[2] By using mathematical integration no loss of accuracy is expected since flow rate integrated vs. time yields flow volume. The adopted time series data was generated by differentiating the flow volume with respect to time at the new time step (i.e. 15 minutes).

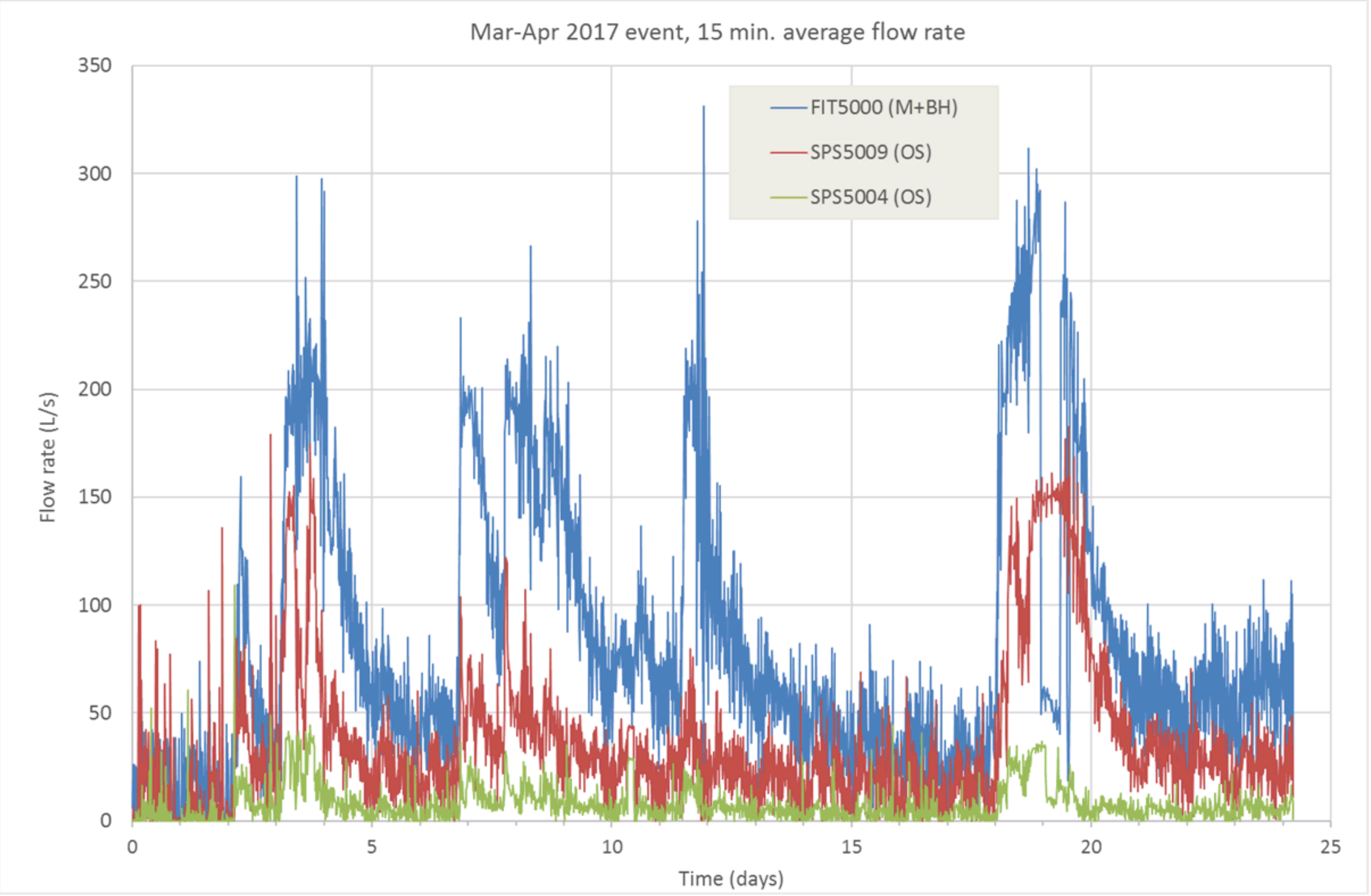


Figure 1. March-April 2017 event flow rates adopted (15 minute average calculated from the original SCADA dataset). Time 0 (days) is 12/3/2017 03:34:53.

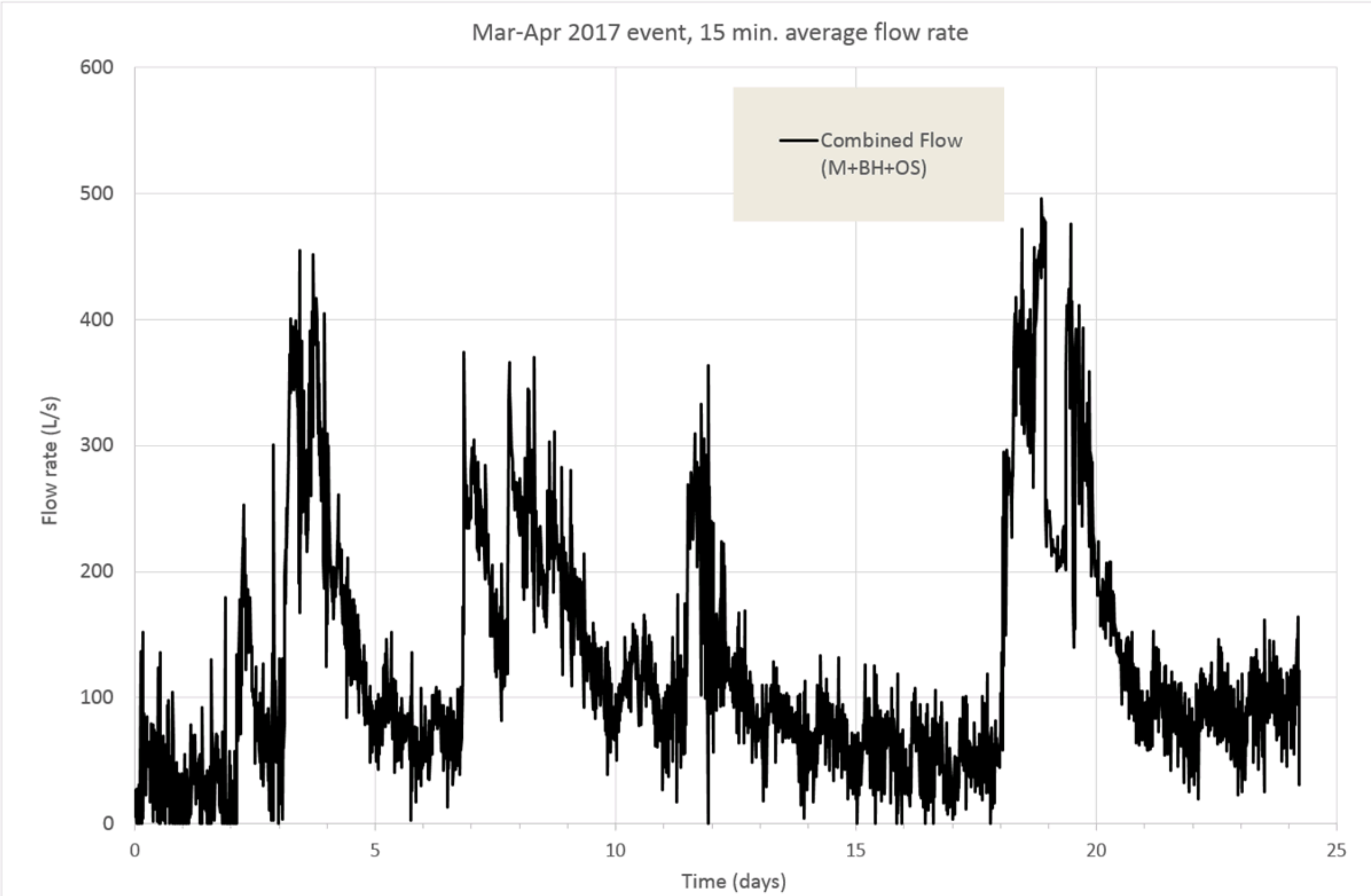


Figure 2. March-April 2017 event combined catchments flow rate adopted (15 min. ave. calculated from SCADA data). Time 0 (days) is 12/3/2017 03:34:53.

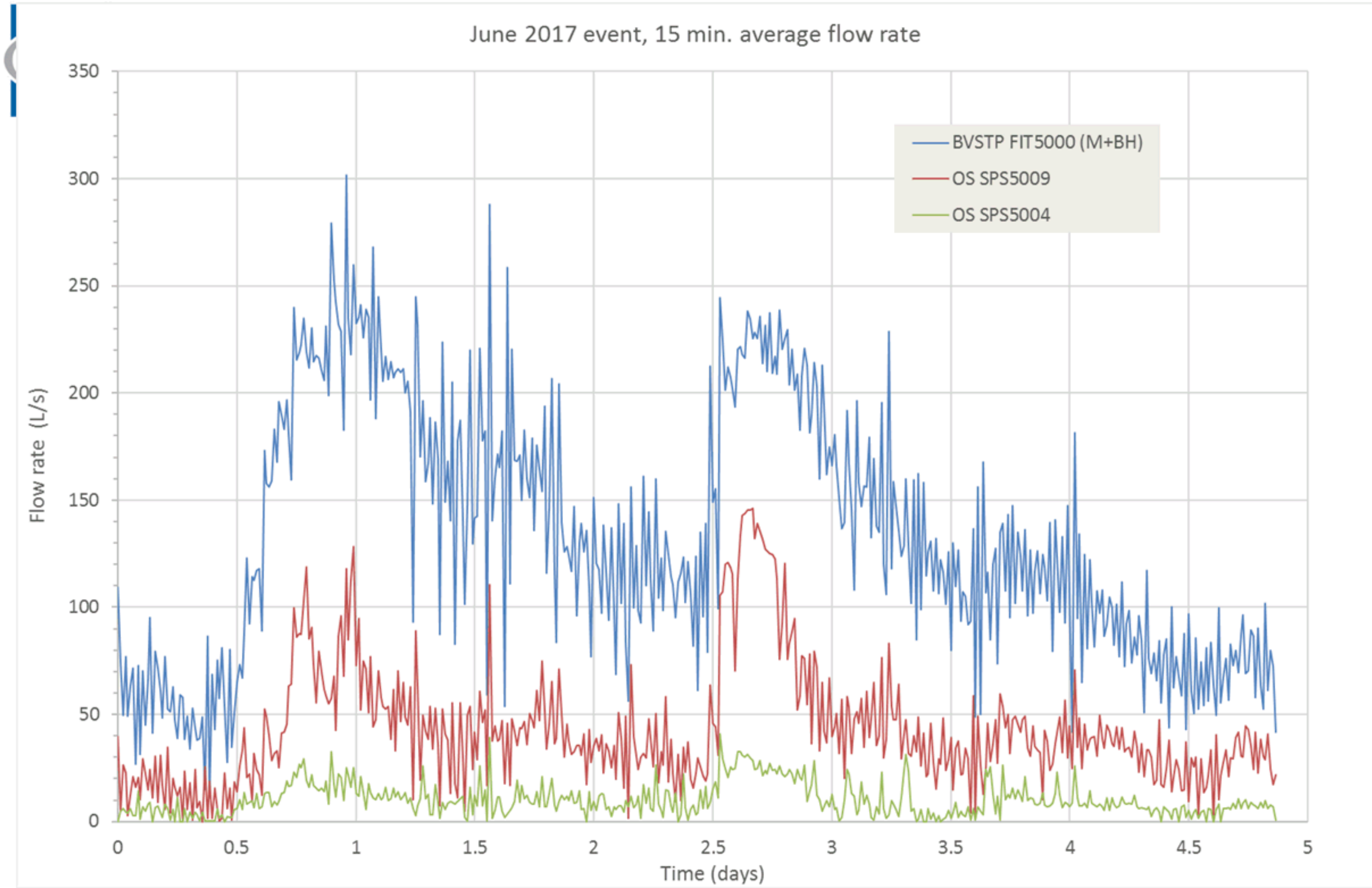


Figure 3. June 2017 event flow rates adopted (15 minute average calculated from SCADA data). Time 0 (days) is 10/6/2017 15:03:19.

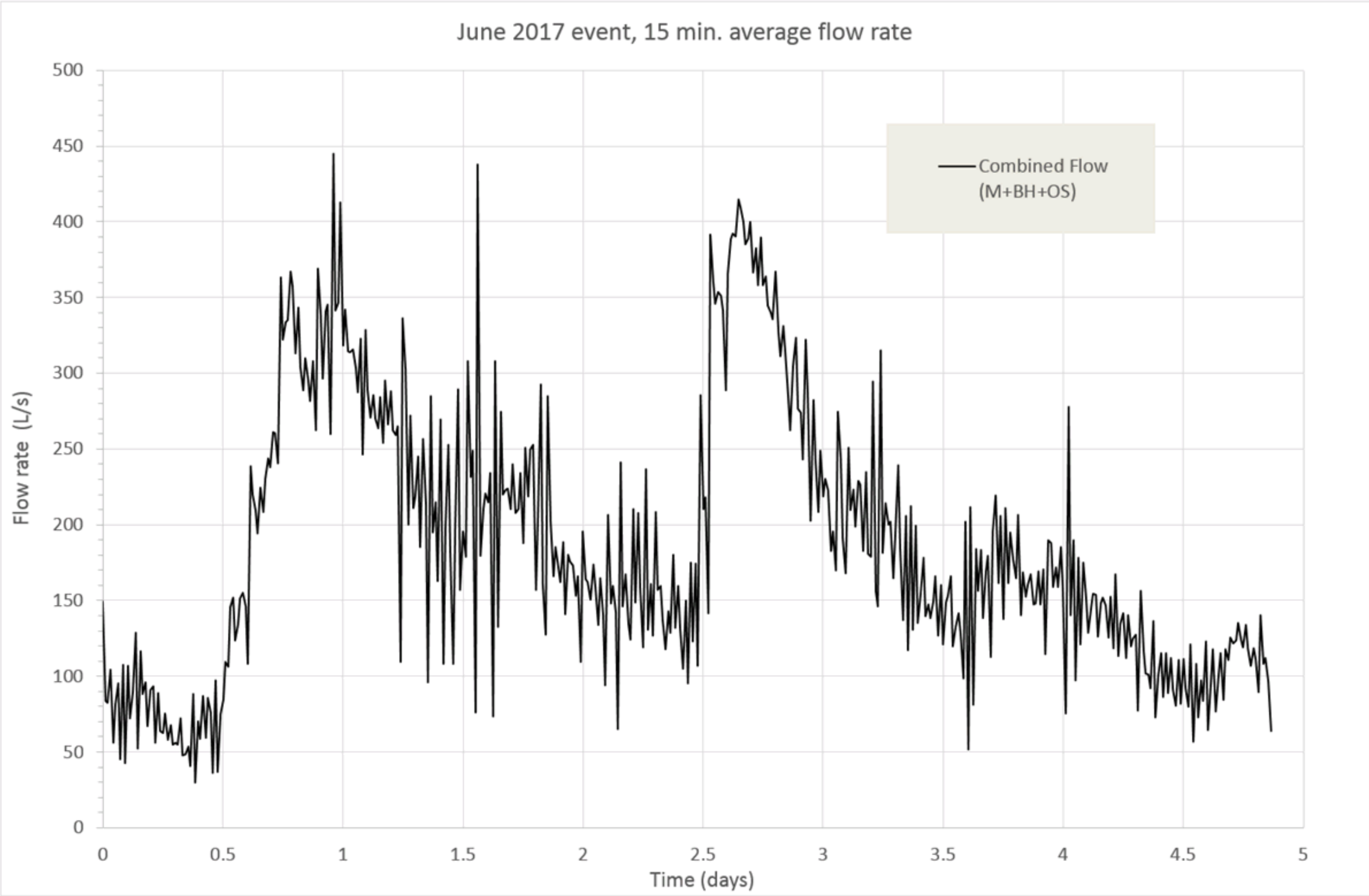


Figure 4. June 2017 event combined catchments flow rate adopted (15 min. average calculated from SCADA data). Time 0 (days) is 10/6/2017 15:03:19.

Memorandum

2.2 Water balance model

A water balance model was constructed for the storm dam. Refer to the schematic in Figure 5 below.

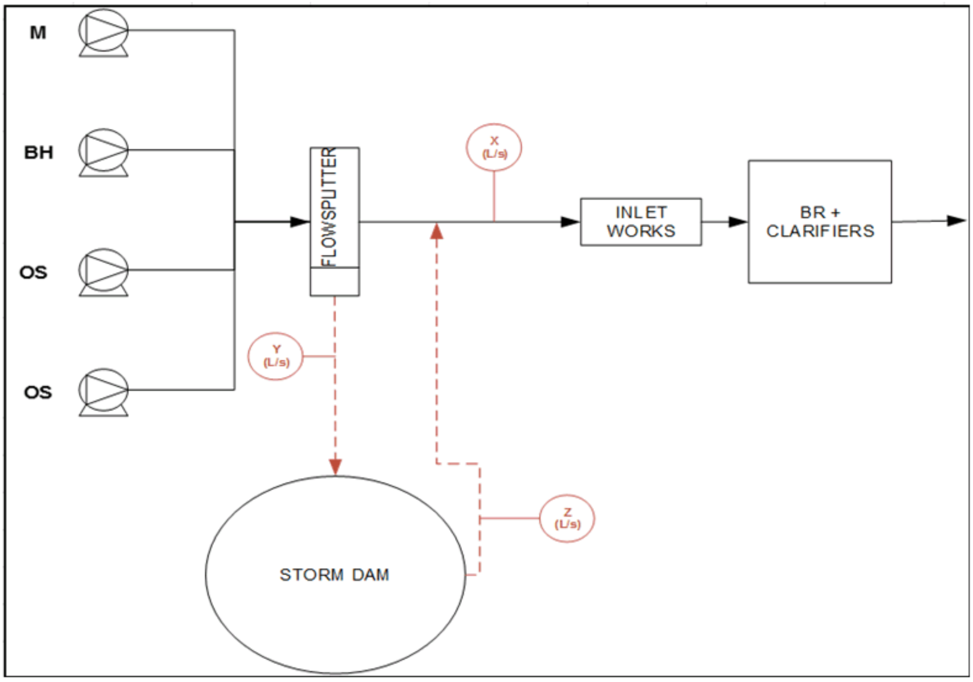


Figure 5. Schematic of water balance model (BR denotes bioreactor)

The design ADWF of the existing plant was taken as **3.8 ML/d (44 L/s)**, as per the Feasibility Study (GHD, 2016 and the original design report[3] (by Fulton Hogan, 2010)).

The model key variables were as follows:

- Treatment process Maximum (sustained) Flow Set point (X), sensitivity range tested:
  - 176 L/s (4x ADWF) in line with operational experience for the existing secondary clarifiers, RAS system and sludge settleability (refer to Section 5.2.2 of Feasibility Study (GHD, 2016)) ; to
  - 251 L/s (5.7x ADWF), as per original design (Fulton Hogan, see above)
- Setpoint for combined inflow to the plant (M+BH+OS) at which return flow from the storm dam is allowed was a constant in the model set at 88 L/s (2x ADWF). The rationale was that this flow threshold is indicative of typical peak dry weather flow rates for this plant. The addition of return

[3] Information supplied to GHD by BSC – refer to Feasibility Study (GHD, 2016)

Memorandum

flows from the storm dam (see below) would not be expected to place undue flux stress on the clarifiers, and therefore this setpoint is considered to be feasible (i.e. a reasonable assumption).

- Return flow from storm dam setpoint (Z), as follows:
  - Zero when the storm dam is empty
  - Sensitivity Range tested: 22 to 44 L/s (0.5 to 1x ADWF)
- Diversion to storm dam flow (Y), range calculated by difference
  - Combined inflow to the plant (M+BH+OS) *minus* X (see above).

The water balance (volume inventory) for the storm dam was calculated, using the above logic, by integration of the following flow rate equation with time at each time step (15 min intervals)

$$\frac{\partial V}{\partial t} = Q_Y - Q_Z$$

where Q_Y and Q_Z are defined above and V is the volume of water required to be stored in the storm dam (assuming no surcharge) and t is time.

Other assumptions were as follows:

- The storm dam was assumed to be empty (V = 0) at the start of the wet weather event modelled. This aspect was separately considered in the interpretation of the results i.e. contingency allowances in storm dam volume (see Section 3 below).
- Rainfall capture across the surface of the storm dam was not included in the model. For interpretation of the results (see Section 3 below), rainfall capture was considered on the basis that the dam surface area would be indicatively be in the range 1 to 1.5 ha (10,000 to 15,000 m^2) for a dam with a total storage capacity in the range 20 to 30 ML (water depth not exceeding 2.0 m).
- Highest cumulative rainfalls[4] for the wet weather events modelled were:
  - 29-31 March 2017: 433 mm (3-day cumulative max.); 441 mm (7-day cumulative max.)
  - 11-13 June 2017: 249 mm (3-day cumulative max.); 285 mm (7-day cumulative max.)

3 Results

3.1 Scenarios

The scenarios modelled are summarised in Table 1.

[4] Taken from BOM records for the Mullumbimby Fairview Farm (Station No. 58040)

Memorandum

Table 1. Model scenarios

Event	Scenario No.	Max. sustained flow treated via STP process (X), L/s *[times ADWF[5]]*	Return flow rate from storm dam when operating[6] (Z), L/s *[times ADWF]*
Mar-Apr 2017	A-1	176 *[4x ADWF]*	44 *[1x ADWF]*
	A-2	251 *[5.7x ADWF]*	22 *[0.5x ADWF]*
Jun 2017	B-1	176 *[4x ADWF]*	44 *[1x ADWF]*
	B-2	251 *[5.7x ADWF]*	22 *[0.5x ADWF]*

3.2 Graphical results

The model results are summarised graphically as follows:

- Figure 6 for Scenarios A-1 and B-1
- Figure 7 for Scenarios A-2 and B-2

3.3 Tabulated results

The model results are tabulated in Table 2. The tabled results for required storm dam capacity include allowances for rainfall capture across the surface of the storm dam. Refer to the table footnotes for related assumptions.

[5] Design ADWF = 44 L/s

[6] When storm dam is not empty and if plant flow inlet is <88 L/s (2 times design ADWF)

GHD

Memorandum

Table 2 Summary of model results

Event	Scenario No.	Max. sustained flow treated via STP process (X), L/s	Return flow rate from storm dam, when operating (Z), L/s	Storm dam required Volume for wastewater storage (ML)	Adopt Storm dam Volume Total	Surface area of storm dam (m^2), indicative	Maximum Rainfall capture (mm cumulative)
		[times ADWF]	*[times ADWF]*	Excluding rainfall capture	Including allowance for rainfall capture and/or contingencies	at Max. water depth = 2 m	Included in allowance for storm dam Volume Total adopted
Mar-Apr 2017	A-1	176 *[4x ADWF]*	44 *[1x ADWF]*	23.5	**30**	15,000	433
	A-2	251 *[5.7x ADWF]*	22 *[0.5x ADWF]*	12.5	**16.7**	8,350	503
	A-2 (alt)	224 *[5.1x ADWF]*	30 *[0.68x ADWF]*	15.7	**20**	10,000	433
Jun-17	B-1	176 *[4x ADWF]*	44 *[1x ADWF]*	17.2	**22**	11,000	436
	B-2	251	22	6.7	**9**	4,500	511

GHD

Memorandum

Event	Scenario No.	Max. sustained flow treated via STP process (X), L/s	Return flow rate from storm dam, when operating (Z), L/s	Storm dam required Volume for wastewater storage (ML)	Adopt Storm dam Volume Total	Surface area of storm dam (m^2), indicative	Maximum Rainfall capture (mm cumulative)
		[times ADWF]	*[times ADWF]*	Excluding rainfall capture	Including allowance for rainfall capture and/or contingencies	at Max. water depth = 2 m	Included in allowance for storm dam Volume Total adopted
		[5.7x ADWF]	*[0.5x ADWF]*				

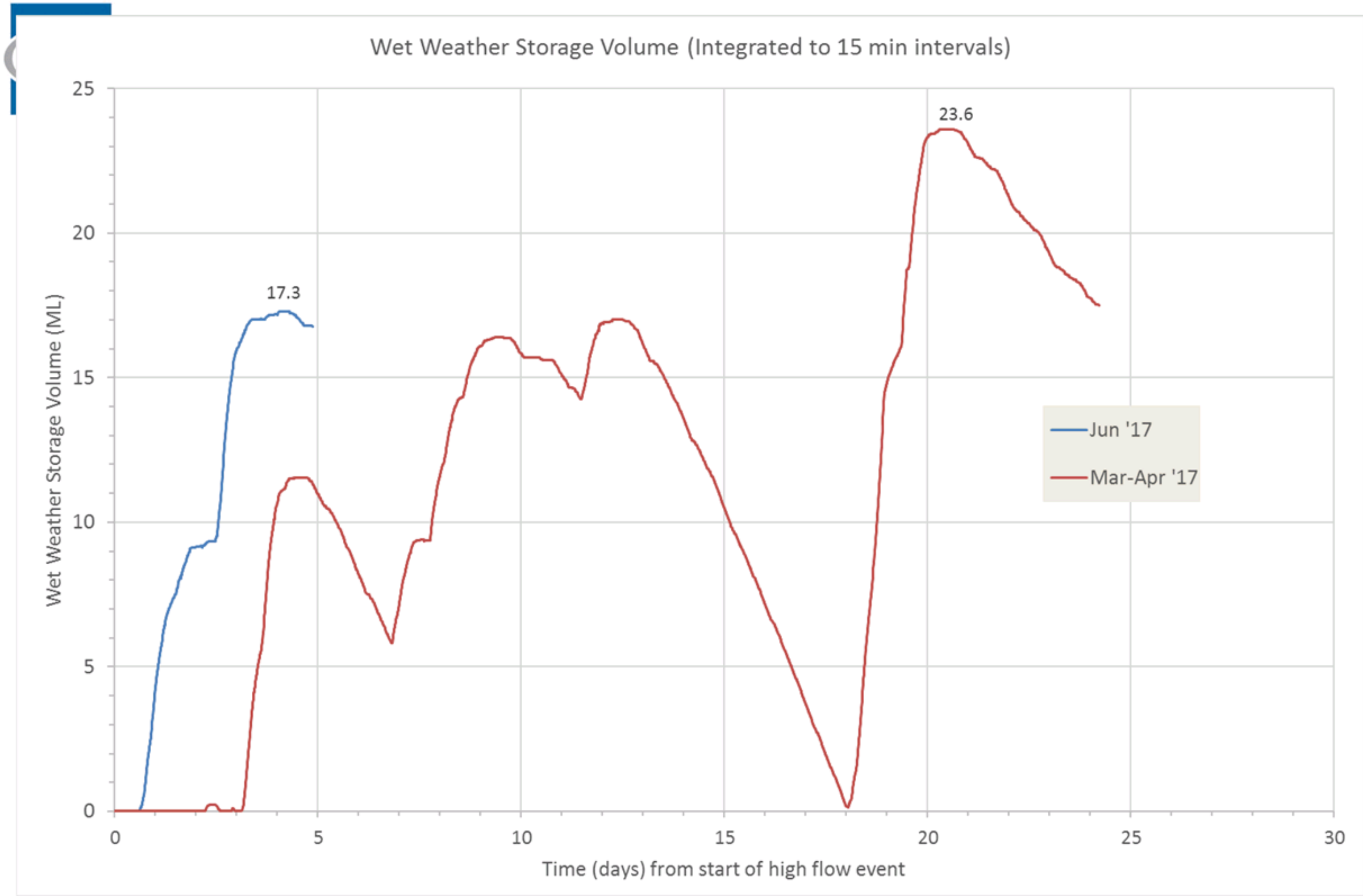


Figure 6. Water balance model predicted storm dam storage volume required (excluding site rainfall capture) for Scenarios A-1 (Mar-Apr '17) and B-1 (Jun '17).

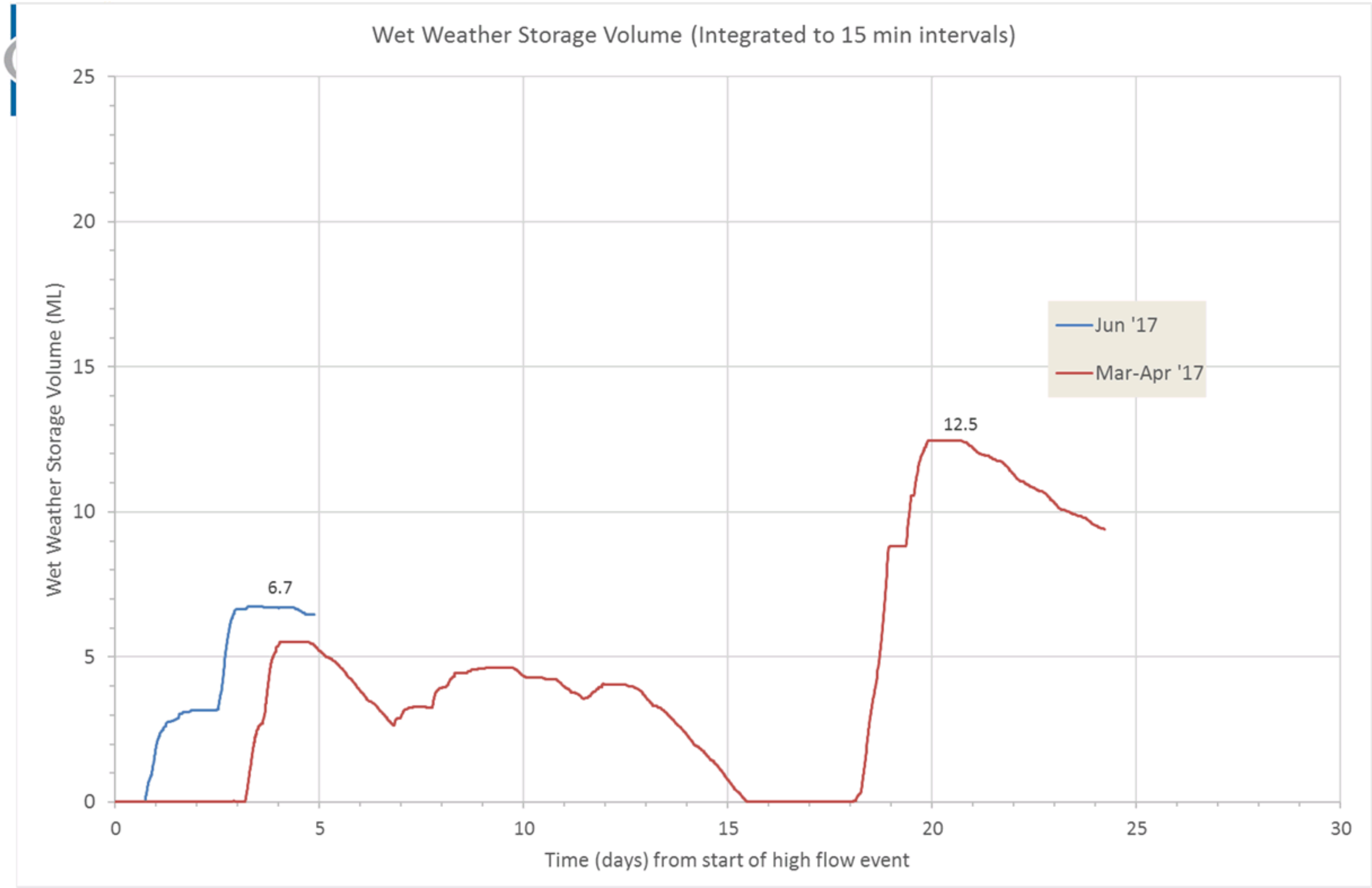


Figure 7. Water balance model predicted storm dam storage volume required (excluding site rainfall capture) for Scenarios A-2 (Mar-Apr '17) and B-2 (Jun '17).

GHD

Memorandum

3.4 Interpretation

The following key points can be highlighted from the model results (Section 3.3, Table 2):

- Scenario A-1: A storm dam storage volume capacity of **30 ML** will provide theoretically 100% reliability for major wet weather events typified by the March-April 2017 event. During that event, the total inflow to the BVSTP (from the combined catchments, including Ocean Shores after the transfer) reached a peak of just under 500 L/s (496 L/s as a 15 min. average) and the cumulative maximum three-day rainfall was 433 mm. A storm dam with a 30 ML capacity would be sufficient to store surplus flows (without surcharging), including rainfall capture, with the conservative assumption that the sustainable maximum process capacity of the treatment process is **176 L/s** (4x design ADWF), which is 70% of the original design capacity. The return pumping rate from the storm dam was assumed to be at most 44 L/s (when operating) and the return pumps will not operate[7] unless the inflow to the process is less than 88 L/s (2x design ADWF). The conservative treatment capacity assumption is based on information supplied by the operators, based on recent experience with the plant, and supported by a preliminary assessment of clarifier capacity (refer to Feasibility Study, GHD, 2017). Further clarifier modelling, based on update sludge settleability data for the plant, will be required to confirm this estimate.
- A storm dam volume capacity of approximately **20 ML** (16.7 to 22 ML) will provide sufficient storage for the following cases:
  - Scenario B-1: Smaller wet weather events, typified by the June 2017 event and assuming that treatment process sustained capacity is limited to 176 L/s (see above)
  - Scenario A-2 (alt): Larger wet weather events, typified by the Mar-Apr 2017 event, provided that treatment process sustained capacity is at least **224 L/s** (5.1 times design ADWF i.e. close to the design sustained capacity of 5.7x design ADWF) and the return rate from the storm dam is at most 30 L/s (when operating).
- By way of example, a smaller storm dam volume capacity will provide sufficient storage for the following cases:
  - Scenario A-2: **17 ML** storage capacity will be sufficient for larger wet weather events, typified by the Mar-Apr 2017 event, provided that treatment process sustained design capacity of at least **251 L/s** (or 5.7x design ADWF) is achieved and the return rate from the storm dam is at most 22 L/s (when return is operating).
  - Scenario B-2: **9 ML** storage capacity will be sufficient for smaller wet weather events, typified by the June 2017 event, provided that treatment process sustained design capacity is achieved (**251 L/s**, see above) and the return rate from the storm dam is at most 22 L/s (when operating).
- If the storm dam is not empty at the start of the wet weather event, then the capacity for surface rainfall capture by the storm dam will be reduced commensurately from the tabulated values (Table 2).

[7] Common assumption for all scenarios

GHD

Memorandum

- For Scenario A-1, if the storm dam storage volume provided is limited to 20 ML, then at least 6.8 ML of diluted raw wastewater (potentially diluted by approximately 4.3 ML of captured rainwater for a total of approximately 11 ML) will be surcharged to the Brunswick River (via the constructed proposed wetland) during a major wet weather event typified that which occurred in Mar-Apr 2017.

4 Summary

Using daily time step data and excluding rainfall capture considerations, the Feasibility Study (GHD, 2016) suggested the following storm dam storage volumes and reliabilities[8] for a future scenario (indicatively reached somewhere in the years 2035-37) at a projected[9] ADWF = 4.5 ML/d for the combined catchments:

- Assumptions (similar to Scenario A-2 in this study; refer to Table 1):
  - Sustained treatment capacity is 265 L/s (5.1 times projected ADWF);
  - Storm dam return rate is 22 L/s when plant inflows are <2 times projected ADWF
  - Rainfall capture not included.
- 20 ML: 97.5% reliable (surcharging indicatively 9 days per annum, on average)
- 31 ML: 99.7% reliable (surcharging indicatively 1 day per annum, on average)

A storm dam capacity of 20 ML was used for the recommended option (Option 4) capital cost estimate in the Feasibility Study (GHD, 2016).

Based on real wet weather event data, and including an allowance for rainfall capture in the storm dam, this study has confirmed that a storm dam storage volume of **at least 20 ML** is required for BVSTP to cater for the combined catchments after the transfer of wastewater from Ocean Shores. However, this volume will **only** be adequate if a process treatment capacity of **at least 224 L/s can be reliably sustained** (i.e. at least 90% of the design maximum process treatment capacity).

Current indications are that the reliable maximum treatment process capacity at BVSTP is closer to 176 L/s (or about 70% of the design capacity), rather than 224 L/s. This aspect requires further investigation, focussing on clarifier (and/or RAS) process capacity, which appears to be the limiting factor. **If the process capacity cannot be reliably increased to approach design capacity (see above), then a larger storm dam storage volume (indicatively 30 ML) will be required.** The final choice of storm dam volume will depend on a balance of factors relating to process risks (as discussed above), capital cost and feasibility (e.g. site and related civil engineering design constraints).

5 Recommendations

The following recommendations are made arising from this study:

[8] Reliability defined as probability that the storm dam will not surcharge (from four years of flow data at daily time steps)

[9] Current ADWF (2016-17) was estimated to lie in the range 3.2 to 3.6 ML/d for the combined catchments (M+BH+OS).

GHD

Memorandum

1. By means of process modelling using updated plant data (including sludge settleability), further investigate the risk that reliable (i.e. sustained) maximum treatment capacity at BVSTP is significantly less than the design capacity.
2. If the sustained maximum treatment capacity is significantly less than 224 L/s peak wet weather flow (i.e. approximately 90% of the design value or 5 times existing design ADWF), then the risk of surplus wet weather flow surcharging from the proposed storm dam (ultimately to the Brunswick River) should be mitigated by increasing the storm dam capacity provided (i.e. >20 ML storage volume), before the transfer of flows from Ocean Shores.
3. If the sustained maximum treatment capacity 176 L/s peak wet weather flow (i.e. approximately 70% of the design value or 4 times existing design ADWF), then a storm dam capacity of at least 30 ML is recommended.
4. The final selection of storm dam capacity can be delayed until the reliable maximum treatment process capacity has been confirmed (see above). It will ultimately depend on a balance of factors relating to risk, capital cost and feasibility (including site and design considerations). For projected flows up to ADWF 4.5 ML/d (indicatively by year 2037), a storm dam capacity in the range 20 to 30 ML is recommended at this stage, subject to treatment capacity confirmation, as described above.

David De Haas
Principal Professional (Wastewater Treatment)

Appendix B - Technical memorandum: Clarifier modelling

Memorandum

14 December 2017

To	Byron Shire Council		
Copy to	Dean Baulch		
From	David de Haas	Tel	(07) 3316 3715
Subject	BVSTP Clarifier capacity modelling	Job no.	41/31098

1 Introduction

1.1 Background

On behalf of Byron Shire Council (BSC), we carried out a previous study (GHD, 2016) that examined the feasibility of transfer of wastewater from Ocean Shores STP (OSSTP) to Brunswick Valley STP (BVSTP). The idea is to consolidate treatment at BVSTP, followed by the closure of OSSTP. That study identified a potential deficit in clarifier capacity at BVSTP to treat the projected future loads of the combined catchments, in the longer term. In the short to medium term (ca. 2020 to 2035/36) the combined loads of the catchments are predicted to take BVSTP close to its design capacity, or marginally over at ~115% of design capacity by 2035/36. The need was identified for additional clarifier (as well as bioreactor) capacity to be provided for BVSTP, particularly in the medium to long term. The recommended option was to proceed with the transfer from OSSTP and to undertake essential capital upgrade works in the short term to enable the transfer of flows (e.g. provision of a new flow splitter structure, modifications of inlet works, and wet-weather storage dam). In the recommended option, the provision of additional bioreactor and clarifier capacity would be deferred until ca. 2035-36 when growth of the catchments necessitated it.

Before proceeding with the recommended option for OS-BVSTP transfer (see above), it was further recommended that the process risks associated with operating the existing BVSTP mainstream process (i.e. oxidation ditch bioreactor and clarifiers) at or slightly above design capacity be better quantified, understood and accepted by BSC.

The aim of this memo is to document the outcomes of further modelling undertaken specifically to quantify the process capacity limits and associated risks relating to the BVSTP clarifiers in the context of the proposed transfer of additional wastewater loads from OSSTP.

1.2 Previous work

A summary of the previous work[1] (GHD, 2016) is given below.

[1] Extract from Sections 6.4 and 6.5 of the Report no. 41/28941/46789 (GHD, 2016) to BSC, dated 23 November 2016 (rev0).

1

Memorandum

1.2.1 Existing clarifier design

BVSTP is currently equipped with two (2 no.) circular clarifiers. Refer to Table 1 for the design basis of these clarifiers and a comparison with the Byron STP clarifiers, which are similar but have more conservative design assumptions (e.g. in respect of sludge settleability).

Table 1 Comparison of design basis for existing Brunswick Valley and (West) Byron STP clarifiers

Design parameter	Units	BVSTP	(W)BSTP	Notes
Number of clarifiers	No.	2	2	
Diameter, each	m	23	33	
Area, each	m^2	415	855	
Area, total	m^2	831	1711	
Design Stirred SVI, 90%ile	mL/g	59	120	See Note 1
Design MLSS, Peak (90%ile)	mg/L	4,900	4,500	See Note 1
Design ADWF	ML/d	3.8	6.95	
Maximum design hydraulic flow (instantaneous)	(xADWF)	7.1	7	
Peak design process flow for full treatment	(xADWF)	5	3	
Mixed liquor by-pass	-	No	Yes	
Max. RAS ratio at peak flow	(xADWF)	3.5	2	
Ave. surface solids loading rate at average flow, ave. MLSS, excl. RAS	$kg/(m^2.h)$	0.75	0.50	Ave. Overflow rate x MLSS
Peak surface solids loading rate at maximum hydraulic loading rate incl. RAS	$kg/(m^2.h)$	9.9	5.9	Without reactor mixed liquor by-pass operating
	$kg/(m^2.h)$	N/A	2.5	With reactor mixed liquor by-pass operating (>3 ADWF)
Peak surface solids loading rate for full treatment incl. RAS	$kg/(m^2.h)$	7.9	3.3	
Peak overflow rate	m/h	1.35	1.19	At max. hydraulic flow rate
	m/h	0.95	0.51	At peak process design flow rate (full treatment)

Stirred SVI: Stirred Sludge Volume Index

BVSTP: Brunswick Valley STP; (W)BSTP: (former West) Byron STP

Note 1: According to the designers report (John Holland/ Cardno/ Ken Hartley, 2005), the (W)STP clarifiers (2 no., 33 m diameter each) were conservatively designed for a low effluent suspended solids (<4 mg/L) at a low average solids loading rate of 0.5 $kg/(m^2.h)$ at a 50%ile MLSS of 3,000 mg/L. Elsewhere the design report states that the worst clarifier loading condition was considered to be at the 'maximum' MLSS of 3900 mg/L and 50%ile SSVI of 90 mL/g. Cross-checking with modified flux theory calculations predicted the nominated clarifier size (2 no. 33 m dia. each) at SSVI = 120 mL/g and MLSS 4,500 mg/L, which are the tabulated (90%ile) values.

1.2.2 Clarifier flux model results

Refer to Table 2.

2

Memorandum

1.2.3 Interpretation

The clarifier model results from the previous study illustrate operational issues, which have been found with the BVSTP from time to time during peak wet weather events. In summary, the following points can be noted from the previous study model results:

- The existing clarifiers (2 no. 23 m diameter) have a relatively 'aggressive' design, being for a design settleability of SSVI = 59 mL/g (90%ile). That is, the design assumed significantly better settleability than more conservative designs (e.g. previously at (West) Byron STP with had a 50% design SSVI of 90 mL/g, which corresponds to an SSVI of ~120 mL/g on a 90%ile basis). This is illustrated in Table 13 (see above). Table 13 shows that the existing clarifiers have a margin of safety (25% spare capacity) at sustained process peak flows of 5.8 times ADWF or 255 L/s (Case 1.1), and zero margin of safety (0% spare capacity) at a peak flow of 7.1 times ADWF or 312 L/s (Case 1.2), where ADWF is 3.8 ML/d (44 L/s) for the existing plant.
- Given that the actual settleability at BVSTP might at times be worse than the design settleability (e,g, SSVI range ~60 to 90 mL/g reported during post-commissioning optimisation in ca. 2012-13), it is not surprising that the operators anecdotally report problems with biomass retention under sustained peak flow conditions. Table 13 (Cases 2.1 and 2.2) shows that theoretically the clarifiers have a 'deficit' in capacity (i.e. a tabulated negative value for spare capacity) for the combination of peak month design MLSS (4900 mg/L) and an SSVI of 90 mL/g. This correlates with information supplied by BSC[2] that aeration in the oxidation ditch is switched off at peak inflow rates above 150 L/s in order to limit the solids loading on the clarifiers and prevent potential solids loss issues, due to rising sludge blankets.
- Based on a more conservative assessment, including allowance for sustained future peak flows from the combined Mullumbimby, Brunswick Heads and Ocean Shores catchments, the GHD (2016) feasibility study recommended that provision be made in the plant augmentation for a minimum clarifier process capacity of sustained operation at 6 times ADWF or 396 L/s (where the augmented plant ADWF is 5.7 ML/d or 66 L/s).
- Using a more conservative sludge settleability (SSVI 90 mL/g i.e. the Byron STP design 50%ile value), provision for two new clarifiers (23 m diameter each to match the two existing clarifiers) for the plant augmentation was recommended (GHD, 2016).

With a total of 4 no. 23 m diameter clarifiers (100% augmentation) provided in future, compared with only 50% bioreactor process capacity augmentation), a change in plant flow splitting and operating philosophy will be required. These changes are described in the GHD (2016) report, but in summary will entail the following:

- The new process train (one third of total bioreactor capacity after plant augmentation) will be hydraulically coupled to the two new clarifiers (representing one half of the total clarifier capacity after augmentation).
- A new raw influent flow splitter upstream of inlet works will be provided to split the flow in a ratio commensurate with future bioreactor capacity, namely:
  - Nominally 33% to the new bioreactor and 67% to the existing bioreactor under dry weather conditions (i.e. time-averaged influent flow rates nominally less than 2 times design ADWF); and

[2] Information supplied to GHD (D de Haas) by BSC (Ray Collins), during discussions on 22 Nov. 2017.

3

Memorandum

- Nominally 50% to each bioreactor (new and existing) under wet weather conditions (i.e. time-averaged influent flow rates nominally greater than 2 times design ADWF)

- Surplus wet weather flows (time-averaged influent flow rates nominally greater than 6 times design ADWF) will be diverted to a new wet weather storage facility. Provision to divert more flow to the storage facility will be made, which will be an 'emergency' operational strategy invoked by the plant operators, if required (e.g. if one or more clarifiers is out of service).
- A new RAS flow splitter will be provided downstream of the inlet works and upstream of the bioreactors. The purpose of the RAS flow splitter will be to combine the RAS from all four clarifiers (new and existing), to provide RAS screening and then to re-divide the RAS in proportion to process requirements.
- A new mixed liquor flow splitter will be provided downstream of the bioreactors to combine mixed liquor flows (influent and RAS) from the two process trains and then to re-divide the combined flow in proportion to the number of clarifiers that are on line, for example:
  - 25% to each clarifier with 4 no. clarifiers on line
  - 33% to each operating clarifier with 3 no. clarifiers on line (1 no. off line)

  Note: Mixed liquor flow splits to the clarifiers will not be directly related to dry vs. wet weather flow considerations.

Memorandum

Table 2 Summary of Previous Study (GHD, 2016) Clarifier Flux Model results for BVSTP

	CLARIFIER FLUX CALCULATIONS - KEY OUTPUTS												
			Assuming: Peak month MLSS = 4900 mg/L; SSVI = 59 mL/g (BVSTP design 90%ile)										
Model Case No.	Scenario	Mixed liquor bypass	ADWF (ML/d)	PWWF/ ADWF ratio to clarifiers	PWWF (L/s)	Max. RAS (L/s) per clarifier	No. of Clarifiers	Required Clarifier Total Area (m^2)	Existing Clarifier Total Area (m^2)	Required Clarifier Diameter (m) each	Existing (or proposed) Clarifier diameter (m) each	Approx. spare clarifier capacity (% of total area provided)	Notes
Case 1.1	Current Design at 5.8 ADWF	No	3.8	5.8	255	77	2	622	831	19.9	23.0	25%	Existing clarifiers do not have reactor flow-bypass facilities; RAS is recycled via inlet works for screening
Case 1.2	Current Design at 7.1 ADWF	No	3.8	7.1	312	77	2	834	831	23.0	23.0	0%	Ditto
			Assuming: Peak month MLSS = 4900 mg/L; SSVI = 90 mL/g (approx. BVSTP actual 90%ile; Byron STP design 50%ile)										
Case 2.1	Current Design at 5.8 ADWF	No	3.8	5.8	255	77	2	1283	831	28.6	23.0	-54%	Existing clarifiers do not have reactor flow-bypass facilities; RAS is recycled via inlet works for screening
Case 2.2	Current Design at 7.1 ADWF	No	3.8	7.1	312	77	2	2091	831	36.5	23.0	-152%	Ditto
Case 3.1	Proposed Future Design at 6 ADWF	No	5.7	6.0	396	77	4	1660	1662	23.0	23.0	0%	Proposed 50% ADWF and bioreactor capacity plant augmentation. For consistency with current design, asssume new reactor and clarifiers will also not be equipped with reactor flow by-pass
Case 3.2	Proposed Future Design at 7.1 ADWF	No	5.7	7.1	468	77	4	2170	1662	26.3	23.0	-31%	Ditto

5

Memorandum

1.3 Wet weather storage and related assumptions

Since the GHD (2016) OS-BVSTP transfer study, more detailed water balance modelling was undertaken for the wet weather ('Storm dam') storage facility proposed for BVSTP. Among the key conclusions from interpretation of that water balance modelling was the following:

- A storm dam volume capacity of approximately 20 ML (16.7 to 22 ML) will provide sufficient storage for the following cases:
  - Smaller wet weather events, typified by the June 2017 event and assuming that the ***treatment process sustained capacity is limited to 176 L/s*** (a conservative assumption that the sustainable maximum process capacity of the existing treatment process is 176 L/s i.e. 4 times design ADWF), which is 70% of the original design capacity)
  - Larger wet weather events, typified by the Mar-Apr 2017 event, ***provided that treatment process sustained capacity is at least 224 L/***s (i.e. 5.1 times design ADWF of the existing treatment process i.e. close to the original design capacity of 5.7x design ADWF sustained) and the return rate from the storm dam is at least 30 L/s (when operating).
- A larger storm dam volume capacity of 30 ML will provide sufficient storage for the following cases:
  - Larger wet weather events, typified by the Mar-Apr 2017 event, ***if the treatment process sustained capacity is limited to 176 L/***s
  - Increased reliability for handling wet weather events in general, and approaching 100% (99.7% reliability) for the combined catchments in future after the plant has been upgraded, provided ***the sustained treatment capacity is at least 265 L/s*** (or 5.1 times ADWF = 4.5 ML/d projected for the combined catchments in 2035/36).

1.4 Objective

The objective of this memo was to document the outcomes of a more detailed assessment of the existing BVSTP clarifier capacity. The aim was to better understand the process risks associated with operating the plant in the short to medium term with limited clarifier capacity but with a wet weather storage facility (storm dam). The key process risk is the limitation posed by the existing clarifier design (i.e. risk of environmental licence non-compliance, due clarifier capacity being either reached or exceeded).

For a given size of clarifier and number of clarifiers (i.e. both) on line during peak flow events, the key variables that limit clarifier capacity are:

- Sludge settleability
- Maximum mixed liquor suspended solids (MLSS), at time of peak flow events
- Maximum ('peak') flow
- Return Activated Sludge (RAS) rate (pump maximum capacity)

This study examined each of these variables with a view to quantifying the risk of process capacity being exceeded.

2 Methodology

A combined probability assessment of clarifier capacity was undertaken as a tool to quantify risk. Clarifier capacity was modelled using modified flux theory (Ekama et al., 1997). Modified flux theory assumes that

6

the maximum sustainable flux (solids loading rate) is set to no more than 80% of the theoretical maximum in standard (idealised) flux theory. In this way, the modified theory makes allowance for non-idealities (e.g. flow distribution, solids collection, baffling, wind effects etc.) in real clarifiers that idealised flux theory does not take into account.

Memorandum

2.1 Model inputs and uncertainties

The uncertainties discussed below were incorporated within the modified flux model applied here.

2.1.1 Sludge settleability

Settleability Data

The design settleability (Stirred Sludge Volume Index, SSVI) assumptions of BVSTP are given in Table 1. Very limited recent settleability data was vailable and no equipment[3] available on site at the STP to measure SSVI. For the purposes of this study, the operators measured undiluted and diluted Sludge Volume Index (i.e. SVI or DSVI) over the period of week (in Oct. 2017) and the available data is listed in Table 3. Due to the limited settling that occurred without dilution, the SVI results were considered to be less reliable and not used in this study. Similarly, the result on 6/10/2017 was considered to be unreliable, since it was insufficiently diluted, and not used here.

The remainder of the DSVI tests showed an average value of 108 mL/g (Table 3).

It is noted from the MLSS results (Table 3) that the plant is currently operating in excess of its design 90th percentile MLSS. This is one factor that will limit clarifier capacity – refer to Section 2.1.2.

Table 3 Recent sludge settleability data for samples from the BVSTP Oxidation Ditch

Date	Dilution factor (-)	SV(30 min) mL/L	MLSS mg/L	DSVI mL/g	SVI mL/g
4/10/2017	0	900**	5330		169**
	0.333	190	5330	107	
5/10/2017	0.333	190	5350	107	
6/10/2017	0.667	520**	5330	146**	
9/10/2017	0	890**	5030		177**
	0.333	180	5030	107	
10/10/2017	0	880**	5810		151
	0.333	210	5810	109	

** Data considered less reliable (not used)

Settleability distribution functions

In the absence of a larger actual dataset, the assumptions listed in Table 4 were made for the purposes of establishing probability distribution functions of sludge settleability metrics.

[3] A 'settlometer' is required.

8

Memorandum

Table 4 Sludge settleability characteristics adopted for this study

Percentile	SSVI (mL/g)	DSVI (mL/g)
10th percentile	50	75
50th percentile	67	100
90th percentile	90	135

Assumption[4]: SSVI = 0.67 * DSVI

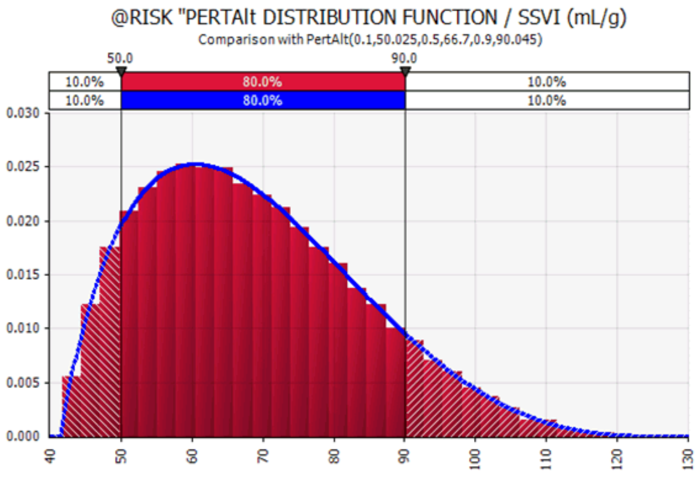
Important note: The sludge settleability characteristics for this study adopted were more conservative than the original design for BVSTP, but less conservative than the design values for (W)BSTP (refer to Table 1). The adopted 50th percentile DSVI aligns reasonably well with the current measured average (see above).

@RISK™ software (Excel™ add on) was used to fit a theoretical probability distribution to the adopted settleability data in Table 4, using an "expert function" (known as "PertAlt"). The resultant distributions are shown plotted in Figure 1 (for SSVI) and Figure 2 (for DSVI).

Based on these distributions, and the settleability parameters relationships described by Ekama et al. (1997), it was possible to derive functions to predict the flux theory Vesilind settleability parameters (Vo and n) on a probabilistic basis. For modelling purposes, equal weighting was placed on the Vesilind parameter values predicted from SSVI and DSVI relationships, by taking the average of the two. SSVI and DSVI were assumed to be have correlated probability distributions with a correlation coefficient of 0.9.

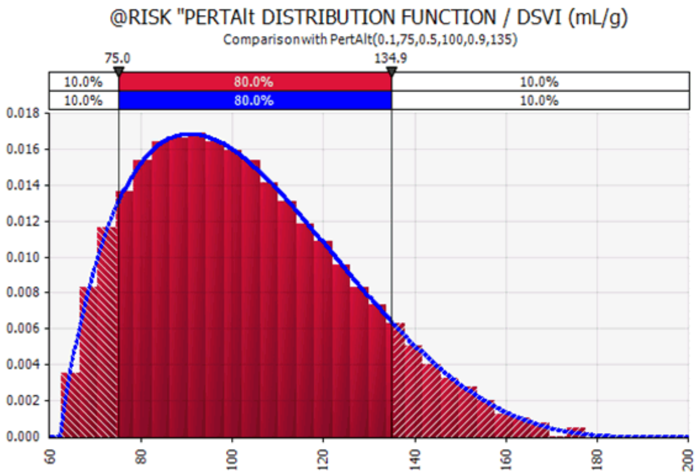
[4] Ekama and Marais (1986), cited in Ekama et al. (1997)

Memorandum



Statistics	@RISK "PERT..	PertAlt(0.1,50..
5%	47.175	47.188
10%	50.002	50.025
15%	52.396	52.413
20%	54.555	54.592
25%	56.643	56.659
30%	58.666	58.666
35%	60.616	60.649
40%	62.600	62.634
45%	64.605	64.643
50%	66.667	66.700
55%	68.793	68.827
60%	71.038	71.052
65%	73.362	73.409
70%	75.939	75.943
75%	78.663	78.721
80%	81.828	81.845
85%	85.454	85.495
90%	90.012	90.045
95%	96.549	96.551
99%	107.165	107.367

Figure 1 Distribution function for SSVI derived from adopted values in Table 4



Statistics	@RISK "PERT..	PertAlt(0.1,75..
5%	70.66	70.75
10%	74.97	75.00
15%	78.53	78.58
20%	81.80	81.85
25%	84.89	84.95
30%	87.90	87.96
35%	90.89	90.93
40%	93.89	93.90
45%	96.89	96.92
50%	99.96	100.00
55%	103.16	103.19
60%	106.52	106.52
65%	110.02	110.06
70%	113.84	113.86
75%	117.97	118.02
80%	122.66	122.71
85%	128.12	128.18
90%	134.93	135.00
95%	144.72	144.75
99%	160.55	160.97

Figure 2 Distribution function for DSVI derived from adopted values in Table 4

10

Memorandum

2.1.2 MLSS

Data

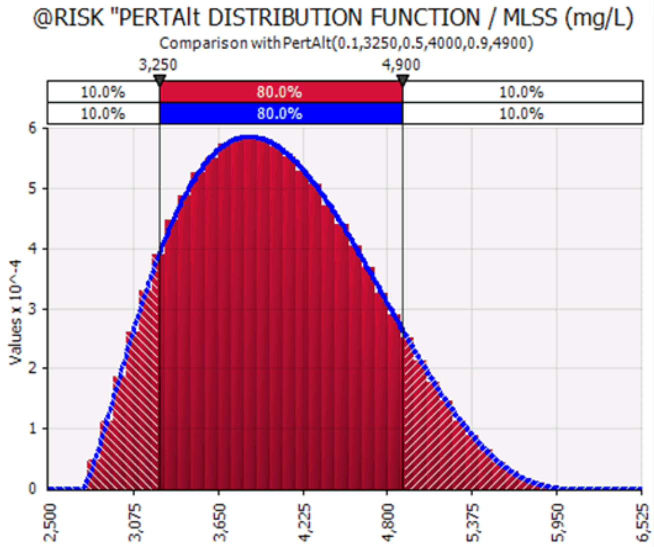
Mixed liquor suspended solids varies with plant loading and sludge age (wasting). The design 90%ile MLSS for BVSTP was 4,900 mg/L (a relatively high value). More typical design values for activated sludge systems are in the range 3,000 to 4,500 mg/L, such as at (W)BSTP (refer to Table 1). Limited recent data (see Table 3) indicated operating MLSS around 5,000 to 5,800 mg/L, which is high and likely to represent close to the maximum sustainable MLSS concentration at this plant (around 6,000 mg/L). The plant sludge age during recent operation was likely significantly longer than the design value (due to relatively low wasting rates), and is expected to be limiting clarifier capacity.

Distribution function

For the purposes of establishing a probability distribution function for MLSS in this study, the assumptions in Table 5 were made. The associated log-normal distribution derived using @RISK (refer to Section 2.1.1 above) is given in Figure 3.

Table 5 MLSS concentrations adopted for this study

Percentile	MLSS (mg/L)
10th percentile	3,250
50th percentile	4,000
90th percentile	4,900



Statistics

	@RISK "PERT..	PertAlt(0.1,32..
5%	3,102.27	3,102.85
10%	3,249.62	3,250.00
15%	3,366.97	3,367.20
20%	3,470.15	3,470.31
25%	3,565.14	3,565.38
30%	3,655.49	3,655.57
35%	3,742.77	3,742.86
40%	3,828.42	3,828.63
45%	3,913.93	3,914.01
50%	3,999.94	4,000.00
55%	4,087.30	4,087.57
60%	4,177.54	4,177.78
65%	4,271.61	4,271.91
70%	4,371.49	4,371.59
75%	4,478.70	4,479.09
80%	4,597.79	4,597.92
85%	4,733.89	4,734.12
90%	4,899.99	4,900.00
95%	5,129.01	5,129.92
99%	5,490.63	5,492.45

Figure 3 Distribution function derived for MLSS from adopted values in Table 5

Memorandum

2.1.3 Flow

Flow dataset and related assumptions

- Plant inflow was modelled at a short time step of 15 minutes, based on actual flow recorded data for two periods:
  - 12 March to 5 April 2017
  - 10 to 15 June 2017
- Both of the abovementioned periods included at least one significant wet weather event[5] and considered to reflect the peak pumping capacity from the catchments of Mullumbimby, Brunswick Heads and Ocean Shores. In doing so, the following assumptions were made in relation to flow:
- Instantaneous flows at a time step shorter than 15 min were not modelled. This was considered a reasonable assumption, considering that the flow splitter (to the storm dam, see below) would divert instantaneous peak flows away from the treatment process and clarifiers. Furthermore, some degree of flow attenuation is expected within the oxidation ditch and clarifiers, due to small changes in volume of these reactors within the design hydraulic grade line of the plant.
- The proposed new storm dam will be in place, with associated flow diversion and (pumped) return flow facilities.
- Plant inflow (from the combined catchments) will be split at the proposed new flow splitter and the instantaneous maximum flow allowed to pass through the treatment process to the clarifiers will be capped (nominally up to 6x original design ADWF). The capped flow was a model input variable (a key uncertainty – see below). The remainder of the flow will be diverted to the storm dam.
- The concept of maximum or 'sustained peak' flow, as modelled here, is framed within the context of the above definitions i.e. the wet weather event adopted datasets and flow diversion set points ('capped' flow to treatment process - see above). Since total inflow in the adopted datasets exceeded the capped flow to the process for extended periods (in the order of hours to one or more consecutive days), the treatment process and clarifiers would receive maximum (capped) sustained flows for that duration.
- Return flows from the storm dam were not modelled. This was considered to be a reasonable assumption since the proposed return rate from the storm dam was <1x design ADWF (existing plant) and was proposed to only operate when flows to the process were <2x design ADWF (existing). Hence, for the times when the storm dam return was operating, the process would receive <3x design ADWF, which is well below the nominal design capacity of the clarifiers (5.7x design ADWF, existing). Therefore, inclusion of the return pumping flow rates was not expected to materially change the model predictions in this study.

The flow pattern for the March-April 2017 dataset adopted in shown in Figure 4. A peak flow rate in the range 200 to 495 L/s (4.5 to 11.3 times the design ADWF of the existing plant i.e. 3.8 ML/d or 44 L/s) was sustained for 1.38 days (33 hours) at around 18-19 days in the timeline plotted in Figure 4.

The flow pattern for the June 2017 dataset adopted in shown in Figure 5. A peak flow rate in the range 243 to 415 L/s (or 5.5 to 9.43 times the design ADWF of the existing plant – see above) was sustained for 0.41 days (9.8 hours) at around 2.5 to 3 days in the timeline plotted in Figure 5.

[5] Refer to GHD Memo 4131098-REP-2 (updated 8 Nov 2017) for a discussion of this data. The same datasets were used to assess the risks relating to storm dam capacity for this study.

12

Memorandum

The projected combined ADWF for the combined catchments (after transfer from Ocean Shores) was projected[6] (GHD, 2016) to be 4.34 ML/d (or 50 L/s) by the year 2035/36. Therefore, the peak flow rate (243 to 415 L/s) in the June 2017 dataset represents a range of approximately 4.9 to 8.3 times ADWF. GHD confirmed[7] that BSC is not planning to increase the absolute peak pumping capacity from the respective catchments within the foreseeable future (i.e. the planning horizon under consideration here, namely not before 2036/36). On this basis, the June 2017 event was considered to reasonably represent a 'typical' peak wet weather event within the planning horizon for purposes of risk assessment within the framework of this study.

Similarly, the March-April 2017 event covered a longer period of 24 days and was considered representative of a series of wet weather events, culminating in a relatively large event that included short periods of instantaneous at maximum peak pumping capacity from the catchment[8].

Based on information reviewed in the previous study (GHD. 2016)[9], the nominal instantaneous peak pumping capacities given in Table 6 for the respective catchments and combined total were adopted and used as guide in this study.

Table 6 Nominal peak pumping capacities of pump stations connected to BVSTP (after Ocean Shores transfer) adopted for this study

Pump Station	Flow rate (nominal design capacity)
Mullumbimby (SPS 4000):	156 L/s
Brunswick Heads (SPS 2000):	158 L/s
Ocean Shores Kiah Close (SPS 5009):	170 L/s
Ocean Shores Rajah Rd (SPS 5004):	62 L/s
Total (Combined catchments):	**546 L/s**

Flow distribution fit

For the June 2017 dataset, @RISK™ software (Excel™ add on) was used to fit a log-normal distribution to the flow dataset depicted in Figure 5. The resultant fit is shown in Figure 6. The fit is reasonable and suggests a model 99th percentile flow rate of 548 L/s, which is very close to the nominal peak pumping capacity of the combined catchments (see above).

The log-normal distribution function defining the fitted curve in Figure 6 was therefore used to predict flow rate in the clarifier flux model applied here.

The log-normal distribution function applied might predict peak flow rates greater than nominally 550 L/s (see above existing total peak pumping capacity 546 L/s). However the probability of flow rates (>550 L/s) is very low (<1% during a major wet weather event typified by the June 2017 event used here; and possibly <0.1% overall). Peak flow predictions >550 L/s from the probability function (see above) were

[6] Refer to OS-BVSTP Transfer Feasibility Study (GHD, 2016), Figure 3, based on adopted values from population projections according to BSC Business Plan (2016).

[7] Record of phone discussion held between D Baulch (BSC) and D de Haas (GHD), email dated 19/09/2017.

[8] Instantaneous pumping rate not fully reflected in the flow data plotted in Figure 4 (flows aggregated to 15 min data intervals).

[9] Refer to OS-BVSTP Transfer Feasibility Study (GHD, 2016), Section 2.3.2

ignored for the purposes of this assessment (i.e. considered to be beyond the existing peak pumping capacity of the combined catchments).

For the March-April dataset, flow was modelled in @RISK™ by resampling (in time series order) the original dataset depicted in Figure 4. An idealised distribution function fit was therefore not necessary. Zero flow values in the dataset were adopted as 1 L/s, to avoid maths errors in the model calculations. The probability distribution for this dataset is shown in Figure 7.

2.2 Other model assumptions

Other important model inputs (not modelled as distributed uncertainties) and related assumptions were as follows:

- Existing plant design ADWF = 3.8 ML/d (44 L/s);
- Both clarifiers on line during peak wet weather events;
- Typical RAS ratio 1:1, with respect to inflow treated through clarifiers i.e. clarifier overflow rate (from Fulton Hogan Design Report);
- RAS pumps existing capacity maximum with both clarifiers in operation 150 L/s (from Fulton Hogan Design Report). This corresponds to a 1:1 RAS recycle ratio (flow paced) up to a peak weather flow of 3.4 times design ADWF (notionally 'full treatment' up to this flow rate). At higher peak flows, the RAS pumps will operate at maximum and the RAS recycle ratio (relative to plant flow treated) will decrease proportionally;
- RAS pumps existing capacity maximum with one clarifier in operation 80 L/s (assumption);
- RAS pumps existing capacity minimum with both clarifiers in operation 20 L/s (from Fulton Hogan Design Report); and
- RAS pumps existing capacity minimum with one clarifier in operation 15 L/s (assumption).

Memorandum

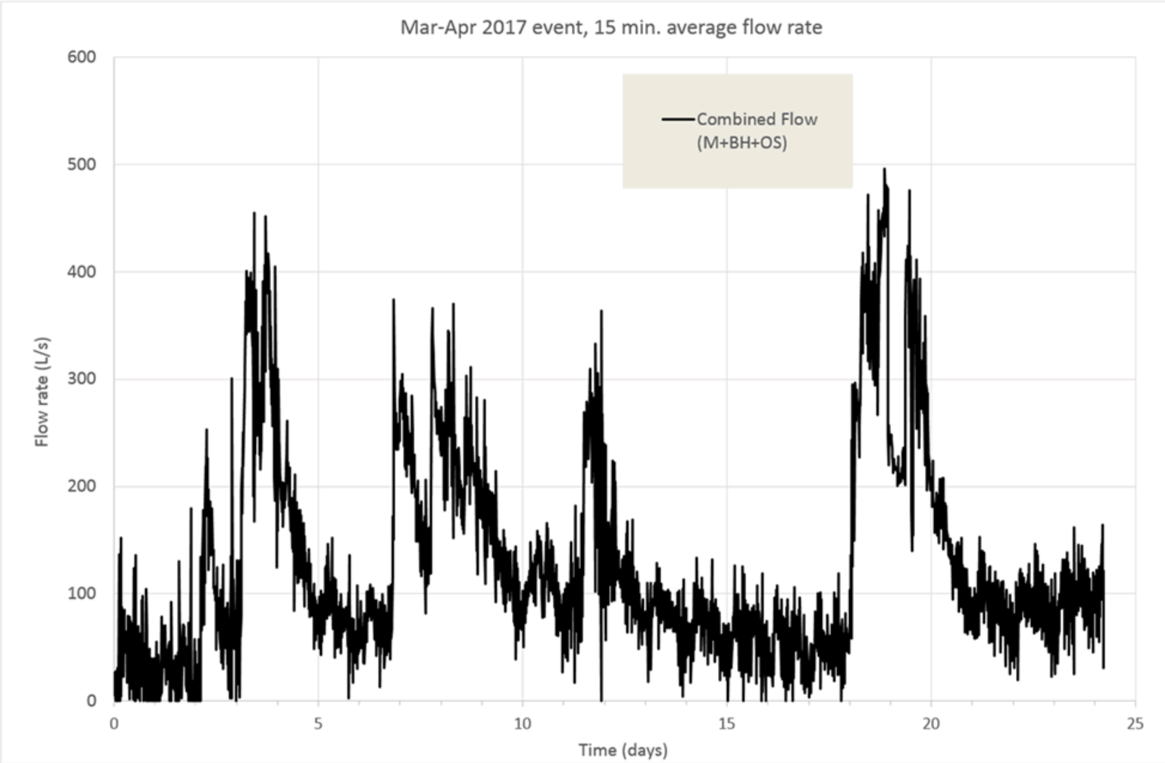


Figure 4 **March-April 2017 event *combined* catchments flow rate adopted (15 min. ave. calculated from SCADA data). Time 0 (days) is 12/3/2017 03:34:53.**

15

Memorandum

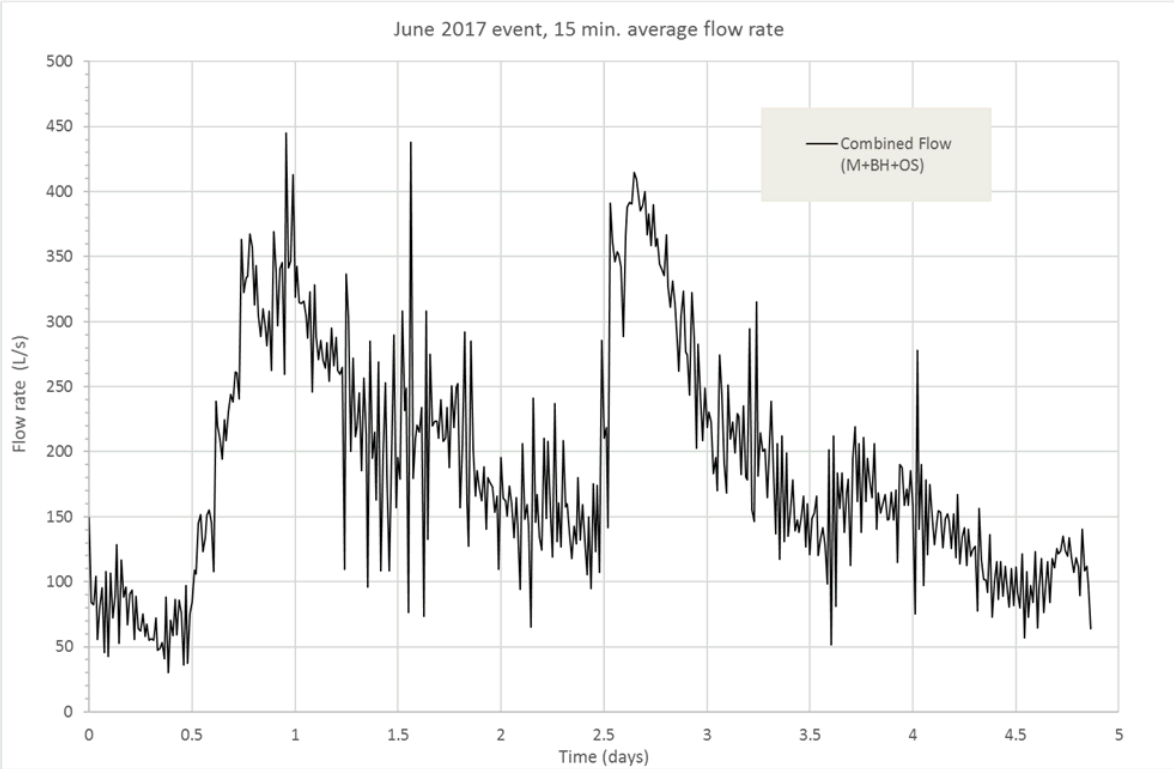
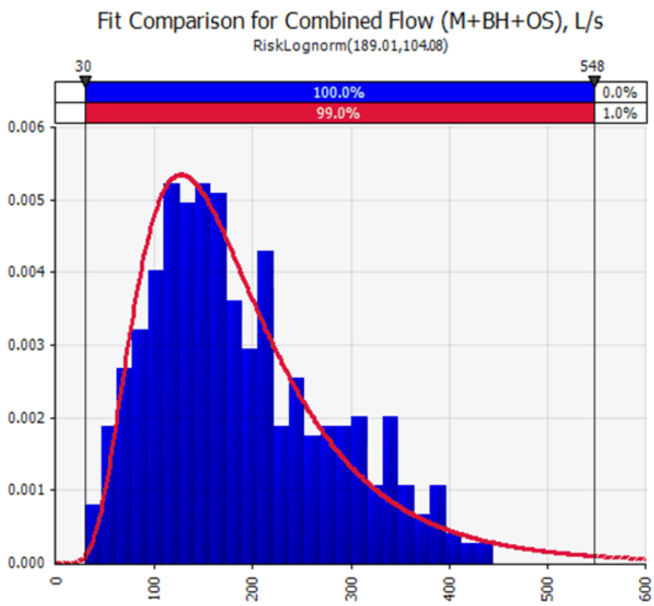


Figure 5 June 2017 event *combined* catchments flow rate adopted (15 min. average calculated from SCADA data). Time 0 (days) is 10/6/2017 15:03:19

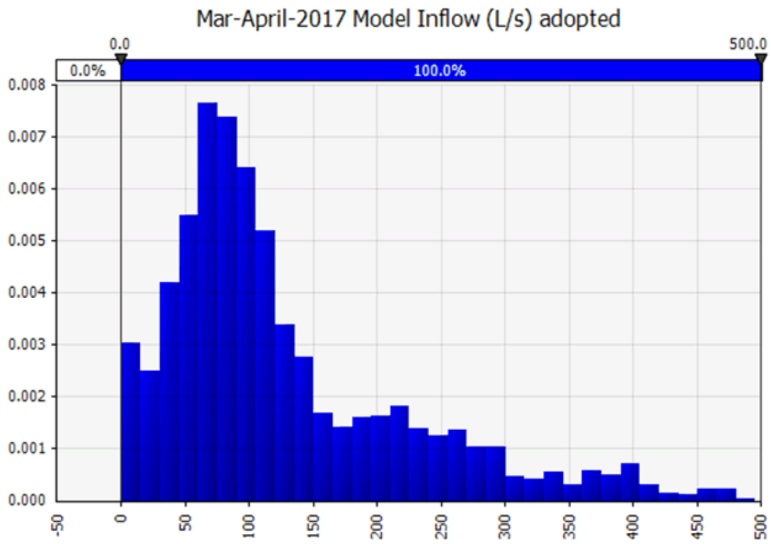
16

Memorandum



Statistics	Input	Lognorm
Right P	100.0%	99.0%
Dif. X	518.17	518.17
Dif. P	100.0%	99.0%
1%	42.72	50.01
5%	64.45	71.02
10%	83.97	85.61
15%	96.43	97.13
20%	109.73	107.37
25%	119.18	117.01
30%	128.63	126.41
35%	140.34	135.79
40%	149.11	145.33
45%	158.61	155.20
50%	166.06	165.57
55%	179.46	176.63
60%	195.20	188.62
65%	208.76	201.88
70%	220.41	216.86
75%	243.48	234.27
80%	269.99	255.32
85%	292.43	282.24
90%	323.42	320.19
95%	358.25	386.01
99%	409.66	548.17

Figure 6 Lognormal distribution fit for 10-15 June 2017 (wet weather event) flow dataset used for this study



Statistics	Flow By-pass Co..
1%	1.00
5%	17.15
10%	34.45
15%	46.23
20%	55.55
25%	63.82
30%	70.01
35%	76.39
40%	82.91
45%	89.71
50%	98.10
55%	105.41
60%	113.42
65%	125.31
70%	141.58
75%	168.97
80%	202.18
85%	232.78
90%	270.16
95%	341.34
99%	423.57

Figure 7 Probability distribution for 12 March to 5 April 2017 (successive wet weather events) flow dataset used for this study

17

Memorandum

3 Results

3.1 By-pass condition 255 L/s (5.8x existing design ADWF)

According to the Design Report (Fulton Hogan, 2010), the plant clarifiers were designed to handle a peak sustained flow rate of 255 L/s or 5.8 x design ADWF at 90 percentile SSVI (60 mL/g) and peak MLSS concentration (4.9 g/L) i.e. without process failure [10].

The existing plant has no stormdam and no bypass for diversion of high flows. The starting assumption was that the proposed bypass condition for diversion of high flows to a stormdam could be set at 255 L/s, in accordance with the existing plant design.

The model predictions clarifier capacity for this scenario are given as follows:

- March-April 2017 event in Figure 8; and
- June 2017 event in Figure 9.

The results indicate that there is a probability of approximately a 7 to 12% (1:14 to 1:8) that the clarifiers will have a *deficit* in process capacity during a typical peak wet weather events. The process is therefore predicted to have a 7% to 12% change of nominal 'failure' during a peak wet weather event. Nominal 'failure' here is defined as potential gross solids loss over the clarifier weirs during peak wet weather flows i.e. operating beyond the 'safe' condition defined for maximum permissible clarifier overflow rate from modified flux theory. This illustrates the process risk related to the clarifier design being less conservative, compared with the assumptions for this study for likely actual performance (e.g. around settleability, see Section 2.1).

To mitigate this risk, more flow will need to be diverted (away from the process) to the storm dam i.e. the by-pass condition flow set point will need to be lower.

3.2 By-pass condition 224 L/s (5.1x existing design ADWF)

3.2.1 Maximum RAS rate 150 L/s

Water balance modelling, as part of this study [11] suggested that a storm dam of 20 ML capacity would be sufficient if the process (i.e. limited by clarifier capacity) could handle a sustained flow rate of 224 L/s. Setting the by-pass condition to 224 L/s gave the model predictions in Figure 11. The results indicate that there is a probability of approximately 8.3% (1 in 12) that the clarifiers will have a *deficit* in process capacity during a typical peak wet weather event, typified here by the 10-15 June 2017 event. This represents a small improvement compared with 11.6% in the previous case (Section 3.1) – compare Figure 11 with Figure 9. Similarly, for the March-April 2017 dataset, the probability of clarifier 'failure' (capacity) deficit decreased to 5% (from 7%) – compare Figure 10 and Figure 8.

To further mitigate the risk of clarifier 'failure', two options were considered:

[10] Hydraulic capacity was provided for higher maximum instantaneous flows (up to 314 L/s or 7.1 x ADWF), but not on a sustained basis (i.e. MLSS would need to be 'stored' in the process e.g. by turning off aeration in the oxidation ditch to allow partial settlement as a mitigation measure to allow sustained high flows >255 L/s to pass through the clarifiers without risking the gross solids loss i.e. sludge blanket rising to the surface of the clarifiers).

[11] Refer to GHD Memo 4131098-REP-2 (updated 8 Nov 2017) for a discussion of this data.

Memorandum

- Increasing the maximum RAS rate in attempt to maximise available clarifier capacity, which will require an upgrade of the existing RAS pumps (refer to Section 3.2.2) ; or
- By-passing more flow (i.e. lower bypass condition set point), which will increase storm dam capacity requirement (refer to Section 3.3).

3.2.2 Maximum RAS rate 200 L/s

The potential benefit of an increased RAS rate was investigated for a scenario in which the maximum RAS rate (with both clarifiers in operation) was hypothetically increased from 150 L/s (existing capacity) to 200 L/s. This represents a 33% increase and broadly considered to be the largest feasible [12] increase by changing the pumps but retaining the existing RAS system (pipework, valves etc.).

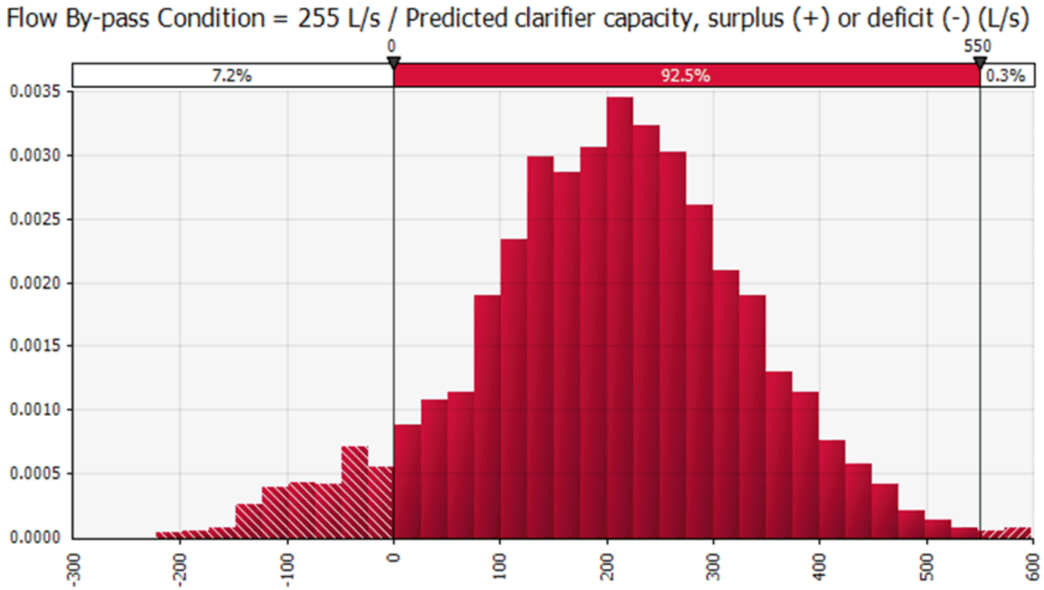
For the same by-pass condition (224 L/s), the model results (graphs not shown) indicated a small decrease in probability of clarifier 'failure' (capacity deficit), namely:

- 3.7% for the March-April 2017 dataset at a total max. RAS rate of 200 L/s, as compared with 5% at max. RAS 150 L/s (see Section 3.2.1); or
- 6.5% for the June 2017 dataset at a total max. RAS rate of 200 L/s, as compared with 8.3% at max. RAS 150 L/s (see Section 3.2.1).

[12] Indicative only; detailed investigation of the RAS system (outside the scope of this study) will be required before possible implementation of this option.

19

Memorandum

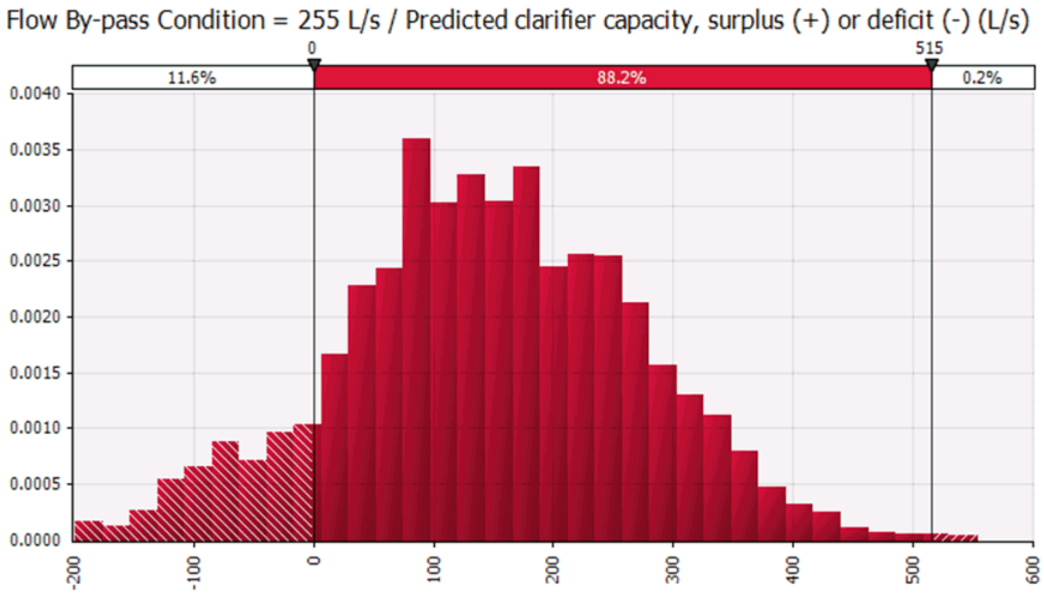


Statistics

	Flow By-pass Conditio..
1%	-124.48
5%	-30.77
10%	32.16
15%	76.53
20%	102.50
25%	124.45
30%	141.13
35%	160.47
40%	175.38
45%	192.07
50%	207.35
55%	221.19
60%	236.82
65%	252.88
70%	268.12
75%	286.10
80%	307.20
85%	332.32
90%	365.03
95%	406.06
99%	483.19

Figure 8 **Combined probability modified flux model predictions for clarifier capacity (surplus or deficit, L/s) during a successive 'typical peak' wet weather events (based on March-April 2017 events), assuming 255 L/s by-pass condition (to storm dam)**

Memorandum



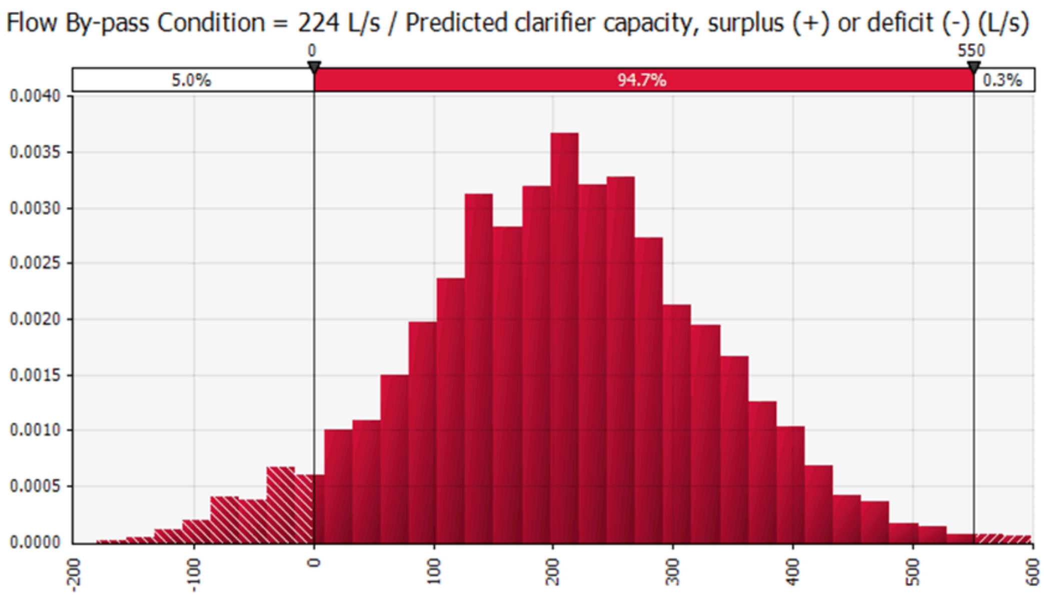
Statistics	Flow By-pass Conditio..
1%	-136.23
5%	-72.71
10%	-15.30
15%	24.35
20%	47.52
25%	66.46
30%	84.22
35%	97.97
40%	114.74
45%	131.34
50%	145.75
55%	162.26
60%	177.16
65%	192.79
70%	213.74
75%	233.38
80%	253.89
85%	276.50
90%	305.62
95%	348.51
99%	431.28

Figure 9 Combined probability modified flux model predictions for clarifier capacity (surplus or deficit, L/s) during a 'typical peak' wet weather event (based on 10-15 June 2017 event), assuming 255 L/s by-pass condition (to storm dam)

21

GHD

Memorandum

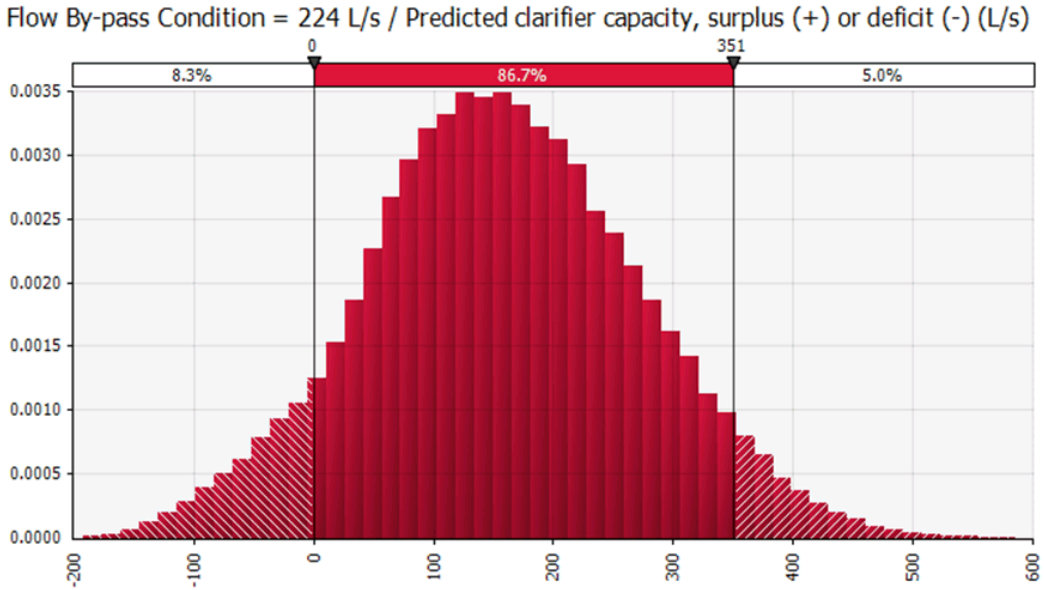


Statistics	Flow By-pass Conditio..
1%	-79.75
5%	-3.25
10%	49.40
15%	85.70
20%	109.22
25%	129.20
30%	145.13
35%	163.74
40%	179.36
45%	195.36
50%	209.60
55%	223.04
60%	237.83
65%	253.59
70%	268.85
75%	286.52
80%	307.40
85%	332.96
90%	365.03
95%	406.06
99%	483.19

Figure 10 **Combined probability modified flux model predictions for clarifier capacity (surplus or deficit, L/s) during a successive 'typical peak' wet weather events (based on March-April 2017 events), assuming 224 L/s by-pass condition.**

22

Memorandum



Statistics	Flow By-pass Conditio..
1%	-101.07
5%	-32.10
10%	12.73
15%	41.72
20%	63.12
25%	80.75
30%	96.85
35%	112.13
40%	126.87
45%	141.12
50%	155.47
55%	170.03
60%	184.94
65%	200.48
70%	216.72
75%	234.56
80%	255.19
85%	278.36
90%	307.71
95%	350.84
99%	425.29

Figure 11 Combined probability modified flux model predictions for clarifier capacity (surplus or deficit, L/s) during a ‘typical peak’ wet weather event (based on 10-15 June 2017 event), assuming 224 L/s by-pass condition

23

Memorandum

3.3 By-pass condition 176 L/s (4x existing design ADWF)

Water balance modelling, as part of this study suggested that a storm dam of 30 ML capacity would be sufficient if the process (i.e. limited by clarifier capacity) could handle a sustained flow rate of only 176 L/s. Setting the by-pass condition to 176 L/s gave the model predictions in Figure 12 and Figure 13.

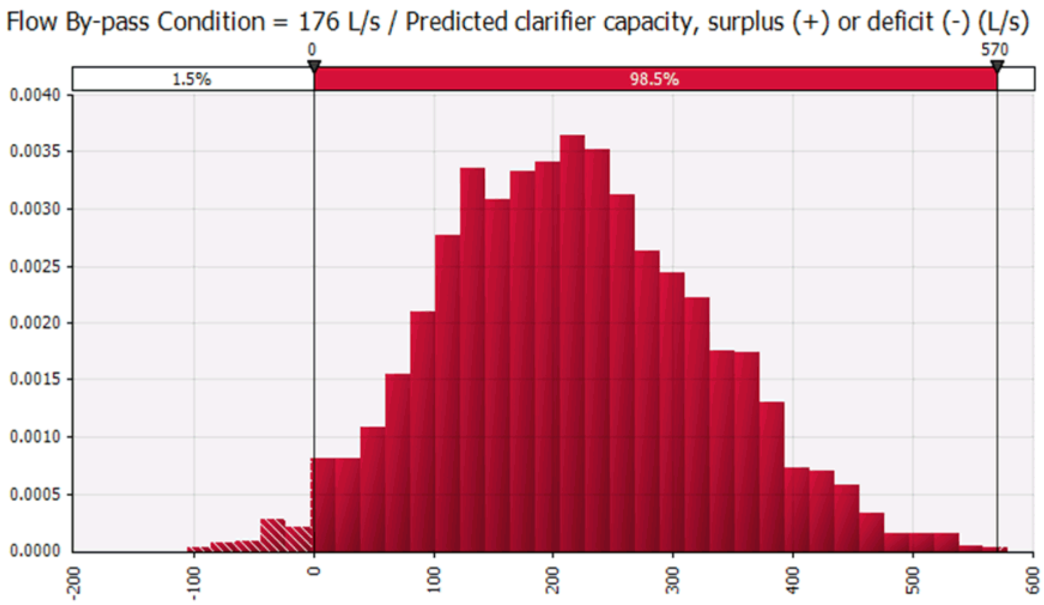
Assuming the existing total max. RAS rate (150 L/s), the results indicated probabilities of clarifier 'failure' (capacity deficit), as follows:

- 1.5% for the March-April 2017 dataset (Figure 12); or
- 3.4% for the June 2017 dataset (Figure 13)

Increasing the total max. RAS rate hypothetically to 200 L/s, gave marginally decreased predicted probabilities of clarifier 'failure' (capacity deficit), as follows (not graphed):

- 1.2% for the March-April 2017 dataset; or
- 2.8% for the June 2017 dataset

Memorandum



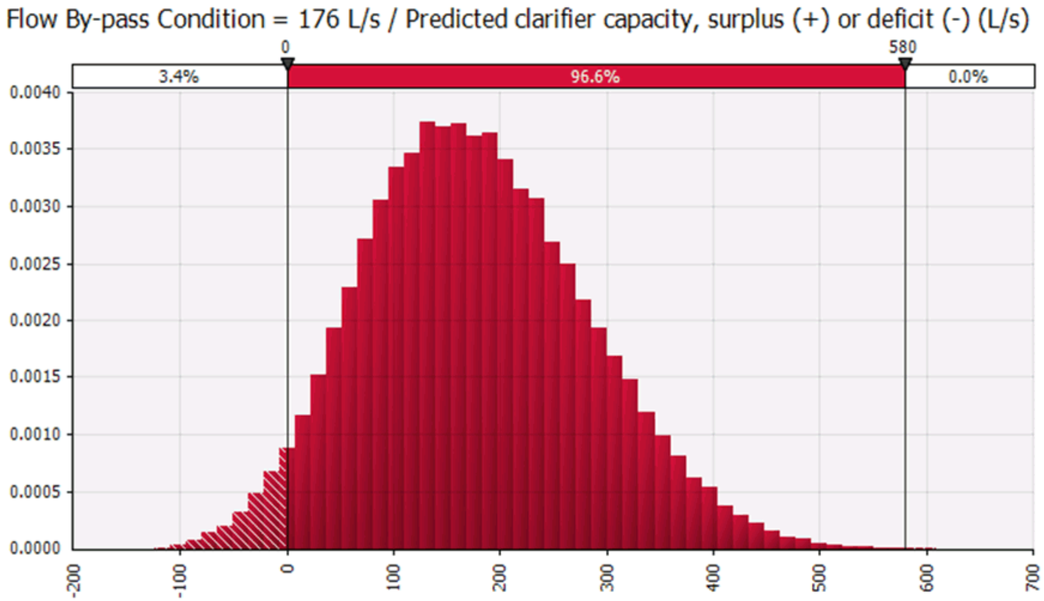
Statistics	Flow By-pass..
1%	-14.63
5%	43.87
10%	79.75
15%	103.93
20%	121.96
25%	136.45
30%	152.60
35%	168.03
40%	183.67
45%	198.56
50%	212.47
55%	225.75
60%	239.43
65%	255.53
70%	272.42
75%	292.28
80%	312.48
85%	335.34
90%	364.07
95%	401.65
99%	476.46

Figure 12 Combined probability modified flux model predictions for clarifier capacity (surplus or deficit, L/s) during a successive 'typical peak' wet weather events (based on March-April 2017 events), assuming 176 L/s by-pass condition.

25

GHD

Memorandum



Statistics

	Flow By-pass..
1%	-38.69
5%	16.44
10%	47.70
15%	69.79
20%	87.24
25%	102.77
30%	117.50
35%	131.42
40%	144.93
45%	158.34
50%	171.74
55%	185.67
60%	199.45
65%	214.17
70%	230.08
75%	246.89
80%	266.17
85%	288.57
90%	316.67
95%	357.35
99%	430.80

Figure 13 Combined probability modified flux model predictions for clarifier capacity (surplus or deficit, L/s) during a 'typical peak' wet weather event (based on 10-15 June 2017 event), assuming 176 L/s by-pass condition

26

GHD

Memorandum

4 Summary

This study has highlighted that the existing BVSTP clarifiers (2 no.) have a design capacity that is somewhat less conservative than other designs (e.g. Byron STP). There is currently a significant risk (around a 1:10 probability) that the process capacity of the clarifiers will be exceeded during major wet weather events, even if the flows to the clarifiers (both on line) is limited to the design flow rate for full treatment nominated by the designers (i.e. 255 L/s).

Similarly, in future, there is a risk that the process capacity of the existing clarifiers and RAS pumps might be exceeded during wet weather events, even if surplus flows from the combined catchments of Mullumbimby, Brunswick Heads and Ocean Shores are diverted to the proposed storm dam. Mitigating that risk will involve a trade-off against increased storm dam holding capacity (volume) and hence capital cost. Although it might not be feasible to completely eliminate the risk of failure, a modest decrease in risk of clarifier failure can be achieved by increasing the capacity of the RAS pumps. Since upgrading the RAS pumps will involve less capital cost than increasing the storm dam capacity, this aspect merits be further investigation to assess its feasibility and benefits in more detail.

The results of this study are summarised in Table 7.

Table 7 Summary results of BVSTP existing clarifier process capacity risk assessment

Storm dam Volume recommended[13] ML	Risk condition By-pass flow set point[14] L/s *[x ADWF]*[15]	Risk condition Max. Total RAS rate (2 no. clarifiers in operation)	Probability of clarifier process failure[16] during wet weather event	
			March-April 2017 event	June 2017 event
16.7	255 *[5.8x]*	150	7.2%	11.6%
20	224 *[5.1x]*	150	5.0%	8.3%
		200	3.7%	6.5%
30	176 *[4x]*	150	1.5%	3.4%
		200	1.2%	2.8%

[13] Refer to GHD Memo 4131098-REP-2 (updated 8 Nov 2017), including allowance for rainfall capture and/or contingencies

[14] Plant inflows greater than this setpoint will be diverted to the proposed storm dam; flows less than this setpoint directed through the treatment process including clarifiers.

[15] ADWF for existing design capacity (3.8 ML/d or 44 L/s)

[16] Nominal failure defined as capacity deficit predicted from modified flux theory model.

27

Memorandum

5 Recommendations

The following recommendations are made from this study:

1. This study identified the risk of nominal process failure of the BVSTP clarifiers at a probability of approximately 5 to 8% (1:20 to 1:12) during major wet weather events, assuming a storm dam of 20 ML capacity is built and flows >224 L/s are diverted from the mainstream treatment process to the storm dam. If that risk if considered unacceptable to BSC, then additional mitigation measures will be required.
2. Alternative mitigation measures could involve one or more of the following:
   - Lower flow setpoint for diversion (by-passing) to the storm dam and an increase in storm dam capacity (volume) – subject to capital cost constraints;
   - Increased in capacity of the RAS system (upgrade of RAS pumps) – subject to existing pipework constraints and to be confirmed by further investigation; and/ or
   - Other process control measures, also requiring further investigation (e.g. installation of more in-plant instrumentation and control in the form of sludge blanket detectors in the clarifiers, coupled with automation to turn off aeration in the oxidation ditch to allow partial solids settlement and 'storage' within the bioreactor for a limited time, in the event of a high clarifier sludge blanket level).

6 References

Ekama G.A. et al. (1997) Secondary Settling Tanks: Theory, Modelling, Design and Operation. Scientific and Technical Reports No. 6. IWA Publishing, London. https://www.iwapublishing.com/sites/default/files/ebooks/9781780408996.pdf

Fulton Hogan/ Cardno (2010). *Brunswick Valley Sewage Treatment Plant (Contract No. 2008-00001) – Design Report*. Report compiled for Byron Shire by Fulton Hogan/ Cardno/ GHD (Aug. 2010, Version 9).

Regards

David de Haas
Principal Professional, Wastewater Treatment

Appendix C – Oxidation Ditch and Process modelling

Memorandum

30 November 2017

To	Byron Shire Council		
Copy to	Dean Baulch		
From	David de Haas	Tel	(07) 3316 3715
Subject	BVSTP Process Risk Assessment - Oxidation Ditch & Process Modelling	Job no.	41/31098

1 Background

1.1 Introduction

As part of the process risk assessment relating to the transfer of wastewater loads from Ocean Shores to Brunswick Valley STP (BVSTP), the capacity of the oxidation ditch and related bioreactors at BVSTP was assessed in more detail by means of dynamic modelling (kinetic simulation). This memo summarises approach and key the outcomes of that modelling.

1.2 Objectives

The main objectives of dynamic simulations were to check the capacities of the following existing reactors or items of equipment at BVSTP using projected loadings in ca. 2035/36 for the combined catchments of Mullumbimby, Brunswick Heads and Ocean Shores:

- Oxidation ditch aeration system (diffusers and blowers);
- Oxidation ditch solids inventory (MLSS);
- Aerobic digester aeration system (diffusers and blowers);
- Aerobic digester solids inventory (MLSS), volatile solids (VSS) destruction;
- Aerobic digester operation (DO, mode, decanting of supernatant etc.); and
- Alum dosing requirements.

The projected loadings at a nominal date of 2035/36 was chosen (refer to OS-BVSTP Transfer Feasibility Study GHD, 2016 – Section 8.3 and Section 13) as a 'worst case', on the basis that the proposed process capacity augmentation could be deferred to no later than this date. The previous study (GHD, 2016) identified that by 2035/36 the existing plant would be nominally operating at approximately 114% of this design load and higher loadings would likely be infeasible. For the augmented process capacity to be operational no later than 2035/36 (as proposed), planning, design and implementation would need to commence earlier (indicatively 2032/33).

2 Methodology

The simulation package BioWin™ version 5.2.0.1157 was used.

1

GHD

Memorandum

2.1 Model inputs

2.1.1 Flow

The projected average dry weather flow (ADWF) for the year 2035/36 was adopted at 4.34 ML/d, based on the OS-BVSTP Transfer Study[1] (GHD, 2016).

A diurnal flow pattern (Figure 1) normalised to average, was adopted. It was based on SCADA data supplied by BSC for:

- BVSTP in early March 2017 (during dry weather); and
- OSSTP from a previous planning study (GHD, 2014) for the period Jan-Dec-2013, dry weather periods only.

The combined diurnal flow pattern was flow-weighted according to the average flow from the respective catchments (BVSTP for Mullumbimby and Brunswick Heads; SPS 5009 and 5004 from Ocean Shores) – refer to Figure 1.

2.1.2 Raw wastewater characterisation

Original design

The design 50%ile concentrations taken from the designers report (Fulton Hogan, 2010) were adopted. Where values were not stated (e.g. for TSS, VSS or ISS), reasonable assumptions were made, based on our experience for typical wastewaters in Australia. Similarly, typical diurnal concentration patterns (normalised to average) were adopted, based on our experience for similar-sized plants in Australia. The adopted curves are shown in Figure 2. The diurnal concentration model inputs were derived by calculation from the 50%ile concentrations for the respective parameters and the normalised diurnal profiles shown in Figure 2. Suspended solids were assumed to be proportional to COD. Where no data was available, constant values were assumed (pH 7.2, Alkalinity 4.6 mmol/L or 230 mg/L CaCO3; Nitrate 0.02 mgN/L and zero DO). Other influent concentration parameters (Ca and Mg) were set at model defaults.

The diurnal peak mass (theoretical total oxygen demand) load factor, relative to average, from the data presented in Figure 2) was 2.1, adopted for simulation purposes. This compares with the design assumption of 2.5. That is, the simulations are within the design envelope for diurnal mass loading peaking factor relative to average, although the simulated future average loads are expected to exceed design values, in mass terms.

Other key model assumptions relating to raw wastewater characterisation (e.g. COD fractions) were taken from the design report (refer to Table 1).

Additional data

During the course of this study, GHD requested additional sampling data for characterisation of BVSTP. For a number of logistical reasons, the additional sampling required was delayed. Only one complete set

[1] Refer to Figure 3 in Section 2.2 of the Report no. 41/28941/46789 (GHD, 2016) to BSC, dated 23 November 2016 (rev0). Additional allowance for I/I not included on the basis that the dry weather flow wastewater concentrations (design values) adopted were assumed to represent conditions when I/I was minimal. i.e. following a prolonged dry spell.

2

Memorandum

of results for 24-hour time-series samples (taken on 31 October 2017) were provided by the time of writing this report (November 2017).

Figure 3 shows a comparison between adopted values for COD, TKN and TP concentrations with those from the recent dataset (31 October 2017). The average COD measured in the recent dataset was 648 mg/L, which is very close to the design value (540 mg/L). However the diurnal peak COD concentration measured (1390 mg/L) was higher than the adopted value. The average TKN measured in the recent dataset was 62 mgN/L, which is significantly higher than the design value (54 mgN/L). The diurnal peak TKN concentration was also higher (88 vs. 74 mgN/L).

However, Figure 3 also shows that the diurnal peak concentrations for both COD and TKN occurred later in the day for the recent dataset than assumed in the adopted dataset. Therefore the diurnal mass load peaks would be attenuated to some extent since the flow is a little lower later the day at the time of the peak concentrations. The mass load peaking factor for COD was only slightly higher in the recent dataset (2.2 vs. 2.0 in the adopted dataset) but the theoretical oxygen demand mass load peaking factor was slightly lower (1.9 vs. 2.1). The timing and magnitude of the peak loads is expected to change when the Ocean Shores loads are transferred to BVSTP and will need to be checked using more complete datasets prior to project implementation (see below).

The recent dataset also showed somewhat higher influent suspended solids and lower BOD, which will influent process model predictions (less favourable in terms of predicted process capacity).

Overall, the impact of the recent wastewater dataset was expected to produce a mixed set of predictions, relative to the adopted dataset. The process simulations were repeated using a rough re-estimation of wastewater characteristics, based on the one available set of recent samples (31 October 2017) taken at the BVSTP inlet (i.e. Mullumbimby and Brunswick Heads catchments only). Only key differences in the simulations from this recent dataset (compared with the assumptions made based on the original design values, see above) were documented for the purpose of this study.

Further wastewater characterisation (including datasets from both the existing BVSTP and Ocean Shores catchments) and process simulations are recommended, prior to detailed design and/or project implementation, in order to confirm the results of this study.

Memorandum

Table 1 Adopted wastewater characteristics (from Fulton-Hogan, 2010)

Parameter	Value		
50%ile Loads:	**Load**		**Concentration**
Flow (ADWF)	3.8 ML/d		---
COD	2050 kg/d		540 mg/L
TKN	205 kg/d		54 mg/L
TP	38 kg/d		10 mg/L
TA	---		230 mg$CaCO_3$/L
SO_4	---		37 mg/L
Sulfide (estimated generation in sewage rising mains at 19-24-29 degC)	---		2-5-9 mgS/L
Peaking Factors (x 50%ile):	**90%ile**	**Peak Rate**	**Diurnal Peak**
Flow:			
Hydraulics			
Sustained	---	5.8	---
Instantaneous	---	7.1	---
Process	1.3	7.1	2
COD mass load	1.3	---	2.5
Peak flow rate:			
Sustained	255 L/s; 920 m3/h; 22 ML/d		
Instantaneous	314 L/s; 1120 m3/h; 27 ML/d		
50%ile Sewage Characteristics:			
COD/BOD	2.4		
Unbiodeg soluble COD / total COD, fus	0.05		
Unbiodeg particulate COD / total COD, fup	0.20		
RBCOD / CODtotal, fbs	0.15		
TKN/CODtotal	0.100		
TP/CODtotal	0.019		
Unbiodegradable soluble N fraction, fnus	0.035 (raw, decreased by alum dosing)		
ML Temperature (min-ave-max)	19-24-29 degC		

Memorandum

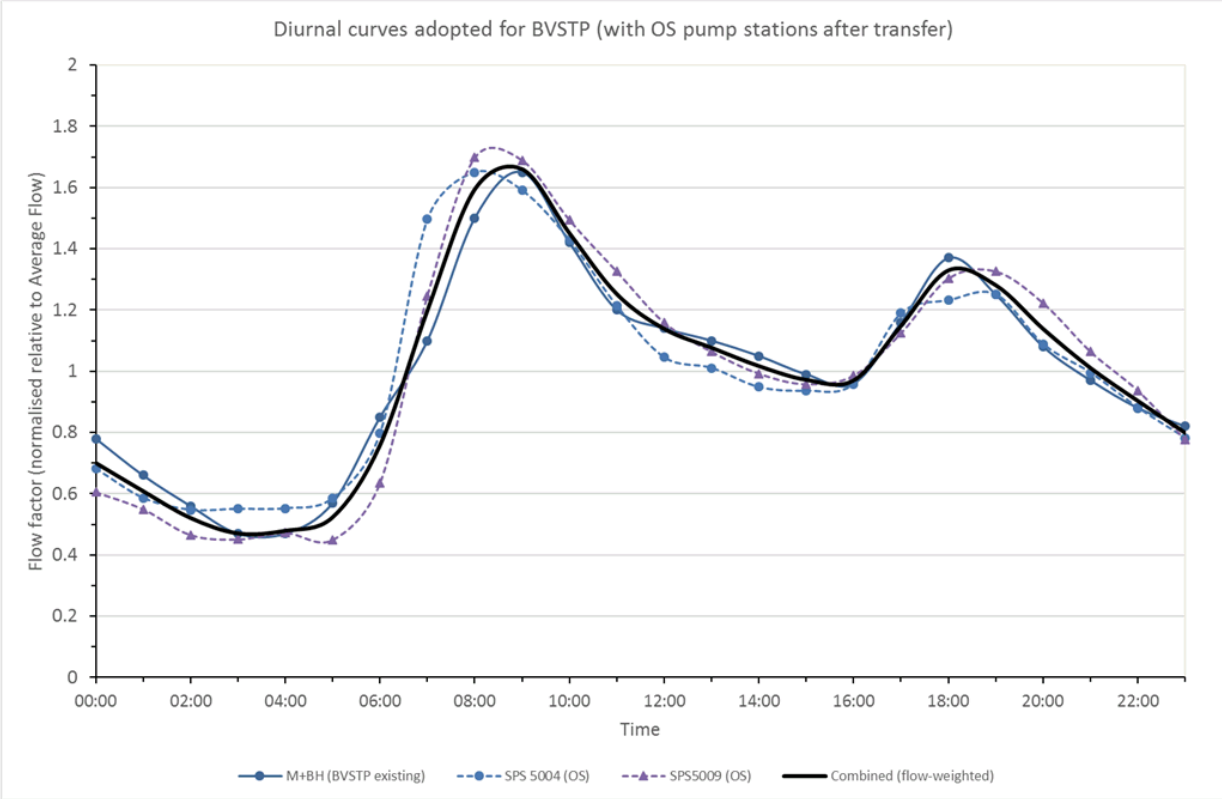


Figure 1 **Adopted diurnal flow curve for BVSTP, including transfer from Ocean Shores catchment**

5

GHD

Memorandum

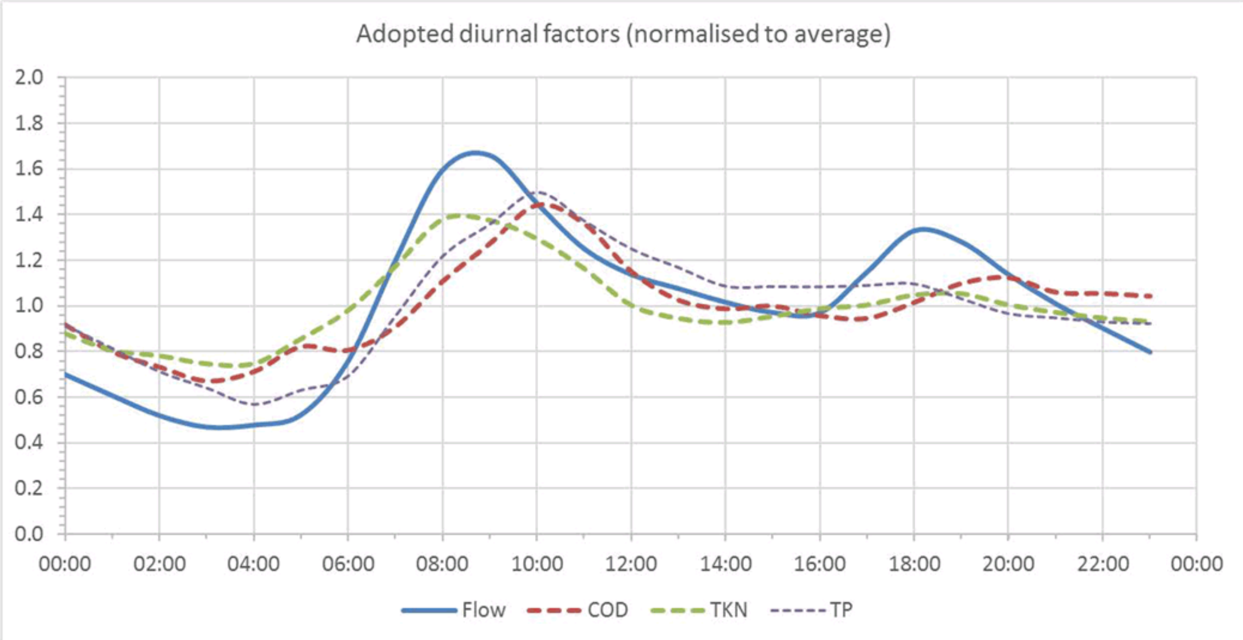


Figure 2 **Adopted diurnal concentration profiles for key parameters (also showing adopted flow curve for comparison, taken from Figure 1).**

6

GHD

Memorandum

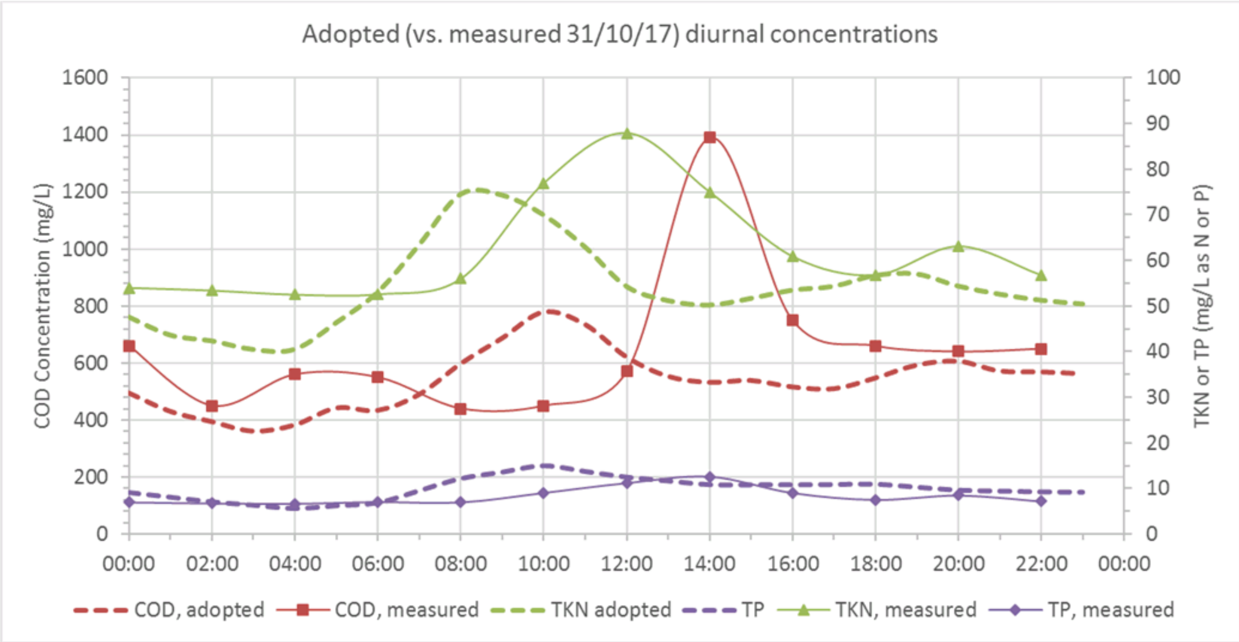


Figure 3 Adopted vs. recently measured (31 October 2017) BVSTP raw influent data for key parameters (compare with Figure 2)

7

Memorandum

2.1.3 Model parameters

All model parameters were set at default, with the exception of those listed in Table 2.

Table 2 Non-default model parameters adopted

Model parameter	Units	Default value	Adopted value	Comment
DO Half-saturation constant for heterotrophs	mg/L	0.05	0.10	Experience with modelling oxidation ditch systems
Ammonia half-saturation constant for ammonia oxidising bacteria (AOB)	mg/L as N	0.70	0.15	
Vesilind settling parameter, Vo	m/d	170	187	For consistency with clarifier modelling in this study. Matched to adopted 50%ile SSVI = 67; DSVI = 100 mL/g (current average DSVI ~107 mL/g)
Vesilind settling parameter, n	m^3/kg	0.37	0.38	
Al:P stoichiometry	mol Al:mol P	0.8	1.5	For alum dosing. Based on research experience for systems with effluent TP<1 mgP/L
Aeration alpha(F) factor for oxidation ditch	-	0.50	range 0.50 to 0.70	Sensitivity testing (to allow for oxygen transfer efficiency and diffuser fouling for likely MLSS ~5,000 mg/L))
Aeration alpha(F) factor for aerobic digester	-	0.50	range 0.25 to 0.35	to allow for oxygen transfer efficiency and diffuser fouling for likely MLSS ~10,000 to 12,000 mg/L)
Superficial gas velocity model parameters (K_1, K_2, Y)	-	Multiple	Multiple	To calibrate to SOTE curve adopted as per Figure 4..
Temperature (mixed liquor)	°C	(20) user defined	25	Indicative of average temperature expected

8

Memorandum

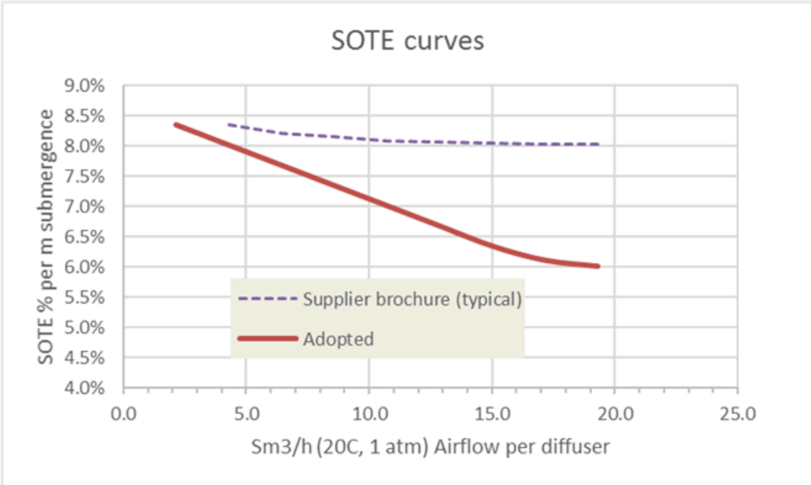


Figure 4 Diffuser oxygen standard oxygen transfer efficiency (SOTE) curve adopted

2.2 Other model set up information

All model setup information (e.g. reactor volumes, aeration system etc.) was derived from the design report (Fulton Hogan, 2010).

Aeration was controlled using a Proportional-Integral (PI) controller, similar to the one used in practice, to control DO downstream of the aerated zone within the oxidation ditch to a DO setpoint of nominally 0.4 mg/L. This resulted in DO depletion to near-zero values at the start of the anoxic fraction of the ditch in accordance with the concept design for the plant (Fulton Hogan, 2010).

3 Results & discussion

The results are presented in respect of the main key process risks identified, as described below.

3.1 Oxidation ditch aeration

The oxidation ditch is aerated via diffused air (in the form of 6 no. Aquablade™ diffuser grids, each equipped with 18 no. Aquablade™ diffusers) and supplied from two duty positive displacement blowers (30 kW each) with one standby blower. 2 m long Aquablade™ diffusers are rated by the supplier (Aquatec-Maxcon) for a maximum airflow per diffuser of around 18 Nm^3/h (or 19.3 Sm^3/h at 20 °C, 1 atm), which equates to a maximum airflow (all diffusers) to the oxidation ditch of 2,086 Sm^3/h. The blowers are rated for 1,005 Nm^3/h (1,078 Sm^3/h each or 2,157 Sm^3/h with 2 no. duty blowers operating).

For the purposes of this assessment, a nominal) maximum capacity rating of 1,078 Sm^3/h each for the two duty blowers was adopted (rounded value for convenience). Similarly, the aeration diffusers were divided equally (54 no. diffusers each) between two aerated ‘cells’, named ‘AE1’ and ‘AE2’ within the

Memorandum

oxidation ditch model reactor series, also with a maximum airflow capacity of 1,078 Sm^3/h each (or 20 Sm^3/h per diffuser).

The model predictions were tested for the sensitivity as follows:

- Diffusers in 'clean' condition: alpha(F) factor = 0.70
- Diffusers in 'dirty/ partially fouled' condition: alpha(F) factor = 0.50

The results are given in Table 3.

Example plots of model airflow predictions are given in Figure 5 and Figure 6.

Table 3 Model airflow predictions for oxidation ditch

Alpha(F) factor	Average Airflow (Sm^3/h, 20°C, 1 atm)	Peak Airflow (Sm^3/h, 20°C, 1 atm) [max. airflow duration, h/d]	DO (mg/L) average in oxidation ditch aerated zones (AE; AE2)	Effluent Ammonia average [max.] mgN/L	Effluent NOx average [max.] mgN/L	Effluent Total N average [max.] mgN/L
0.7 ('Clean' diffusers)	1,559	2,157 [max. airflow 4 h/d]	0.8; 1.4 [0.95; 1.7]	0.5 [1.1]	1.5 [2.4]	4.7 [6.1]
0.5 ('Partially fouled' diffusers)	2,090	2,157 [max. airflow 16 h/d]	0.6; 1.1 [0.75; 1.25]	1.4 [2.7] Note 1	<0.2 [0.4] Note 1	4.3 [5.8] Note 1

Target effluent quality license requirements:

- Ammonia 90th percentile: 2 mgN/L; Max. 4 mgN/L
- Total N 90th percentile: 10 mgN/L; Max. 15 mgN/L

Note 1: Model predictions showed potential failure of nitrification process, with long-term compliance with ammonia and TN licence requirements at risk. This risk will be exacerbated under winter conditions e.g. at 20°C, ammonia concentrations exceed 2 mgN/L on average and 4 mgN/L maximum (licence limits 2 mgN/L 90%ile; 4 mgN/L Max.)

Using the adopted set of wastewater characteristics, the model predictions (Table 3) show that:

- With diffusers in a nominally 'clean' condition, the process performance is satisfactory with peak airflows reaching the maximum capacity of the aeration system (1078 Sm^3/h per blower or 2157 Sm^3/h with 2 no. duty blowers operating) for a duration of approximately 4 hours per day on average. Dissolved oxygen concentrations predicted in the aerated zones remain within an acceptable range and DO at the measured (control) point remain within a reasonably narrow band (±0.17 mg/L) of the setpoint (SP). The predicted effluent quality is expected to meet licence requirements.

Memorandum

- With diffusers in a nominally 'dirty/ partially fouled' condition, the process performance is unsatisfactory with peak airflows reaching the maximum capacity of the aeration system (1078 Sm^3/h per blower or 2157 Sm^3/h with 2 no. duty blowers operating) for extended periods of approximately 16 hours per day on average. Dissolved oxygen concentrations predicted in the aerated zones are lower than expected for good performance and DO at the measured (control) point drops to well below the setpoint (SP), reaching as low as ~0.05 mg/L during the peak loading period of the day when aeration is at maximum capacity. Although the predicted effluent quality might still theoretically meet licence requirements, the simulations showed signs of process instability (e.g. ammonia concentrations exceeding 2 mgN/L on average at a simulated minimum temperature of 20°C). The underlying cause is expected to be insufficient aerobic retention time and DO in the oxidation ditch, due to simulated diffuser fouling and system constraints. There is a significant risk of process (nitrification) failure, particularly at colder (winter) temperatures. Reliable and year-round license cannot be guaranteed and the process loading may be said to have reached maximum capacity.

The simulations were repeated using wastewater characteristics based on a single set of recent samples taken at BVSTP (31 October 2017 – refer to Section 2.1.2). Using this dataset, the model predictions for oxidation ditch aeration suggest that:

- Airflow requirements will reach existing blower maximum airflows for up to 6 hours per day (compared with 4 h/d using the adopted wastewater dataset – see Table 3) using clean diffusers.

With 'dirty/ partially fouled' diffusers, the airflow requirements will reach existing blower maximum airflows for more than 18 hours per day (compared with 16 h/d using the adopted wastewater dataset – see Table 3). Similarly, DO concentrations in the oxidation frequently are predicted to fall below setpoint values for extended periods in the day and the process shows signs of significant instability, particularly for nitrification, with increasing effluent ammonia concentrations (even at average temperature 25°C). Effluent ammonia and TN concentrations are expected to exceed design values and licence exceedance is likely for either or both of these parameters (refer to Figure 7).

Since aeration is expected to be limiting process capacity, the key risk mitigation strategy will be to maintain the diffusers in a good condition. This will involve a combination of the following maintenance routines:

- Regular lifting of the aeration grids (as per manufacturer's manuals) for inspection of condition and cleaning by means of low-pressure hosing;
- Chemical in-situ testing (e.g. back pressure and or SOTE) and chemical cleaning (e.g. acid vapour) of diffusers, at least on an annual basis; and
- Replacement of diffuser membranes or membrane panels as required (in accordance with manufacturer's manuals and recommendations).

Memorandum

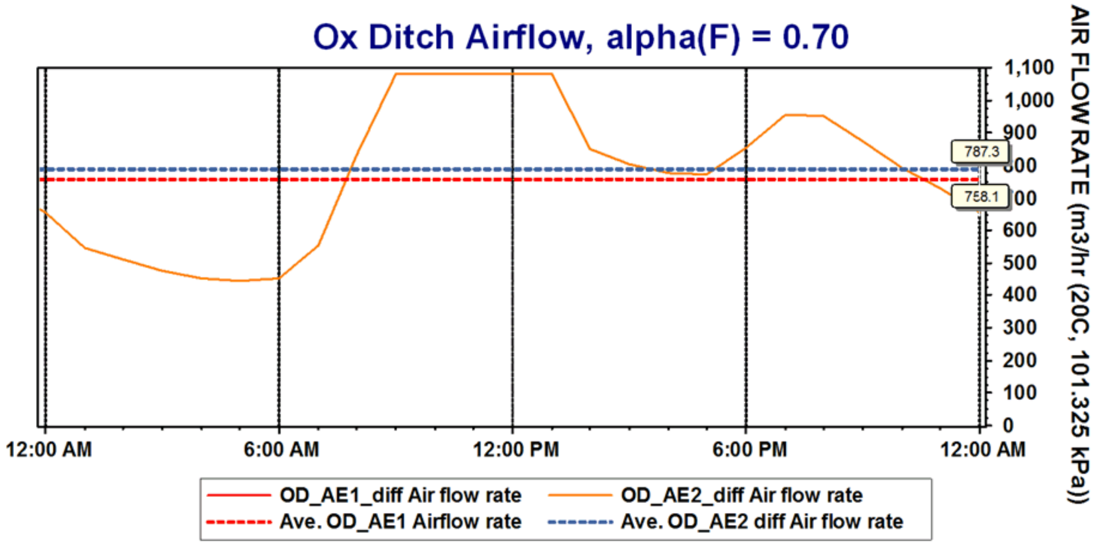


Figure 5 **Model prediction for Oxidation Ditch aerated zones (AE1, AE2) at alpha(F) = 0.70 for 'Clean' diffusers, using adopted wastewater characteristics**

12

Memorandum

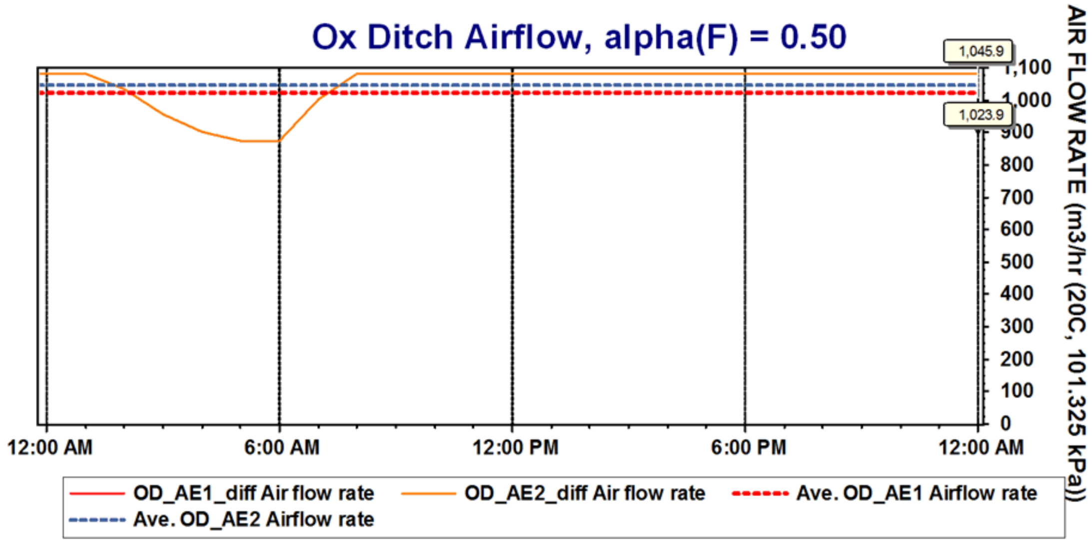


Figure 6 **Model prediction for Oxidation Ditch aerated zones (AE1, AE2) at alpha(F) = 0.50 for 'Dirty/ Partially Fouled' diffusers, using adopted wastewater characteristics**

13

Memorandum

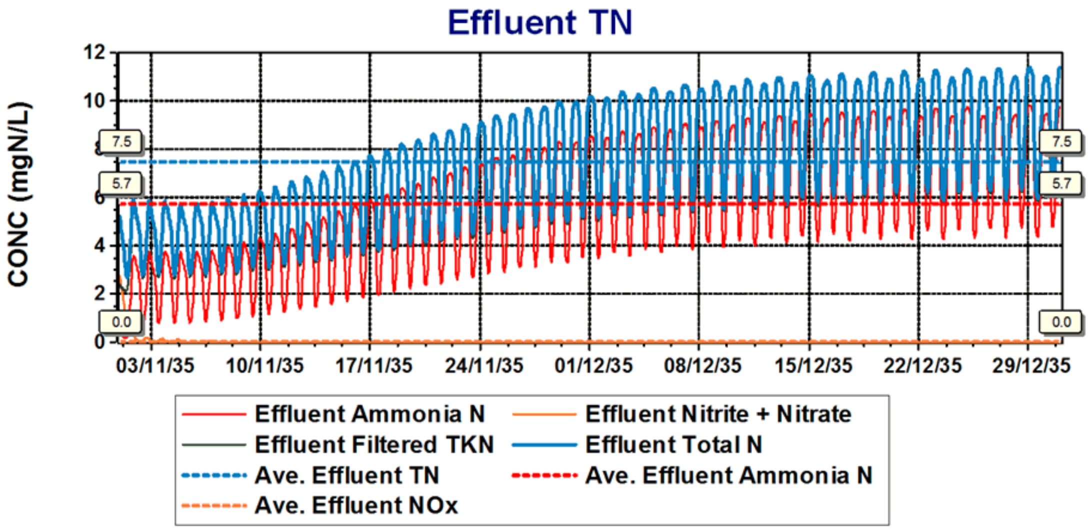


Figure 7 **Model prediction for effluent total N, ammonia and oxidised N, using wastewater characteristics from one recent dataset (31/1/2017) and assuming alpha(F) = 0.50 for 'Dirty/ Partially Fouled' diffusers in the oxidation ditch. Simulation period = 60 days from nominal start date 1/11/2035, at temperature 25 °C. Existing licence requirements: Ammonia 2 mgN/L (90%ile); 4 mgN/L (max); Total N 10 mgN/L (90%ile); 15 mgN/L (max).**

Memorandum

3.2 Oxidation ditch solids inventory

The oxidation ditch was modelled at a nominal sludge age of 18 days, which is 2 days less than the original design (i.e. more 'aggressive' operation to cater for the marginal overloading of the plant by 2035/36, relative to the original design). Solids capture efficiency in the belt filter press (95% assumed) and solids return via decanting of supernatant from the aerobic digester (<750 mg/L TSS in the supernatant) were both taken into account.

Assuming adequate aeration (for 'Clean diffusers' scenario, refer to Section 3.1), the predicted average MLSS in the oxidation ditch was 4,600 mg/L, which is close to (within ~6%) the nominated design 90%ile value of 4,900 mg/L. This shows that process on average at the loading for the 2035/36 projected future scenario has reached or exceeded design capacity (i.e. the peak month or 90%ile MLSS is expected to exceed the design value). Acceptable clarifier performance will be subject to maximum flow passed through the process (i.e. by-passing of high wet weather flows to the proposed storm dam for treatment) as well as sludge settleability – refer to a related Memo in this study for clarifier process capacity risk assessment.

The oxidation ditch capacity will therefore need to be increased in the future for loading exceeding that projected for 2035/36 - refer to strategy proposed in Feasibility Study (GHD, 2016).

One model run was conducted for the 2035/36 loading scenario using the October 2017 additional wastewater characterisation dataset (see Section 2.1.2). The model results, using the same operational assumptions for the oxidation ditch system as for the other model runs (see above) showed a predicted average MLSS of approximately 5,300 mg/L for the latest dataset. This is expected to be close to the maximum MLSS that the clarifiers could tolerate and still be able to treat peak wet weather flows of up to 176 L/s (refer to the related Technical Memorandum on clarifier modelling as part of this study). Further wastewater characterisation and simulation is recommended to confirm this result. If correct, this will serve as further justification for the future process capacity augmentation to be operational no later than 2035-36, as proposed in the Feasibility Study (GHD, 2016).

3.3 Aerobic digester aeration system

Few details were available for the aerobic digester aeration system in the Fulton Hogan (2010) design report. The aerobic digester is known to be fitted with fine bubble diffusers but the number and type were not stated. The same diffusers as in the oxidation ditch (2 m Aquablade™), were assumed for the aerobic digester at an assumed diffuser floor coverage of 6.9%, giving 26 no. diffusers (nominally one grid with 13 pairs of diffusers). The diffusers are served by one duty positive displacement blower with a capacity of 465 Nm^3/h (500 Sm^3/h, 20°C, 1 atm) @ 40 kPa. Aeration was designed to be intermittent (nominally 40% air on time). The digester was also designed to be intermittently decanted (i.e. supernatant decanted). The minimum design water depth was 2.7 m and the maximum water depth 4.5 m. However, a series of decanter pipes were provided for supernatant draw off to different levels.

The digester was modelled with intermittent aeration (2h on/ 2h off i.e. 50% on time) and also operated as an intermittently decanted sequencing batch reactor, with liquid (supernatant) draw off down to a minimum water level corresponding to 80% of maximum water depth (i.e. 3.6 m out of 4.5 m maximum). The total cycle time simulated was 2 days (48 hours) with intermittent aeration assumed to operate for 42

Memorandum

(1d 18 h), followed by settling (2 hours) and decanting (4 hours). Aeration (when on) was to controlled to a nominal DO set point of 0.5 mg/L, but constrained by oxygen transfer efficiency (see alpha factor assumed for aerobic digestion in Table 2) and maximum airflow (500 Sm^3/h, 20°C, 1 atm).

The model results suggested that a DO of 0.5 mg/L should typically be achievable and the maximum airflow requirement should range ~410 to 500 Sm^3/h, i.e. just within the nominal operating range of the blowers. The average operating depth of the aerobic digester was predicted to be 3.645 m, if operated with intermittent decanting of supernatant once every 2 days (see above). The rated operating pressure of the blowers (40 kPa) should be adequate since the highest predicted airflow 500 Sm^3/h occurs when the digester water level is close to minimum where the pressure requirement will be lower (indicatively 33±2 kPa at lower water level of ~2.79 m predicted; design minimum decant level is 2.70 m).

The key risks to maintaining good performance in respect of aerobic digester performance will be:

- Maintaining the diffusers, including periodic cleaning, to ensure sufficient oxygen transfer efficiency;
- Managing the solids inventory in the aerobic digester to not exceed an average of 10,000 mg/L (when full and aerating), also in order to maintain sufficient oxygen transfer efficiency; and
- Maintaining the condition of the single duty blower (no standby) for the digester.

3.4 Aerobic digester solids inventory

As described in Section 3.3, the aerobic digester was modelled as an intermittently aerated and decanted batch reactor with the following cycle:

- Fill/ Aerate (intermittent 50% on/off): 42 hours
- Settle: 2 hours
- Decant: 4 hours
- TOTAL: 48 hours

Thickened (settled) digested WAS was simulated as being withdrawn from the bottom of the digester for the last 4 hours of each cycle, and sent to the model dewatering unit (a belt filter press, BFP) for solids dewatering (95% solids capture).

The results showed an average solids concentration (when aerated) in the aerobic digester of around 9000 mg/L, with a thickened (settled) digested waste activated sludge (TWAS) concentration in the range ~11,000 to 15,000 mg/L. The TWAS was modelled to be withdrawn over a 4h period after the settle period in the digester cycle operation (see above).

The model-predicted percent VSS destruction for the aerobic digester was 18% at a nominal solids retention time (SRT) of 6 days. This is reasonable performance, considering the original design assumed a minimum SRT of 3 days. However, compliance with the sludge SOUR 'stabilisation criterion' in existing NSW Biosolids guidelines was not verified here[2].

[2] Additional modelling will be required to simulate the SOUR (outside the scope of this study).

16

Memorandum

The average daily mass rate of TWAS produced was predicted to be 603 kg/d as TSS. Assuming dewatering every second day (nominally over 4 hours), the solids loading rate to the BFP when operating (as modelled) would theoretically be 302 kg/h TS. This is higher than the design rating of the BFP.

The BFP had a rated design capacity (Fulton Hogan, 2010) of 20 kL/h @ 1.0% TS i.e. 200 kg/h TS.

In practice, the operating time of the belt press would need to be extended to approximately 6.7 hours every second day in order to remain within the design loading capacity of the BFP for the year 2035/36 scenario. This will be possible using the proposed intermittently decanted operation of the aerobic digester with TWAS feed to the BFP withdrawn from the digester during the fill/ aerate part of the cycle at a volumetric rate not exceeding 20 kL/h.

The aerobic digester capacity will need to be increased in the future for loading exceeding that projected for 2035/36 - refer to strategy proposed in Feasibility Study (GHD, 2016).

3.5 Alum dosing requirements

The model predicted that licence requirements for effluent Total P concentration (<1 mgP/L maximum) can be achieved by means of alum dosing, provided stable clarifier performance[3] can be achieved with effluent TSS <4 mg/L as per the original design. Alum dosing was flow-paced according to plant inflow in the model.

The model predicted following alum dosing requirements (as 46% w/w solution[4]) were as follows:

- Average alum flow rate: 434 L/d or 100 L/ML (equivalent to 60 mg/L dry solid alum dose)
- Maximum alum flow rate: 720 L/d (30 L/h for max. 1 hour per day)
- Minimum alum flow rate: 200 L/d (8.3 L/h).

The design 50%ile alum dose was 20 mg/L solid (dry) alum (or 33 L/ML as 46% w/w solution), presumably with a significant degree of bio-P removal assumed to be occurring. The current average dose[5] is 120 L/ML to the oxidation ditch plus approximately 40 L/d to the aerobic digester. This equates[6] to a total alum dose of approximately of 90 mg/L solid (dry) alum (or 195 L/d or 150 L/ML as 46% w/w solution), which is relatively high, suggesting that the bio-P removal mechanism is either not active or partially inhibited.

The model predicted average dose (see above) is higher than the design value but not as high as the current dose. The alum dosing pumps capacity was stated in the Fulton Hogan (2010) design report to be for a design maximum dose rate of 750 L/d (31 L/h). The minimum dose rate was not stated but a turndown of greater than 100:1 is typical for digital dosing pumps and therefore the model predicted range in alum flow rate is expected to be achievable.

[3] Clarifier performance will be function of the combination of peak flows, MLSS (solids inventory management) and sludge settleability - refer to a related Memo in this study for clarifier process capacity risk assessment). The model settler applied in the BioWin™ simulations here at average sludge settleability (refer to parameters in Table 2) indicated that <4 mg/L effluent TSS should be achievable under dry weather loading conditions simulated for the year 2035/36.

[4] Alum (dry solid) formula $Al_2(SO_4)_3.14H_2O$

[5] Information supplied to D de Haas (GHD) by J Arthur (BSC operator) during site visit 15/8/2017.

[6] For the current ADWF of approximately 1.3 ML/d

17

Memorandum

The installed alum storage volume (25,000 L tank) is expected to provide adequate storage (at least 90 days), even if the peak month alum dosing requirement is indicatively double the model predicted average value (see above).

4 Summary

The results from this study of process capacity for the oxidation ditch and related process equipment is summarised in Table 4.

Memorandum

Table 4 Summary of BVSTP process risk assessment at 2035/36 projected loading[7]

Process item	Capacity assessed from model for projected 2035/36 loading	Risk mitigation strategy	Comments
Oxidation ditch aeration system (diffusers and blowers)	Adequate for 'Clean' diffusers Inadequate/ marginal for 'Dirty/ Partially fouled' diffusers condition	Good maintenance (inspection, cleaning, replacement) of diffusers, including regular *in-situ* chemical (acid vapour) cleaning. Good maintenance of existing (dual duty) blowers. Process capacity augmentation (Note 1)	Aeration 'Alpha(F)' factor assessed at: 0.70 for 'Clean' diffusers; 0.50 for 'Partially fouled' diffusers
Oxidation ditch solids inventory (MLSS)	Adequate	Adequate sludge wasting to ensure true sludge age (SRT) of 18 days average (15 days in peak month). Average MLSS ~4,650 to ~5,300 mg/L. Process capacity augmentation (Note 1)	Design 90%ile MLSS 4,900 mg/L expected to be exceeded during peak (summer) month loading at 18d SRT. Increased sludge wasting (15d SRT) during peak summer months likely to be required.
Aerobic digester aeration system (diffusers and blowers)	Adequate	Good maintenance (inspection and cleaning) of diffusers, including regular *in-situ* chemical (acid vapour) cleaning. Good maintenance of existing (single) duty blower. See also *Aerobic digester solids inventory* (below) Process capacity augmentation (Note 1)	Average TSS concentration in the aerobic digester should not be allowed to exceed 10,000 mg/L to avoid significant decrease in oxygen transfer efficiency, compared with typical design (modelled) assumptions.

[7] Based on design wastewater composition and future (2035/36) projected ADWF = 4.34 ML/d, without allowance for dilution due to additional I/I.

19

Memorandum

Process item	Capacity assessed from model for projected 2035/36 loading	Risk mitigation strategy	Comments
Aerobic digester solids inventory (MLSS), volatile solids (VSS) destruction	Adequate (Note 2)	Thicken waste activated sludge in digester by operation of the (existing) supernatant draw off pipes/ valves – see *Comment*. Sufficient operation of the dewatering equipment (belt filter press), to prevent solids recycle to mainstream process via decanted supernatant (maintain TSS in digester below ~10,000 mg/L). Indicative BFP operating times required: 6.7 h every two days, on average.	Automation of the existing (manual) supernatant draw-off valves recommended. Installation of alternative decanter system in the existing digester recommended if the existing supernatant draw-off system is found to be operationally inadequate. Installation of second (standby) belt press recommended (available space in existing sludge dewatering building).
Aerobic digester operation (DO, mode, decanting of supernatant etc.)	Adequate (but requires operational testing)	Thicken waste activated sludge in digester by operation of the (existing) supernatant draw off pipes/ valves – see *Comment*.	Existing (manual) supernatant draw-off valves currently not in use. See above
Alum dosing	Adequate	Good maintenance of existing alum storage and dosing system	-

Note 1: If the catchment growth stagnates (population numbers served become stable) by 2035/36 then the need for capacity augmentation (second, new oxidation ditch) should be reviewed. A possible alternative to relieving the (peak) aeration capacity will be dry weather flow balancing (new tank with mixers and pumps required). However, dry weather flow balancing will not provide relief for the solids inventory constraints.

Note 2: Sludge stabilisation (SOUR of digested sludge) not simulated in this study – requires additional modelling.

Memorandum

5 Recommendations

The following recommendations are made from this study in respect of BVSTP:

1. Put in place an adequate and regular diffuser maintenance and cleaning program for both the oxidation ditch and aerobic digester.
2. Ensure on-going routine and non-routine maintenance of existing blowers for both the oxidation ditch and aerobic digester.
3. Trial operation of the existing (manual) supernatant draw-off valves on the aerobic digester and automate these valves, if feasible. Routine thickening of sludge (i.e. supernatant decanting) in the aerobic digester will be required to sustain adequate solids retention times in the digester in future when the plant is operating close to the design loadings (after transfer from Ocean Shores).
4. Investigate installation of an alternative supernatant decanting system in the aerobic digester if use of the existing valves proves to have operational issues (see above).
5. Ensure sufficient sludge dewatering operating times to maintain solids inventories in the oxidation ditch and aerobic digester that are within the design or operating manual guidelines. Sludge wasting and solids inventory management should be at least sufficient to maintain Ox. Ditch MLSS <4,900 mg/L (90%ile) and aerobic digester TSS <10,000 mg/L (when aerated/ mixed).
6. Consider the need for process capacity augmentation and possible alternatives (e.g. dry weather flow balancing to relieve peak aeration capacity constraint in oxidation ditch) once the plant reaches or approaches design capacity, and implement no later than 2035/36.
7. Prior to detailed design and/or project implementation, carry out additional wastewater characterisation, including both the existing BVSTP and OSSTP raw influents, and confirm the findings from this study by means of further process simulation.

6 References

Fulton Hogan/ Cardno (2010). *Brunswick Valley Sewage Treatment Plant (Contract No. 2008-00001) – Design Report.* Report compiled for Byron Shire by Fulton Hogan/ Cardno/ GHD (Aug. 2010, Version 9).

Regards

David de Haas
Principal Professional, Wastewater Treatment

21

Appendix D – Safety in Design Risk Assessment Matrix

People involved in Risk Assessment:															
					Initial Risk Rating							Residual Risk Rating			
Design Ref	Design Life Cycle Stage (Select from Drop Down Box)	Hazards What could cause injury or ill health, damage to property or damage to the environment	Risk What could go wrong and what might happen as a result	Existing Control Measures	C	L	RR	Potential Control Measures (Consider Hierarchy of Control - Elimination, Substitution, Isolation, Engineering Controls, Administrative Controls, PPE)	Responsibility	By When	Decision / Status	C	L	RR	Comments
1	Investigation and Design	Hydraulic capacity limitation after OSSTP transfer	Raw wastewater overflow	Existing plant hydraulic capacity 314 L/s (instantaneous)	C- Severe	5 - Almost Certain	Significant	Engineering Controls (increase hydraulic capacity via diversion of surplus flows to new stormdam; return flows for treatment after peak wet weather events pass; use new constructed wetland as further environmental buffer for surplus flows in the unlikely event that storm dam spills)	BSC/ designer	Before transfer from OSSTP	Peak flow requirements from combined catchments (M + BH + OS) studied; simulate stormdam capacity requirements using water balance model; 30 ML storm dam capacity recommended (including rainfall capture).	A - Minor	3 - Possible	Negligible	New diversion structure required upstream of existing inlet works at BVSTP; return pump station from stormdam required; constructed wetland downstream of storm dam recommended; smaller wet weather storage capacity (~24 ML, TBC) possible if covered to exclude rainfall surface capture.
2	Investigation and Design	Inadequate odour control/ foul off gas extraction capacity; toxic sewer gases	Sewer gas (odourous; hazardous) escaping existing inlet works; corrosion; OH&S of persons attending site	Four air extraction & treatment from inlet works for odour control via a biofilter (gravel/compost media bed),	C- Severe	5 - Almost Certain	Significant	Engineering Controls (upgraded odour control system with increased airflow rate and additional odour bed/ new OCF; and/or chemical dosing at OS SPS 5004 & 5009 for odour control)	BSC/ designer	Before transfer from OSSTP	Pending; study appropriate odour control measures during concept/ detailed design	A - Minor	3 - Possible	Negligible	Capacity and performance of existing odour control system at BVSTP to be confirmed.
3	Investigation and Design	Limited blower/ aeration system capacity for peak (holiday) plant loads (for combined M+BH+OS catchments); diffusers might be fouled/ aged and perform below design specification	Ammonia spikes in effluent, negative receiving water impacts, Env. Licence non-compliance	Existing aeration system is relatively good condition and might have sufficient spare to cater for anticipated peak oxygen demands (M+BH+OS combined)	B - Major	5 - Almost Certain	Moderate	Regular cleaning of aerationdiffusers. Divert a portion of peak day (peak holiday season) dry weather flows to stormdam and pump back at night. Increase blower size or change drive pulley(s), to increase airflow, only if necessary as futher mitigation.	BSC/ designer	Before transfer from OSSTP	Diurnal process modelling used to confirm peak day oxygen requirements for combined catchments. Acceptable, provided the existing diffusers are maintained in a good state and regularly cleaned.	B - Major	2 - Unlikely	Negligible	A diurnal process model of oxidation ditch was used to simulate expected peak day aeration requirements, using conservative estimates of oxygen transfer efficiency (to allow for some degree of diffuser fouling).
4	Investigation and Design	Limited clarifier &/or RAS pump capacity	Solids carryover, pollution of receiving water (Brunswick River), Env. Licence breach, public (poor disinfection) or ecosystem (solids, DO depletion) health damage	Existing RAS rate max. 150 L/s (combined from 2 no. clarifiers, 4 no. pumps running)	C- Severe	5 - Almost Certain	Significant	Engineering Controls (RAS pump upgrade, if requred; diversion of surplus PWWF to new stormdam to eliminate/ avoid impact on clarifier solids loading)	BSC/ designer	Before transfer from OSSTP	Clarifier capacity confirm by modelling. Current sludge settleability confirmed (DSVI tests). On-going settleability tests recommended. Potential to increase RAS pump max. capacity to ~200 L/s to be futher investigated.	A - Minor	3 - Possible	Negligible	Existing RAS pump design: Max 150 L/s total with 2 no. pumps running (at design peak sewage flow). Min. 20 L/s with 4 no. pumps running (80% of ave. flow at startup load).
5	Investigation and Design	Limited chemical dosing capacity for P removal (high reliance on chemicals; high dose; EBPR mot optimised)	High cost for chemicals (OPEX budget constraints); chemical dosing capacity limit reached; high effluent P (Env. Licence limit breach)	Existing chemical dosing equipment (tanks, pumps & related controls etc.) Alum: 27 kL; Ferric 5 kL; Caustic 9 kL; (Hypo 5 kL, recycled water)	B - Major	4 - Likely	Low	Engineering Controls (install additional tanks for chemical storage with space provided in existing bunds; optimise bio-P removal EBPR mechanism and reduce reliance on chemicals); good maintenance of existing dosing systems.	BSC/ designer	Before transfer from OSSTP	EBPR theoretical bio-P removal and chemical dosing requirements studied by means of modelling; compared theoretical requirements with current chemical dose and installed alum dosing pump capacity.	B - Major	2 - Unlikely	Negligible	Current chemical dose rate setpoints: Ferric Sulphate 25 L/ML (to inlet) Alum 120 L/ML (to Ox. Ditch) + 40.6 L/d to AeDig Caustic Soda 45 L/ML (to Ox. Ditch) + 19.1 L/d to AeDig

continued…./

Design Ref	Design Life Cycle Stage (Select from Drop Down Box)	Hazards What could cause injury or ill health, damage to property or damage to the environment	Risk What could go wrong and what might happen as a result	Existing Control Measures	Initial Risk Rating			Potential Control Measures (Consider Hierarchy of Control - Elimination, Substitution, Isolation, Engineering Controls, Administrative Controls, PPE)	Responsibility	By When	Decision / Status	Residual Risk Rating			Comments
					C	L	RR					C	L	RR	
6	Investigation and Design	Clarifier launder nuisance algal/ sponge growth, causing fouling/ blocking of UV lamps downstream	Public health, receiving water quality/ Env. Licence breach due inadequate disinfection	Existing UV system (in-channel)	B - Major	3 - Possible	Low	Engineering Controls (install improve sieving/ screening upstream of UV lamps; clean sieves/ screens regularly i.e. daily)	BSC/ designer	Before transfer from OSSTP	Pending: check severity of problem, existing sieves/ screens *(if any)* & potential to install new/ better ones etc.	B - Major	2 - Unlikely	Negligible	
7	Investigation and Design	Limited UV disinfection capacity	Public health, receiving water quality/ Env. Licence breach due inadequate disinfection	Existing UV system (in-channel)	B - Major	3 - Possible	Low	Engineering Controls (install additional lamps or reactors with greater UV dose capacity, if required)	BSC/ designer	Before transfer from OSSTP	Subject to satisfactory operation of secondary clarifiers within nominated capacity (see above) and low secondaru effluent suspended solids (~5 mg/L TSS average), existing UV system capacity considered adequate, if well maintained.	B - Major	2 - Unlikely	Negligible	Existing UV system designed for full disinfection at 30 mJ/cm2 dose up to 3xADWF (132 L/s); hydraulic capacity max. 7.1xADWF (314 L/s). Modifications (post commissioning) made to ensure no channel overflows at hydraulic max.
8	Investigation and Design	Limited Aerobic Digester capacity / short hydraulic retention time	Higher odour potential of dewatered sludge (biosolids); increased odour from stored biosolids product or during transport & disposal of cake. Public health nuisance.	Existing Digester volume 500 kL (19m L, 6m W, 4.5m max. water depth) with manual valves for supernatant draw-off	B - Major	5 - Almost Certain	Moderate	Engineering Controls (use supernatant withdrawal via existing pipe with automated valves; install suspended solids meter to automatically detect sludge blanket height in digester for valve auto-control). Build additional digester capacity (new), if required	BSC/ designer	Before or as part of transfer from OSSTP	Aerobic digester SRT requirements and capacity modelled to confirm process requirements. Digester capacity adequate, if operated as designed (with gravity thickening by withdrawal of supernatant).	B - Major	2 - Unlikely	Negligible	Existing digester has facility for decanting supernatant via three draw-off pipes with manual valves. Design intent: Reduce sludge SOUR to <1.5 mg/gTSS.h. 50%ile SRT 3d min.
9	Investigation and Design	No standby sludge dewatering equipment; breakdown of existing single duty gravity drainage deck/ belt filter press &/or related equipment (washwater, polymer dosing)	High solids inventory (due to inability to waste sludge from process), leading to solids carryover from clarifiers (risk as per item 3 above)	Existing (1 no. duty; no standby) Gravity Drainage Deck/ Belt Filter Press + Polymer dosing system	B - Major	4 - Likely	Low	Engineering Controls (bring in hired emergency dewatering equipment; or install standby equipment).	BSC/ designer	Before or as part of transfer from OSSTP	Pending: BSC to consider options (e.g. installation of standby GDD-BFP in available space in existing dewatering building).	B - Major	2 - Unlikely	Negligible	Existing OSSTP GDD-BFP (potentially, if refurbished) or new machine could be installed in existing dewatering buidling, together with ancillary washwaster, polmer dosing and solids loading facilties.
10	Investigation and Design	RAS diversion to Anaerobic Reactor, without screening	More frequent ragging/ maintenance downtime of equipment (mixers and aeration diffusers in Ox. Ditch; RAS & WAS pumps); inpact on process performance and effluent quality (receiving water/ Env. Licence breaches)	RAS recycled via inlet works (not possible in future; hydraulic capacity required for OS transfer)	C- Severe	4 - Likely	Moderate	Engineering Controls (install RAS screen; build future RAS splitter structure with screening facilties; option to relocate existing inlet step screen from OSSTP and possibly to integrate into modified existing structure for RAS in Ox. Ditch).	BSC/ designer	Before or as part of transfer from OSSTP	Pending: BSC to consider option of new RAS splitter structure + screen and associated costs vs. integration of RAS screen into existing OD/ RAS pipework structures, as part of initial plant upgrade detailed design.	B - Major	2 - Unlikely	Negligible	Diverting RAS from inlet works to An. Zone reactor directly will maximise inlet works hydraulic capacity (required for peak inflows with OS transfer); option to use OSSTP existing step screen as RAS screen at BVSTP (capacity likely to be suitable but subject to BSC requirements).

continued…./

Design Ref	Design Life Cycle Stage (Select from Drop Down Box)	Hazards What could cause injury or ill health, damage to property or damage to the environment	Risk What could go wrong and what might happen as a result	Existing Control Measures	Initial Risk Rating			Potential Control Measures (Consider Hierarchy of Control - Elimination, Substitution, Isolation, Engineering Controls, Administrative Controls, PPE)	Responsibility	By When	Decision / Status	Residual Risk Rating			Comments
					C	L	RR					C	L	RR	
11	Investigation and Design	Screenings materials in raw wastewater diverted to new (proposed) stormdam; rags and screening material accumulate in stormdam.	Public health due screenings material drying and blowing away in wind; gross pollutent (rubbish) concerns of screenings material entering environment; risk to health of wildlife, incl. birds & aquatic species	None	C- Severe	4 - Likely	Moderate	Engineering Controls (install fine screen for diverted flows directed to stormdam i.e. upstream of dam).	BSC/ designer	Before or as part of transfer from OSSTP	Pending detailed design considerations around new rising main collector/ diversion/ splitter structure.	B - Major	2 - Unlikely	Negligible	Existing step screen ex. OSSTP might be suitable for relocation (if not redeployed as RAS screen, see item 10. above).
12	Investigation and Design	Stones (e.g. blue stone chips), rocks and large sand quantities in Ocean Shores raw wastewater loads	Damage to mechanical screens, followed by process equipment risks associated with poor screening (similar to item 9. above)	None	C- Severe	4 - Likely	Moderate	Engineering Controls (install coarse screen at new diversion structure on discharge from rising mains, upstream of inlet works or new stormdam fine screens).	BSC/ designer	Before or as part of transfer from OSSTP	Pending design considerations around new rising main collector/ diversion/ splitter structure.	B - Major	2 - Unlikely	Negligible	Coarse screen required, suitable to catching rocks/ stones plus sand trap required for new structure (similar to stormwater sand traps).
13	Investigation and Design	Effluent storage tank capacity limited for combined M + BH + OS catchments; tank overflows on some dry weather days (direct discharge to river or proposed new wetland)	Ebb-tide discharge not possible; recycled water supply to customers is limited by storage (unhappy customers; some recycled water potential lost via tank overflows)	Existing effluent storage tank (capacity TBC)	A - Minor	4 - Likely	Low	Engineering Controls (additional or new effluent storage tank; direct discharge to river ceased if proposed new wetland is built; giving attenuated discharge to river, not direct via existing pipeline)	BSC/ designer	Before or as part of transfer from OSSTP	Pending BSC/ EPA considerations around future licence requirements for BVSTP; recycled water demand to be studied and confirmed; existing tank capacity TBC in relation to recycled water demand.	A - Minor	2 - Unlikely	Negligible	Ebb-tide discharge is a voluntary activity (TBC) by BSC for additional environmental (river health) safeguards; not an existing Env. Licence requirements for BVSTP; future Licence requirements to be agreed with EPA.

GHD

145 Ann Street Brisbane QLD 4000
GPO Box 668 Brisbane QLD 4001
T: (07) 3316 3000 F: (07) 3316 3333 E: bnemail@ghd.com

Document1

Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
A	D de Haas	D Baulch (BSC)				1/12/2017
0	D de Haas	P Ochre	Pip Ochre	P Ochre	Pip Ochre	14/12/2017

www.ghd.com

Report No. 4.2	**Belongil Swamp Drainage Union Report to Council**
Directorate:	Infrastructure Services
Report Author:	Peter Rees, Manager Utilities
File No:	I2018/228
Theme:	Community Infrastructure Sewerage Services

Summary:

DA 10.2017.661.1 (West Byron) has a critical lack of information relating to the impacts the development will have on the hydrology in the catchment.

This lack of information will prevent the Belongil Swamp Drainage Union from implementing a Management Plan in accordance with its obligations under the *Water Management Act 2000* and could have significant negative impacts on the catchment's water quality and quantity which in turn will have a degrading impact on the Belongil ICOL character.

RECOMMENDATION:

That Council supports the Belongil Swamp Drainage Union's submission to DA 10.2017.661.1 and that any decision on the DA (and any other DA's in this catchment) is deferred until critical information regarding the impact of the development on the catchment's hydrology are adequately quantified.

Attachments:

1 BSDU Submission West Byron DA 10.2017.661.1 0118, E2018/12493 , page 100⇩

Report

Since 2015 there have been 11 resolutions of Council regarding Council's interaction with the Belongil Swamp Drainage Union (BSDU) and the Belongil catchment.

Essentially, Council played a major role in the reconstitution of the Belongil Swamp Drainage Union and the re establishment of the Union in accordance with the *Water Management Act 2000* in close cooperation with the NSW Department Primary Industries and Water (NSW DPI)

Council resolution 15-526 resolved that:-

> *"Council estimate a contribution rate it is likely to pay to the reformed S D Union for use of the Drain as part of the effluent path from BBSTP, and pay that amount in advance to assist the Union cover its expenses such as preparing a Management Plan."*

BBSTP – Byron Bay Sewerage Treatment Plant
S D Union – Belongil Swamp Drainage Union (BSDU)

As a result Council has commenced working with the BSDU enabling the BSDU to commission consultants to prepare a management plan for the district. This plan is integral to the proper management of the Belongil Catchment.

Council Resolution 15-525 resolved that "*Council consider the "Bayley Report" as amended after the Panel meeting 28 September complete, and use it as the basis for application for permissions from various authorities (including BSD Union) to introduce a second drainage path as described by Option 2 of the Report."* Council has commenced this planning work.

The BSDU has made a submission that the West Byron DA should not proceed until:-

1. The developer has consulted with the BSDU in accordance with the *Water Management Act 2000.*
2. The BSDU Management Plan for the drainage system is competed.
3. The impacts of the proposed development, in particular the fill and displacement of floodplain and increased pavement area, have on the water quality and quantity of water flowing into the BSDU drainage system.
4. Impacts of the altered floodplain hydrology by the development are quantified.
5. Impacts on the Belongil ICOL of the altered hydrology are quantified.
6. Impacts of the development on acid sulphate soil behaviour.

The BSDU contends the above issues and lack of information will prevent the BSDU from fulfilling its legislative responsibilities under the *Water Management Act 2000* and more information and assessments of impacts on the Belongil catchment are required before the development can be considered for approval.

Financial Implications

Nil

Statutory and Policy Compliance Implications

Compliance with the *NSW Water Management Act 2000*

BELONGIL SWAMP DRAINAGE UNION POBOX 441 BYRON BAY NSW 2481

7th February 2018

The General Manager
Byron Shire Council
POBox 219
Byron Bay, NSW 2482

Dear Sir,

Submission DA 10.2017.661.1 - Ewingsdale Road Byron Bay – proposed 387 Lot subdivision

The board of the Belongil Swamp Drainage Union (BSDU) wishes to advise that we strongly object to DA10.2017.661.1 which is located within the Trust Area.

Under the Water Management Act (Part 3 S214&215) a new subdivision is not entitled to be connected to a Private Drainage Board.
'If a holding is subdivided, a new holding resulting from the subdivision is not entitled to be connected to a PDBs drainage works until a date determined by the board'.
It further states that:
"All works to be constructed must be constructed in accordance with the approval in writing of the board in respect of location, design, form, dimensions and construction".

In regards to DA 10.2017.661.1, located within the drainage district, the BSDU was neither consulted nor informed.
Neither the previous board nor the DPI received correspondence from the developer or council on the matter.
A letter written by Colin Draper, falsely claiming to be secretary of the Belongil Swamp Drainage Union, was submitted by the developer with the DA. The letter was subsequently declared illegal by the board of directors (see correspondence with council 31/01/2018).

In the public interest the board of directors of the BSDU cannot approve the connection of any new holdings created by the proposed subdivision until it is satisfied that this will not negatively impact on the efficient workings of the Union Drain system.

The board has major concerns relating to the drainage management, the excavations and the fill material to be imported within the development area. The implications for the management program of the drainage district are manifold and need to be addressed urgently.
DA 10.2017.661.1 will require extensive works which may adversely affect the flow hydrology for the catchment, increase the potential for acid sulphate within the drainage system (with subsequent adverse environmental impacts), reduction in the ability for the drain to effectively

accommodate drainage from the catchment, affect existing landuses and increase the macroporosity of the catchment.

The information given in the DA is not sufficient. It has not been demonstrated that storm water runoff is being directed into legal points of discharge. Detailed reports of acid sulphate soil management, runoff management during and after construction in regards to the imported soil and the management of storm water and waste water including as to how it will impact on the discharge of the West Byron STP into the drainage system are required.

We believe an EIA (Environmental Impact Assessment) will be required due to the proposed drainage scheme of the proposed development. The currently submitted DA does not include such a statement.

A council report, presented to the Coastal Estuary Catchment Panel in March 2017, recommends that feasibility plans for the development of an additional flow path to deal with the outflow of the West Byron Sewer Treatment Plant should commence. It further recommends feasibility studies for the recommended STP 2025 upgrade.
It is the strong belief of the BSDU board that the construction of the additional flow path and the STP upgrade must be undertaken before any new developments of considerable size are approved.

In cooperation with Byron Shire Council the BSDU has recently commissioned Southern Cross University (Southern Cross GeoScience) in conjunction with Michael Woods & Associates (Environment and Floodplain Management Specialists) to prepare a document addressing issues and resulting in the formulation of a Management Plan for the Drainage area.
Key issues and threatening processes of the development in relation to drainage that have already been identified and which require detailed consideration in the forthcoming Management Plan are:

Floodplain Hydrology

Any works proposed upon the drain have the potential adversely affect the hydrology of the floodplain. As part of the Management Plan preparation, detailed consideration of all potential management options for the drain will be required to be assessed against the objectives and water management principles of the Water Management Act 2000.
Potential future increases in non-stormwater discharges such as from the West Byron Sewer Treatment Plant will need to be considered within the context of the Management Plan.

Tidal Exchange

The drain Management Plan will be required to factor in the intermittent artificial opening of the creek mouth as well as the natural cycle of the closing and opening of the mouth. The ICOLL already services a number of sub-catchments drainage such as: the Town Drain, Union Drain, Industrial Estate Drain and other private and agricultural drainage. The capacity of the ICOLL to cope with current flows is arguably already near or at capacity. Any proposed increase in flow from the catchment should be thoroughly assessed prior to any such increase in drainage flow being permitted.

Acid Sulphate Soils and Groundwater Management

It is known that development within a catchment can result in changes in general groundwater behaviour from proposed hydraulic loading. In swamp areas such as Belongil any significant increases in overburden or increased drainage can affect acid groundwater storage and increases in discharge rates. Increases in groundwater levels are likely to affecting local landuses such as agricultural activities utilising low-lying farm lands.

Considering those key issues and the size and impact of the development the board of directors of the BSDU suggests postponing a decision on the DA until the document being prepared by Southern Cross GeoScience is available and a Drainage Management plan is in place.

In conclusion we refer to The Byron LEP 1988, Part 3 Division 4 Clause 45 "Provision of Services" which states:
"(1) The Council shall not consent to the carrying out of development on any land to which this plan applies unless it is satisfied that prior adequate arrangements have been made for the provision of sewerage, drainage and water services to the land."

The board of directors of the BSDU therefore recommends that council does not consent to the DA before those prior adequate arrangements in relation to drainage (i.e. additional flow path, STP upgrade, EIS, Drainage Management Plan etc.) have been made.

This recommendation is corroborated by the following quotes:

- A letter from the EPA to Phil Holloway dated 17 Sept 2014 stating: "*Whilst the EPA understands that future development was accounted for in the design of existing STP I stress the importance for Council to comprehensively assess the potential impacts of new developments on existing sewerage infrastructure, including effluent re-use and discharge arrangements.",* signed Head Environmental Management Unit North Coast.
- In July 2017 a Council Staff Report tabled by Shannon Burt who heads Sustainable Environment and Economy stated that *"the West Byron development is not in the public interest."*

We confirm that no disclosure of any political donation is required in relation to this submission. Further, no privacy restrictions are requested to be placed on this submission

Sincerely

Tom Vidal
(Secretary Belongil Swamp Drainage Union)
On behalf of the board of directors

Report No. 4.3	**Review of Rural Waste Service Options**
Directorate:	Infrastructure Services
Report Author:	Lloyd Isaacson, Team Leader Resource Recovery and Quarry
File No:	I2018/229
Theme:	Community Infrastructure Waste and Recycling Services

Summary:

Stemming from recent discussion amongst residents, councillors and staff (Rates and Resource Recovery) regarding the current structure of the rural domestic waste service the following report recommends a reviewed structure of the rural waste manage charge structure and service options.

RECOMMENDATION:

That Council approve the introduction of a Rural 140L fortnightly landfill bin service option, with the provision of a Council subsidised compost bin, kitchen caddy and education pack for residents that take up the Rural service, at the introduction of mandatory rural domestic waste charges at commencement of the 2018/19 financial year.

Report

There has been recent discussion amongst residents, councillors and staff (Rates and Resource Recovery) regarding the current structure of the rural domestic waste service. This has stemmed from an on-going historic legacy issue that permitted residents to voluntarily opt out of receiving the service when it was initially introduced approximately 12 years ago, paying only an annual vacant land charge of $25 per annum.

Section 496 of the Local Government Act 1993 requires Council to make and levy and annual charge for Domestic Waste Management services on each parcel of rateable land for which the service is available i.e. properties along the route of the waste collection truck. It is up to Council to decide if the collection charge is mandatory. A charge has to be made and levied whether the service is used or not used but for where it is available. The vacant land or availability charge qualifies as a mandatory domestic waste charge in accordance with section 496. The current adopted Revenue Policy requires that rural occupied properties pay the mandatory collection charge regardless of whether they utilise the service or not, as has been the case for urban properties for many years. This policy attempts to encourage the use of the service where it is available to minimise the cost of the service to everyone that uses it and to ensure waste is properly disposed of (i.e. not illegally dumped or ending up in street bins).

Regardless of the current Revenue Policy, and resulting from the historic legacy issue providing the ability to opt-out of a service, Council currently has 435 properties in the rural collection area that are paying the vacant land charge of $25. An initial review of these properties was conducted to determine the number of these properties likely to have a residential house on-site, utilising data analysis for on-site sewer charge codes, pensioner concessions and owner mailing address comparison to property location address in order to identify possible residential occupancy.

It is anticipated that approximately 390 (90%) of those properties have a residential house on-site. Further geospatial analysis is now being conducted to determine exactly how many of those properties actually lie on an established rural kerbside collection route (there are some areas where a vehicle cannot safely access properties and these properties should not have any charge apply).

This recent discussion of legacy rural rating issues has catalysed the proposed implementation of a rural residential resource recovery and landfill waste reduction program that has been/will be identified as an action in Council's integrated resource recovery and waste management strategy currently being developed. In the 2012 kerbside waste composition audit it was identified that 39.5% of the residual waste bin was organic material.

The proposed program consists of revised service options (in-line with urban collection service options) incentivising Council's strategic objectives of reducing waste to landfill and utilising organic waste material as a resource (via on-site composting).

This will be achieved by providing residents in rural service areas a 140L fortnightly landfill bin service option (in addition to the current 240L fortnightly service option) at a reduced annual charge of $190 (plus $70 waste operations charge) which is 33% or $92 less than the 2018/19 proposed charge of $283 (plus $70 waste operations charge) for a 240L service. This reduction in the annual charge is directly proportionate to that of the urban collection service.

Coupled with a lower annual charge, if a resident takes up a 140L landfill bin at lower annual charge they also have the option to purchase a heavily subsidised compost bin, kitchen caddy and education pack, all delivered to the resident when the existing 240L bin is swapped for the smaller size. The actual charge is yet to be determined, however it would be in the vicinity of a one-off $20-$40 charge that could be added to a customer's rates account and paid in 4 quarterly instalments over the 2018/19 rating/financial year. An associated communication and education program will also be delivered to promote the program.

Although an 80L option is offered in the urban area, it is recommended that it not be included as an option in the rural area for the following reasons:-

- The 80L service accounts for only 6% of urban services and generally related to space constrained multi-unit dwelling premises (requiring smaller bins).
- There is a risk that residents may take up the service only due to the attractiveness of a lower annual charge and not due to the intended incentive of reducing landfill waste volumes. As such, the lower bin volume would provide insufficient capacity resulting in overflowing bins and illegal dumping issues.
- Related to the above, if there was a large uptake of the 80L lower cost service there would be a significant reduction in domestic waste management revenue to Council without necessarily realising the intended objectives of the program to reduce waste volumes to landfill.

Associated with this program will be the implementation of a mandatory Domestic Waste Management charge to all rural properties capable of receiving a kerbside service to align Council's Revenue Policy with the requirements of Section 496 of the Local Government Act 1993.

Also proposed for consideration by the committee is an extension of the current compulsory urban 240L bin weekly organics service (coupled with 80L and 140L landfill bin options) to the village of Federal (with the associated shift to urban 3-bin Domestic Waste Management Charges for those properties). A 2016 waste composition audit identified that 40% of the landfill bin consisted of organic material that could be diverted to the FOGO system, potentially due to lot sizes being smaller than general rural properties resulting in an increased difficulty to manage garden waste material generally not suited to a home compost system. It is also not environmentally, socially or financially viable to run an additional organics collection truck throughout the rural areas, however federal properties are more spatially concentrated and "on-route" to the Lismore organics processing facility.

The proposed program would be rolled-out at the commencement of the 2018/19 financial year to align with the new rating period.

Financial Implications

It is difficult to accurately model the financial implications associated with implementation of the proposed program, due largely to the unknown uptake rates of the 140L landfill bin service option. The below table models the impact on the domestic waste management revenue associated with various uptake %

Table 1: Recommended Scenario - 140L option implemented and mandatory charge applied to all properties

	140L Service uptake Rate				
	0	25%	50%	75%	100%
DWM Revenue Forecast	$900,223	$826,759	$753,295	$679,832	$606,368
Business As Usual (BAU) 18/19 DWM Revenue Forecast (i.e. if no 140L option implemented and No 240L mandatory charge applied)	$801,173	$801,173	$801,173	$801,173	$801,173
Increase (-)/Decreased DWM revenue (from BAU)	-$99,050	-$25,586	$47,878	$121,341	$194,805

With regard to costing the roll-out of the subsidised compost bins, caddies and education campaign precise figure are yet to be determined however it is anticipated to be in the vicinity of $20,000 sourced from the domestic waste budget reserve.

With regard to projected volume of waste to landfill reduction and associated savings to Council in waste management costs, the below table provides a very rough and conservative guide to the volumes that may be realised.

Reduction in Waste		Savings to Council
%	Volume (tonnes)	@100/tonne
10%	78	$7,841
20%	157	$15,681
30%	235	$23,522

Statutory and Policy Compliance Implications

As per above report