3 CLOSED SESSION

MOTION TO CLOSE THE MEETING AT 9.09AM

Moved by: Cr M Edwards
Seconded by: Cr P Bishop

That the meeting be closed to the public under section 72(1) of the *Local Government (Operations) Regulation 2010* to discuss the following item:

3.1 Bunker Road Structure Plan

The reason that this is applicable in this instance is as follows:

> *(h) other business for which a public discussion would be likely to prejudice the interests of the local government or someone else, or enable a person to gain a financial advantage.*

CARRIED

MOTION TO REOPEN MEETING AT 9.56AM

Moved by: Cr M Elliott
Seconded by: Cr P Bishop

That the meeting be again opened to the public.

CARRIED

3.1 BUNKER ROAD STRUCTURE PLAN

Dataworks Filename:	**LUP Planning – Bunker Road Precinct Plan**
Responsible Officer:	**Gary Photinos** **Manager City Planning & Environment**
Author:	**Alan Milijkovic** **Strategic Planner**

EXECUTIVE SUMMARY

A confidential report from Manager City Planning & Environment was discussed in closed session.

PROPOSED MOTION

Moved by: Cr W Boglary
Seconded by: Cr M Elliott

That Council resolve as follows:

1. To adopt the proposed changes to the draft Bunker Road Structure Plan and required Redlands Planning Scheme amendments as detailed in Attachment 2 suggested by the first State interest review for the purposes of ministerial approval; and
2. That the draft Bunker Road Structure Plan and associated proposed amendments, Attachments 2 and 3, remain confidential until:

a) Written agreement from the Minister confirming that Council may proceed to public notification;
b) All landowners within the structure plan area have been given prior notification; and
c) Council proceeds to public notification and a call for submissions.

On being put to the vote the motion was LOST.

DIVISION

FOR: Crs Boglary, Ogilvie and Elliott.

AGAINST: Crs Hardman, Edwards, Williams, Beard, Bishop and Talty.

Crs Hewlett and Gleeson were absent from the meeting.

COMMITTEE RECOMMENDATION

Moved by: Cr K Williams
Seconded by: Cr P Bishop

That Council resolve as follows:

1. **To defer making a decision on the draft Bunker Road Structure Plan to the Environment and Planning Committee scheduled for 8th August 2012 where the committee can:**
    a) **Exercise it with the delegated authority to make a formal decision on the matter; and**
    b) **Allow Councillors to seek further clarification on the matter to occur prior to that committee date.**
2. **That the draft Bunker Road Structure Plan and associated proposed amendments Attachments 2 and 3, remain confidential.**

CARRIED

DIVISION

FOR: Crs Hardman, Edwards, Williams, Beard, Bishop and Talty.

AGAINST: Crs Boglary, Ogilvie and Elliott

Crs Hewlett and Gleeson were absent from the meeting.

12.3 CLOSED SESSION AT COMMITTEE

The Committee meeting was closed to the public under section 72(1) of the *Local Government (Operations) Regulation 2010* to discuss the following item, and following deliberation on this matter, the Committee meeting was again opened to the public.

12.3.1 BUNKER ROAD STRUCTURE PLAN

Dataworks Filename:	**LUP Planning – Bunker Road Precinct Plan**
Responsible Officer:	**Gary Photinos** **Manager City Planning & Environment**
Author:	**Alan Milijkovic** **Strategic Planner**

EXECUTIVE SUMMARY

A confidential report from Manager City Planning & Environment was discussed in closed session at Committee.

PROPOSED MOTION AT COMMITTEE

Moved by: Cr W Boglary
Seconded by: Cr M Elliott

That Council resolve as follows:

1. To adopt the proposed changes to the draft Bunker Road Structure Plan and required Redlands Planning Scheme amendments as detailed in Attachment 2 suggested by the first State interest review for the purposes of ministerial approval; and
2. That the draft Bunker Road Structure Plan and associated proposed amendments, Attachments 2 and 3, remain confidential until:
    a) Written agreement from the Minister confirming that Council may proceed to public notification;
    b) All landowners within the structure plan area have been given prior notification; and
    c) Council proceeds to public notification and a call for submissions.

On being put to the vote the motion was LOST.

DIVISION

FOR: Crs Boglary, Ogilvie and Elliott

AGAINST: Crs Hardman, Edwards, Williams, Beard, Bishop and Talty

Crs Hewlett and Gleeson were absent from the meeting.

COMMITTEE RECOMMENDATION/ COUNCIL RESOLUTION

Moved by: **Cr J Talty**
Seconded by: **Cr M Edwards**

That Council resolve as follows:

1. **To defer making a decision on the draft Bunker Road Structure Plan to the Environment and Planning Committee scheduled for 8th August 2012 where the committee can:**
   a) **Exercise it with the delegated authority to make a formal decision on the matter; and**
   b) **Allow Councillors to seek further clarification on the matter to occur prior to that committee date.**
2. **That the draft Bunker Road Structure Plan and associated proposed amendments Attachments 2 and 3, remain confidential.**

CARRIED (en bloc)

Resolution Memo

To	Gary Photinos – Manager City Planning & Environment
From	Office of Chief Executive Officer
Date	27 July 2012
Dataworks File	LUP Planning – Bunker Road Precinct Plan
Subject	**BUNKER ROAD STRUCTURE PLAN**

General Meeting Minutes of 25 July 2012, Item No. 12.3.1 refers.

The following is the resolution on this item:

COMMITTEE RECOMMENDATION/
COUNCIL RESOLUTION

Moved by: **Cr J Talty**
Seconded by: **Cr M Edwards**

That Council resolve as follows:

1. **To defer making a decision on the draft Bunker Road Structure Plan to the Environment and Planning Committee scheduled for 8th August 2012 where the committee can:**
    a) **Exercise it with the delegated authority to make a formal decision on the matter; and**
    b) **Allow Councillors to seek further clarification on the matter to occur prior to that committee date.**
2. **That the draft Bunker Road Structure Plan and associated proposed amendments Attachments 2 and 3, remain confidential.**

CARRIED (en bloc)

This is now forwarded to you for action in accordance with the resolution.

Susan Rankin
Interim Chief Executive Officer

1.3 BUNKER ROAD STRUCTURE PLAN

Dataworks Filename:	**LUP Planning - Bunker Road Precinct Plan**
Responsible Officer:	**Gary Photinos** **Manager City Planning & Environment**
Author:	**Alan Miljkovic** **Strategic Planner**

EXECUTIVE SUMMARY

At the General meeting of 25 July 2012, Council resolved to defer making a decision on the draft Bunker Road Structure Plan until the Environment and Planning Committee meeting scheduled for 8 August 2012 to allow Councillors to seek further clarification prior to making a decision.

This report seeks to confirm Council's decision to defer the planning for the Bunker Road Emerging Urban Communities (EUC) area.

PURPOSE

The purpose of this report is to confirm deferral of detailed planning for the Bunker Road Structure Plan until after adoption of the new planning scheme. The new planning scheme will identify the Victoria Point local development area in the strategic framework to align with the South East Queensland Regional Plan (SEQRP). Once the new planning scheme is adopted, planning for the Victoria Point local development area, incorporating the detailed planning for Bunker Road, will be undertaken.

BACKGROUND

Past Council Decisions

The draft Bunker Road Structure Plan was first presented to Council at the General Meeting on the 14 December 2011 (Item No. 15.5.1), the Council resolved the following:

1. *To adopt the draft Bunker Road Structure Plan and required Redland Planning Scheme (RPS) amendments for the purposes of first State interest review;*

2. *That the draft Bunker Road Structure Plan and associated proposed amendments to the RPS remain **confidential** pending written agreement from the Minister confirming that Redland City Council may proceed to public notification;*

The Bunker Road Structure Plan was to remain confidential to allow consultation with individual property owners in the area prior to publicly releasing the Structure Plan.

At the General meeting of 25 July 2012, Council resolved to defer making a decision on the draft Bunker Road Structure Plan until the Environment and Planning Committee meeting scheduled for 8 August 2012 to allow Councillors to seek further clarification prior to making a decision.

It has since been proposed that the Bunker Road Structure Plan be deferred until after adoption of the new planning scheme. Planning for the Bunker Road Structure Plan can then be combined with planning for the Victoria Point local development area on Double Jump Road (identified by the South East Queensland Regional Plan 2009-2031).

Planning for this area will only commence once there has been substantial uptake of the South-east Thornlands and Kinross Road development areas and so will be undertaken after commencement of the new planning scheme.

The Bunker Road Structure Plan was therefore withdrawn from the Environment and Planning Committee meeting of 8 August 2012 with a fresh recommendation coming before the current meeting.

ISSUES

The Bunker Road Structure Plan

Location

The Bunker Road Structure Plan area comprises those properties zoned Emerging Urban Community (EUC), and consists of 27ha of land over nine properties on the southern side of Bunker Road, Victoria Point. The Bunker Road EUC is located approximately 2km south-west of the Victoria Point Major Activity Centre.

Planning context

The subject area is a remnant of the Special Planning Intent Area No.5 which was identified in the 1998 Redland Shire Strategic Plan. That plan stated: "*...Bunker Road is considered to be suitable for urban residential purposes. Areas to be retained for conservation, public open space, buffers for existing poultry farms and drainage purposes are to be determined at the time a development application is received".* The balance of the SPI5 area which had not been developed at the time that the 2006 planning scheme came into force became EUC zoned.

The EUC zone under the RPS requires Council to prepare a Structure Plan and amendment to the RPS prior to any development taking place. The Bunker Road EUC is included within the Urban Footprint under the South East Queensland Regional Plan 2009-2031 (SEQRP) and is a small component of the larger Victoria Point local development area on Bunker Road, which has the potential to accommodate future urban development.

The draft Local Growth Management Strategy (LGMS) identified this larger Victoria Point local development area as a major potential greenfield development area, and anticipated that together the areas could provide approximately 600 dwellings. The LGMS also recognised that planning of these areas must address conservation, open space and drainage issues.

A decision to defer planning in this area will not have a substantial long term planning effect. There is currently no demonstrated need for the land to be released for urban growth purposes. The current EUC zoning will control development in the area until a detailed plan is put into place.

RELATIONSHIP TO CORPORATE PLAN

5. Wise planning and design

We will carefully manage population pressures and use land sustainably while advocating and taking steps to determine limits of growth and carrying capacity on a local and national basis, recognising environmental sensitivities and the distinctive character, heritage and atmosphere of local communities. A well-planned network of urban, rural and bushland areas and responsive infrastructure and transport systems will support strong, healthy communities.

FINANCIAL IMPLICATIONS

A decision to defer the Bunker Road structure plan will not have any financial implications on Council.

PLANNING SCHEME IMPLICATIONS

Deferring the Bunker Road Structure Plan will have no immediate effect on the RPS. Future planning for the Victoria Point local development area will result in amendments to the new Planning Scheme.

CONSULATION

The Mayor, Divisional Councillor and senior Council officers were consulted in the preparation of this report.

OFFICER'S/COMMITTEE RECOMMENDATION

Moved by: **Cr P Gleeson**
Seconded by: **Cr P Bishop**

That Council resolve as follows:

1. **To suspend the current planning processes for preparation of the Bunker Road Structure Plan (EUC zoned area);**
2. **That the Bunker Road EUC area be recognised as part of the planning for the broader Victoria Point local development area within the new planning scheme;**
3. **Undertake the planning for the Victoria Point (including Double Jump Road and Bunker Road) at an appropriate time after the adoption of the new Redlands Planning Scheme; and**
4. **That the Minister for State Development, Infrastructure and Planning be advised in writing that council does not intend to proceed further with the Bunker Road Structure Plan and will include the area in a wider planning study for Victoria Point at a later date.**

CARRIED (unanimously)

Crs Williams and Boglary were not present when this motion was put.
Cr Elliott was absent from the meeting.

12.1.3 BUNKER ROAD STRUCTURE PLAN

Dataworks Filename:	**LUP Planning - Bunker Road Precinct Plan**
Responsible Officer:	**Gary Photinos** **Manager City Planning & Environment**
Author:	**Alan Miljkovic** **Strategic Planner**

EXECUTIVE SUMMARY

At the General meeting of 25 July 2012, Council resolved to defer making a decision on the draft Bunker Road Structure Plan until the Environment and Planning Committee meeting scheduled for 8 August 2012 to allow Councillors to seek further clarification prior to making a decision.

This report seeks to confirm Council's decision to defer the planning for the Bunker Road Emerging Urban Communities (EUC) area.

PURPOSE

The purpose of this report is to confirm deferral of detailed planning for the Bunker Road Structure Plan until after adoption of the new planning scheme. The new planning scheme will identify the Victoria Point local development area in the strategic framework to align with the South East Queensland Regional Plan (SEQRP). Once the new planning scheme is adopted, planning for the Victoria Point local development area, incorporating the detailed planning for Bunker Road, will be undertaken.

BACKGROUND

PAST COUNCIL DECISIONS

The draft Bunker Road Structure Plan was first presented to Council at the General Meeting on the 14 December 2011 (Item No. 15.5.1), the Council resolved the following:

1. *To adopt the draft Bunker Road Structure Plan and required Redland Planning Scheme (RPS) amendments for the purposes of first State interest review;*

2. *That the draft Bunker Road Structure Plan and associated proposed amendments to the RPS remain* ***confidential*** *pending written agreement from the Minister confirming that Redland City Council may proceed to public notification;*

The Bunker Road Structure Plan was to remain confidential to allow consultation with individual property owners in the area prior to publicly releasing the Structure Plan.

At the General meeting of 25 July 2012, Council resolved to defer making a decision on the draft Bunker Road Structure Plan until the Environment and Planning Committee meeting scheduled for 8 August 2012 to allow Councillors to seek further clarification prior to making a decision.

It has since been proposed that the Bunker Road Structure Plan be deferred until after adoption of the new planning scheme. Planning for the Bunker Road Structure Plan can then be combined with planning for the Victoria Point local development area on Double Jump Road (identified by the South East Queensland Regional Plan 2009-2031). Planning for this area will only commence once there has been substantial uptake of the South-east Thornlands and Kinross Road development areas and so will be undertaken after commencement of the new planning scheme.

The Bunker Road Structure Plan was therefore withdrawn from the Environment and Planning Committee meeting of 8 August 2012 with a fresh recommendation coming before the current meeting.

ISSUES

The Bunker Road Structure Plan

Location

The Bunker Road Structure Plan area comprises those properties zoned Emerging Urban Community (EUC), and consists of 27ha of land over nine properties on the southern side of Bunker Road, Victoria Point. The Bunker Road EUC is located approximately 2km south-west of the Victoria Point Major Activity Centre.

Planning context

The subject area is a remnant of the Special Planning Intent Area No.5 which was identified in the 1998 Redland Shire Strategic Plan. That plan stated: "*...Bunker Road is considered to be suitable for urban residential purposes. Areas to be retained for conservation, public open space, buffers for existing poultry farms and drainage purposes are to be determined at the time a development application is received".* The balance of the SPI5 area which had not been developed at the time that the 2006 planning scheme came into force became EUC zoned.

The EUC zone under the RPS requires Council to prepare a Structure Plan and amendment to the RPS prior to any development taking place. The Bunker Road EUC is included within the Urban Footprint under the South East Queensland Regional Plan 2009-2031 (SEQRP) and is a small component of the larger Victoria Point local development area on Bunker Road, which has the potential to accommodate future urban development.

The draft Local Growth Management Strategy (LGMS) identified this larger Victoria Point local development area as a major potential greenfield development area, and anticipated that together the areas could provide approximately 600 dwellings. The LGMS also recognised that planning of these areas must address conservation, open space and drainage issues.

A decision to defer planning in this area will not have a substantial long term planning effect. There is currently no demonstrated need for the land to be released for urban growth purposes. The current EUC zoning will control development in the area until a detailed plan is put into place.

RELATIONSHIP TO CORPORATE PLAN

5. Wise planning and design

We will carefully manage population pressures and use land sustainably while advocating and taking steps to determine limits of growth and carrying capacity on a local and national basis, recognising environmental sensitivities and the distinctive character, heritage and atmosphere of local communities. A well-planned network of urban, rural and bushland areas and responsive infrastructure and transport systems will support strong, healthy communities.

FINANCIAL IMPLICATIONS

A decision to defer the Bunker Road structure plan will not have any financial implications on Council.

PLANNING SCHEME IMPLICATIONS

Deferring the Bunker Road Structure Plan will have no immediate effect on the RPS. Future planning for the Victoria Point local development area will result in amendments to the new Planning Scheme.

CONSULATION

The Mayor, Divisional Councillor and senior Council officers were consulted in the preparation of this report.

OFFICER'S/COMMITTEE RECOMMENDATION/ COUNCIL RESOLUTION

Moved by: **Cr J Talty**
Seconded by: **Cr M Elliott**

That Council resolve as follows:

1. **To suspend the current planning processes for preparation of the Bunker Road Structure Plan (EUC zoned area);**
2. **That the Bunker Road EUC area be recognised as part of the planning for the broader Victoria Point local development area within the new planning scheme;**
3. **Undertake the planning for the Victoria Point (including Double Jump Road and Bunker Road) at an appropriate time after the adoption of the new Redlands Planning Scheme; and**
4. **That the Minister for State Development, Infrastructure and Planning be advised in writing that council does not intend to proceed further with the Bunker Road Structure Plan and will include the area in a wider planning study for Victoria Point at a later date.**

CARRIED (en-bloc)

Resolution Memo

Redland
CITY COUNCIL

To	Gary Photinos – Manager City Planning & Environment
From	Office of Chief Executive Officer
Date	2 November 2012
Dataworks File	LUP Planning – Bunker Road Precinct Plan
Subject	**BUNKER ROAD STRUCTURE PLAN**

General Meeting Minutes of 31 October 2012, Item No. 12.1.3 refers.

The following is the resolution on this item:

OFFICER'S/COMMITTEE RECOMMENDATION/
COUNCIL RESOLUTION

Moved by: **Cr J Talty**
Seconded by: **Cr M Elliott**

That Council resolve as follows:

1. **To suspend the current planning processes for preparation of the Bunker Road Structure Plan (EUC zoned area);**
2. **That the Bunker Road EUC area be recognised as part of the planning for the broader Victoria Point local development area within the new planning scheme;**
3. **Undertake the planning for the Victoria Point (including Double Jump Road and Bunker Road) at an appropriate time after the adoption of the new Redlands Planning Scheme; and**
4. **That the Minister for State Development, Infrastructure and Planning be advised in writing that council does not intend to proceed further with the Bunker Road Structure Plan and will include the area in a wider planning study for Victoria Point at a later date.**

CARRIED (en-bloc)

This is now forwarded to you for action in accordance with the resolution.

Susan Rankin
Interim Chief Executive Officer

MAKE A
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MAKE IT
COUNT

Workshop

22 November 2016

Victoria Point Local Development Area Structure Plan

Note: Workshop presentations and discussions are confidential

Redland
CITY COUNCIL

Content

- Purpose
- Proposed Development
- Victoria Point Local Development Area
- Structure Plan

Note: Workshop presentations and discussions are confidential

Proposed Development

- Reconfiguring a Lot for 1 into 289 lots and 7 balance lots - March 2015
  - – later reduced to 263 residential lots
- Request for further information (structure plan) – February 2016
- Response received in November 2016
- 4 week period of community consultation commences tomorrow

Note: Workshop presentations and discussions are confidential

Proposed Development

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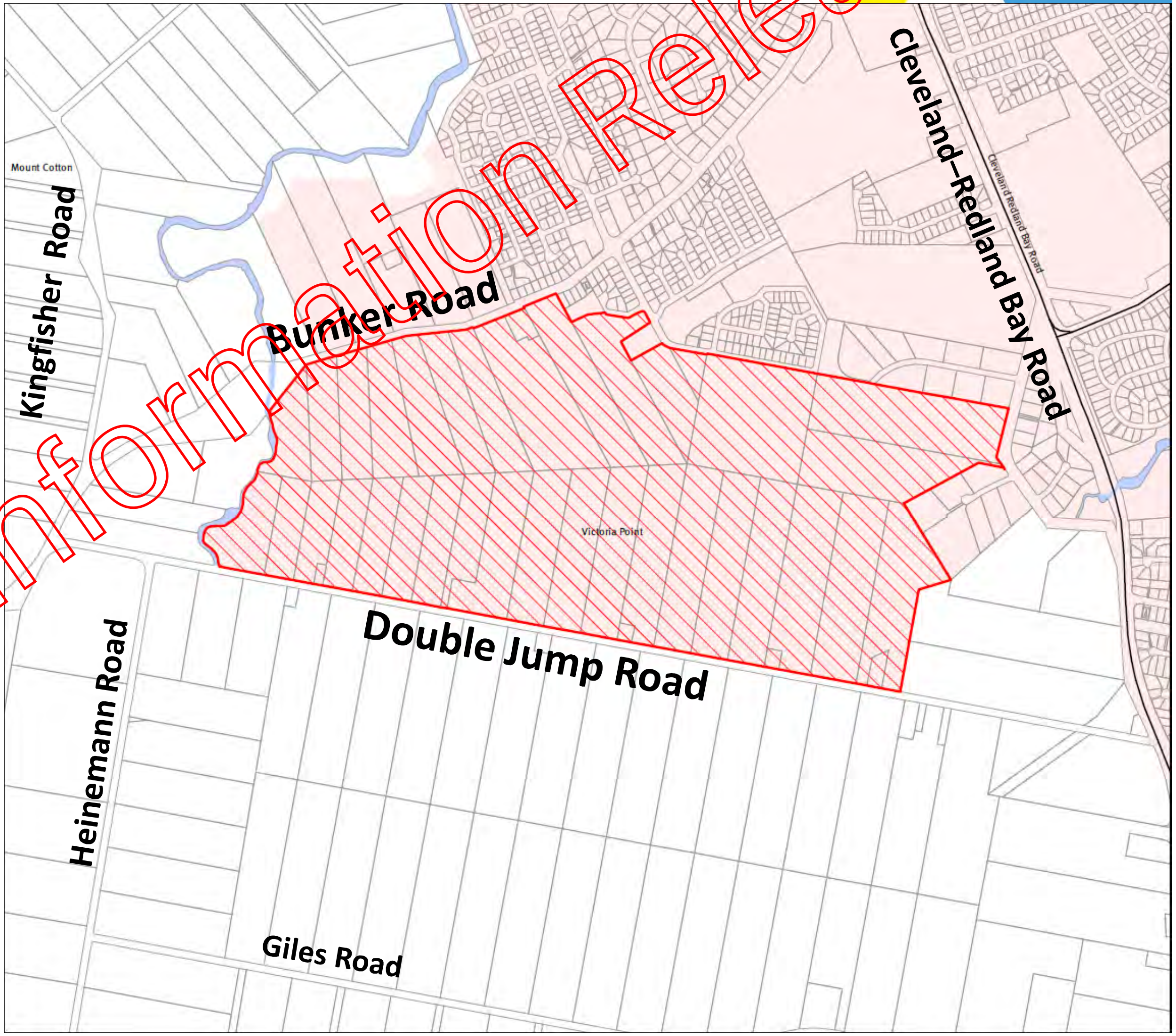
YIELD AND MIX																	
	Premium Villa		Courtyard		Traditional Lot		Premium Villa		Courtyard		Traditional Lot		800sqm+		Vender Lot		TOTAL
average width	12.5m		14m		16m		12.5m		14m		16m		20-25m				
average depth	32m		32m		32m		25m		25m		25m		32-40m				
average size	400sqm		450sqm		512+sqm		312sqm		350sqm		400sqm		800+sqm		2000+sqm		
YIELD	56	21%	61	23%	45	17%	35	13%	23	9%	15	6%	27	10%	7	3%	270

Victoria Point Local Development Area

- Where is it and what is it?
- Designated by the *South East Queensland Regional Plan 2009* alongside South East Thornlands and Kinross Road
- Within the Urban Footprint

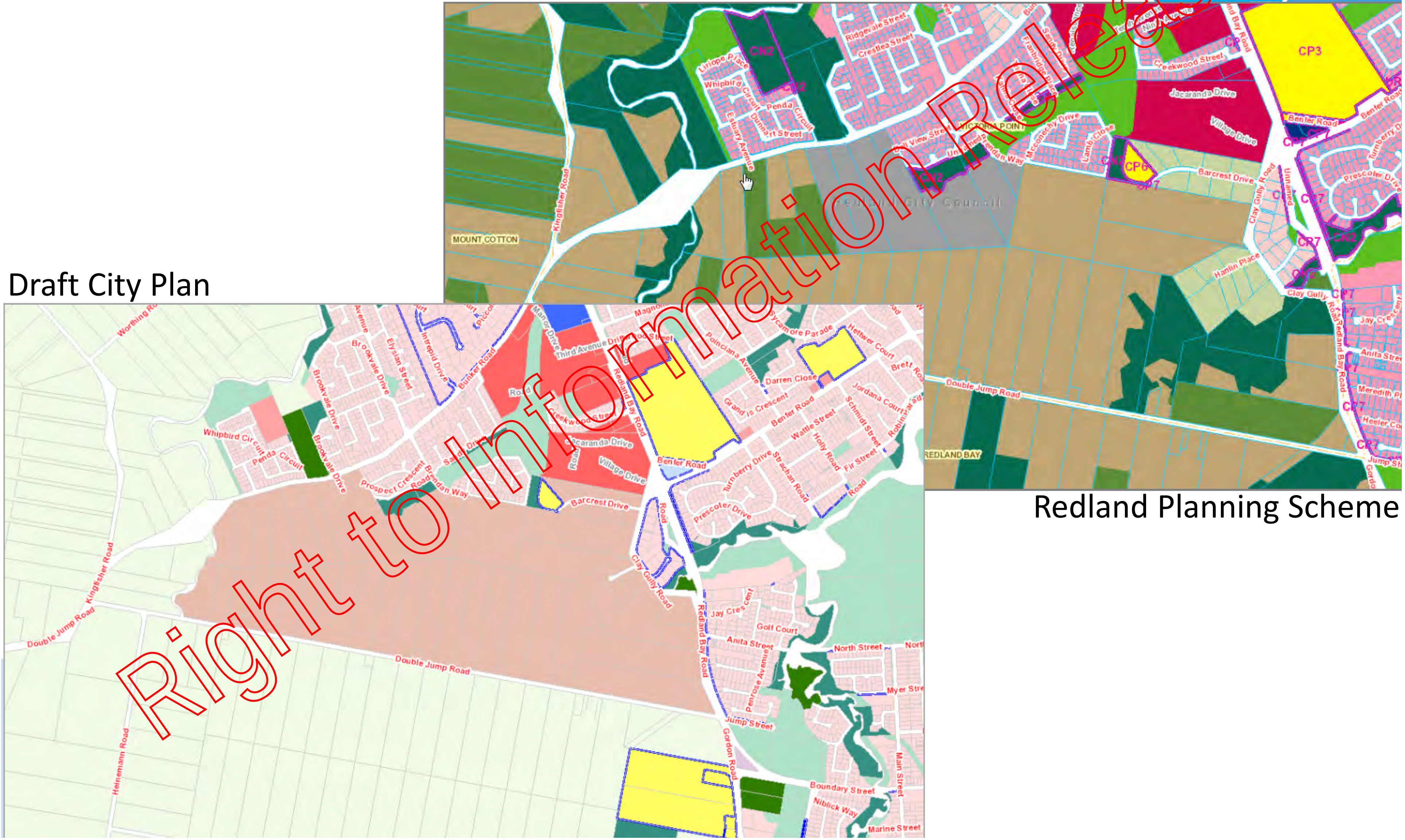


What is a local development area?

- South East Thornlands and Kinross Road
- Focus for accommodating regional dwelling and employment targets
- Comprehensive planning to co-ordinate development with infrastructure delivery

Note: Workshop presentations and discussions are confidential

Redlands Planning Scheme & draft City Plan



Draft City Plan

Redland Planning Scheme

Structure Plan process

- SEQ Regional Plan
  - Planning of development areas to be led by councils, developers or the State government
    - Analysing the area and its context
    - Consideration of Council and State policies and requirements
    - Examining infrastructure needs, staging, timing and funding
  - Plans can be:
    - Prepared formally as a Structure Plan where the Minister has declared the area a Master Plan area
    - Prepared informally and then used as a basis for submitting a planning scheme amendment or development application

Note: Workshop presentations and
discussions are confidential

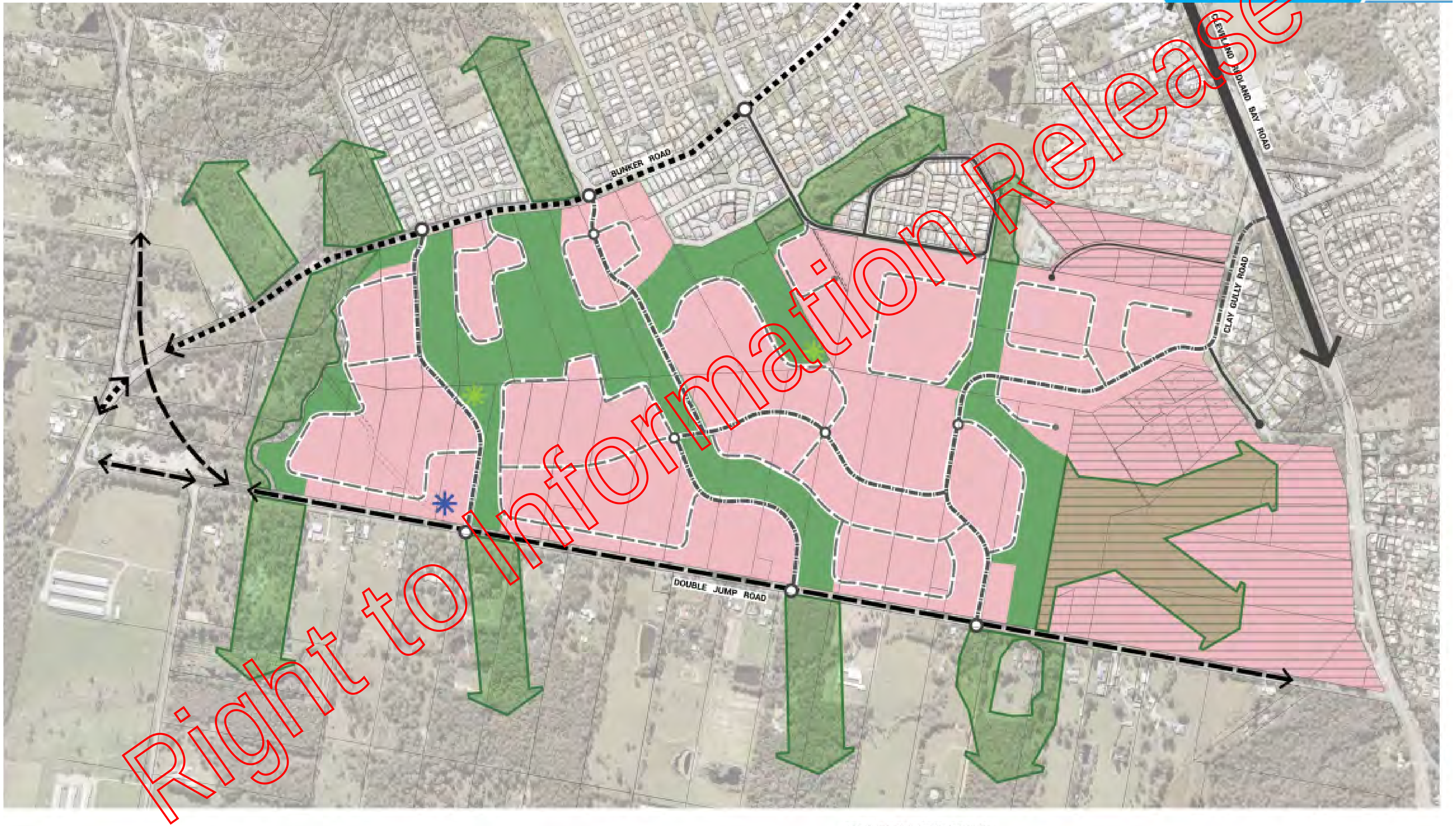
Structure Plan process

MAKE A DIFFERENCE MAKE IT COUNT

- Draft City Plan
  - Emerging Community Zone code identifies that a structure plan is required before development can proceed in the zone (but not before an application can be made)
  - Emerging Community Zone code and planning scheme policy details the work that must be undertaken to underpin a Structure Plan

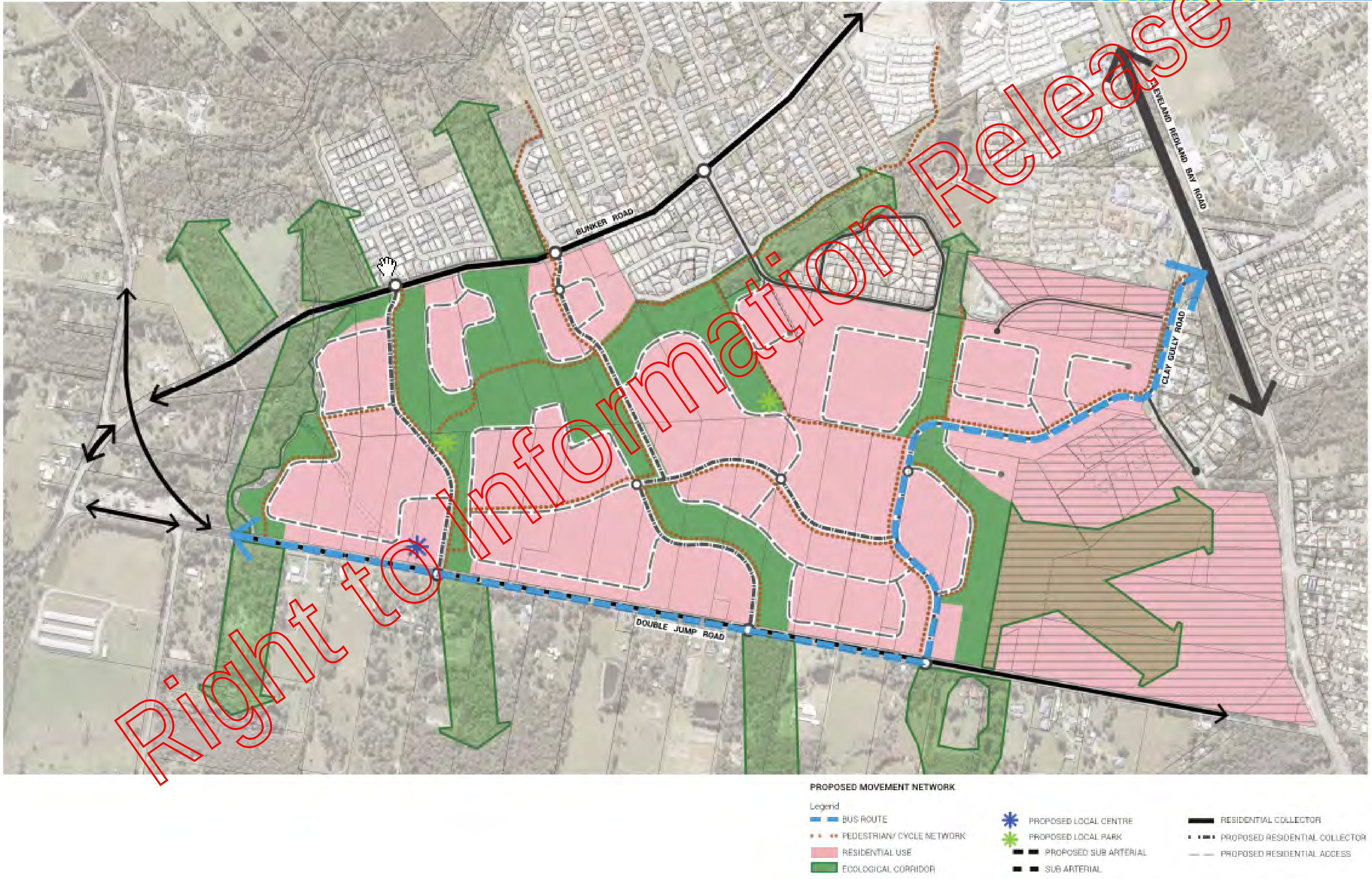
Note: Workshop presentations and
discussions are confidential

Structure Plan

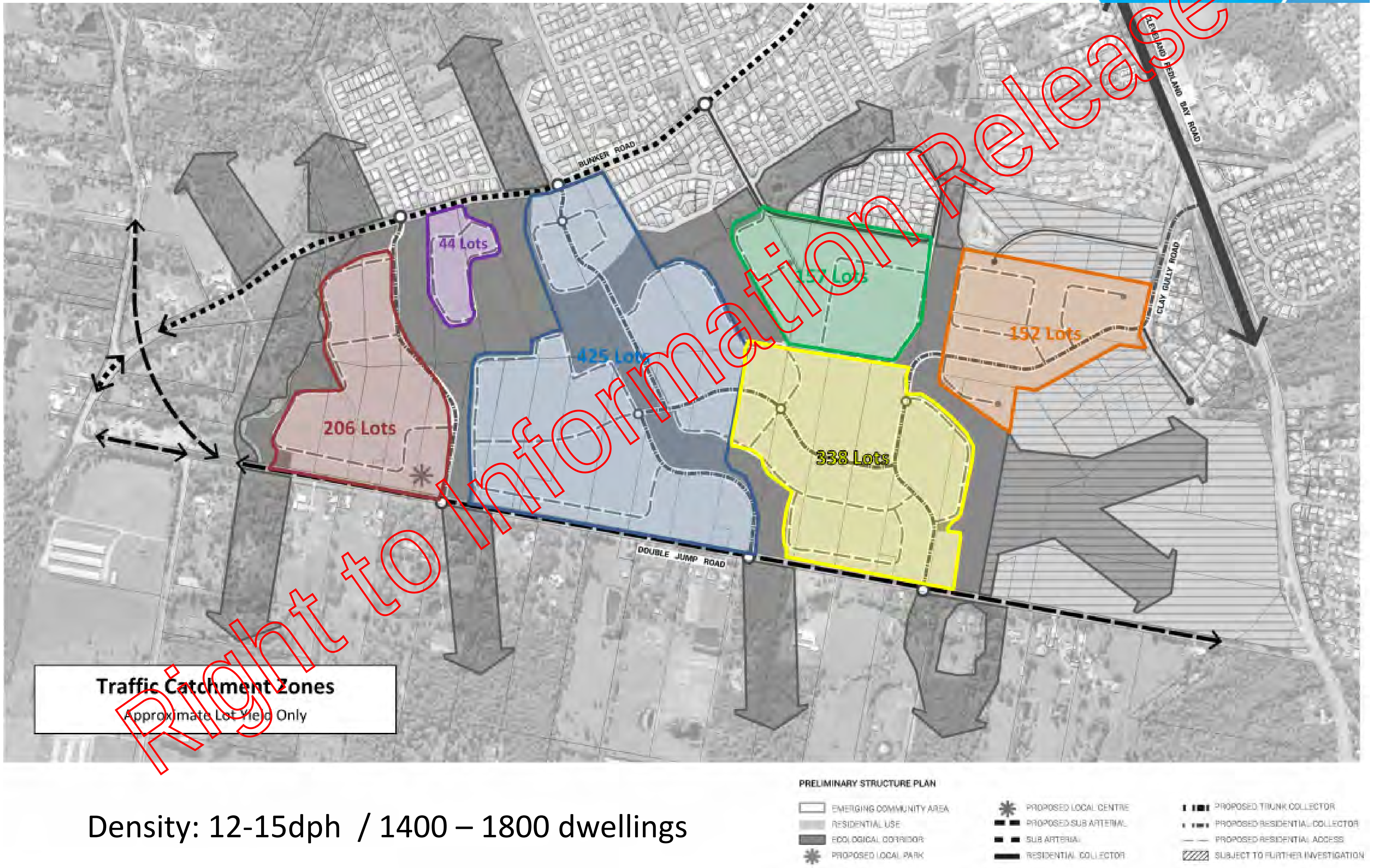


Structure Plan area: 176 hectares / 62 Lots

Structure Plan



Density / Yield



Traffic Catchment Zones

Approximate Lot Yield Only

Density: 12-15dph / 1400 – 1800 dwellings

Local park / open space catchments

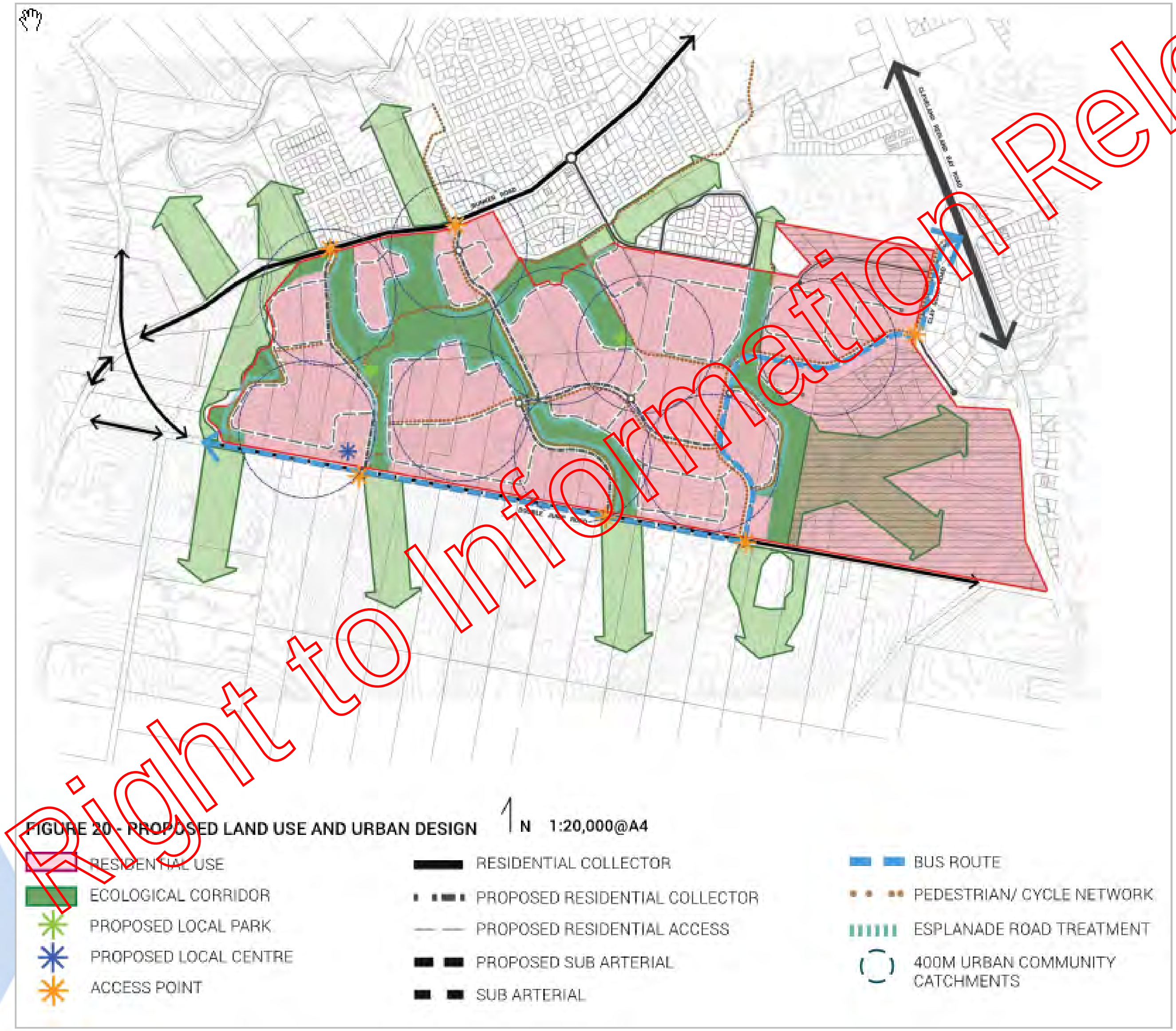


FIGURE 20 - PROPOSED LAND USE AND URBAN DESIGN

Staging

FIGURE 26 - PROPOSED PHASING

N 1:20,000@A4

[- - -] INDICATIVE STAGING BOUNDARIES

Corridors and fauna crossings

FIGURE 27 - PROPOSED HABITAT CORRIDORS N 1:20,000@A4

🟩 PROPOSED HABITAT CORRIDORS

◌ FAUNA CROSSING POINTS

Corridor widths 60m+

Poultry Industry

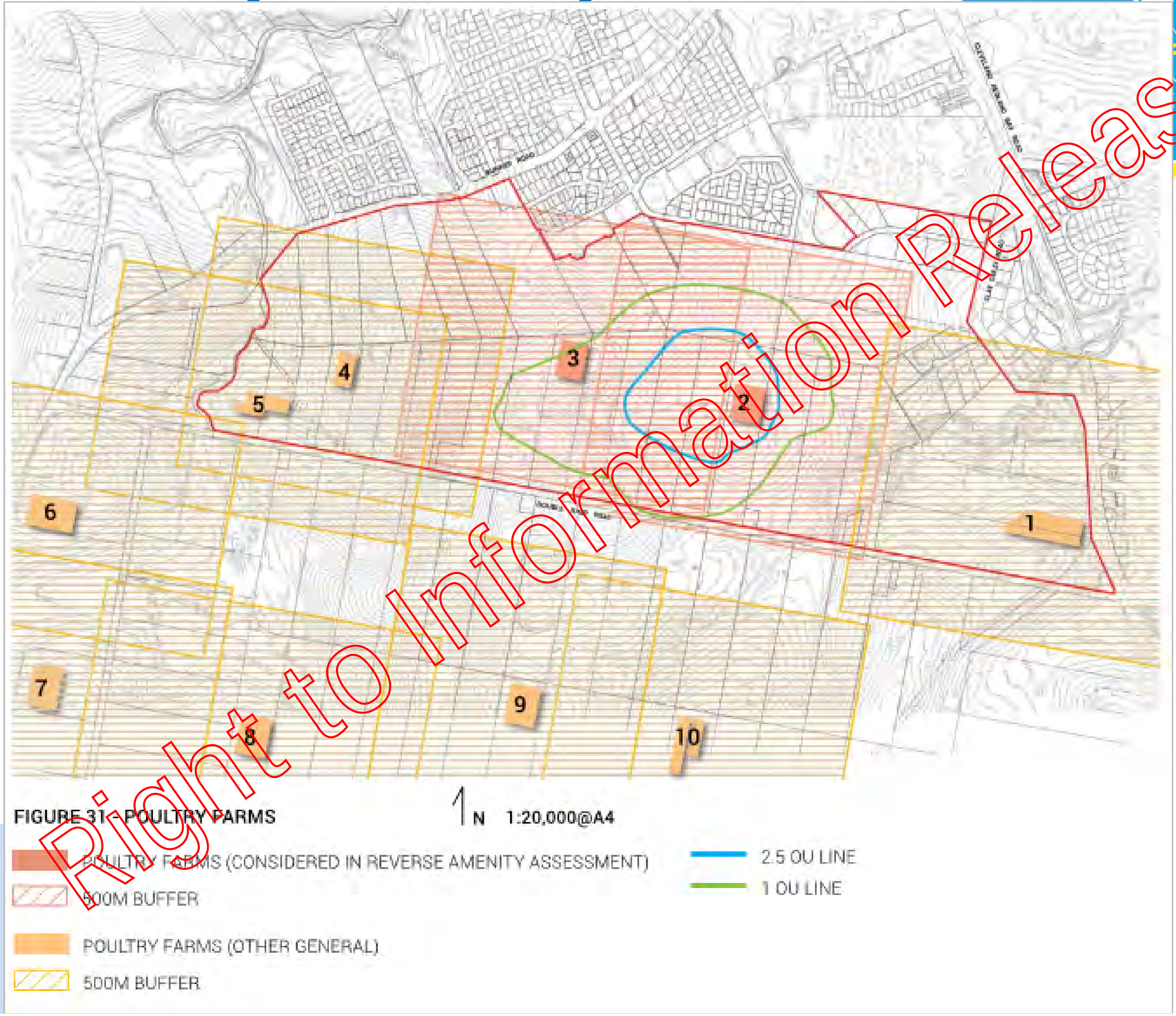


FIGURE 31 - POULTRY FARMS

Supporting Technical Information

- Environmental Advice
- Traffic Impact Assessment
- Engineering and Infrastructure Report

Note: Workshop presentations and discussions are confidential

What does this all mean?

- The proposed development
  - Planning Assessment and State officers will consider the structure plan in the context of the proposed development
  - Officers will present to councillors again following a detailed review of the structure plan, with recommendations on whether further work/advice may be necessary
  - The application is called in, therefore officers will bring the recommendation to a separate workshop for decision
- The Structure Plan – options:
  - Consider adopting the Structure Plan, pending the officer detailed review and recommendations:
    - Council could decide it is happy for Ausbuild to undertake the consultation and no further consultation is necessary
    - Council could decide to undertake a separate consultation on the Structure Plan prepared by Ausbuild
  - Council could seek to undertake a separate Council-led structure plan process

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MAKE IT
COUNT

Questions?

THANK YOU

Note: Workshop presentations and
discussions are confidential

Jill Driscoll

From:	Janice Johnston
Sent:	Monday, 30 October 2017 1:31 PM
To:	Janice Johnston
Subject:	FW: ROL005912 Clay Gully Road subdivision - officer advice following applicant presentation

Janice Johnston
Senior Planner - Strategic Planning
Redland City Council
Ph. 3829 8971

From: Janice Johnston
Sent: Tuesday, 18 July 2017 11:39 AM
To: Janice Johnston
Subject: FW: ROL005912 Clay Gully Road subdivision - officer advice following applicant presentation

Met with Steve, David and Emma on 18 July.

Agreed that strategic would stay involved in the current assessment of Ausbuild's application, but would wait and see what Fitini supply in terms of a structure plan before we go ahead and do our own (given it sounds like fitini are doing a very thorough investigation so no point us doing the same thing concurrently)

Janice Johnston
Senior Planner - Strategic Planning
Redland City Council
Ph. 3829 8971

From: Stephen Hill
Sent: Wednesday, 5 July 2017 11:31 AM
To: Emma Martin
Cc: Janice Johnston
Subject: RE: ROL005912 Clay Gully Road subdivision - officer advice following applicant presentation

Hi Emma

Just to let you know the proposed PMO project and associated budget to undertake a structure plan over the Victoria Point LDA this financial year has been approved. Over the next few weeks we will need to finalise a detailed project plan and work through how and when we commence this work and how and if we can best integrate this work with the various developer lead structure plans currently being prepared for this area.

Steve

Stephen Hill
Acting Manager City Planning and Assessment
Redland City Council
Cnr Bloomfield and Middle Streets
PO Box 21 | Cleveland Qld 4163
☎ 07 3829 8232 Mobile 0417617097
✉ Stephen.hill@redland.qld.gov.au

From: Emma Martin
Sent: Thursday, 29 June 2017 4:10 PM

To: Jill Driscoll
Subject: FW: ROL005912 Clay Gully Road subdivision - officer advice following applicant presentation

Kind regards

Emma Martin
A/Principal Planner
City Planning & Assessment
☎ (07) 3829 8556

From: Emma Martin
Sent: Wednesday, 21 June 2017 5:21 PM
To: Cr Lance Hewlett
Cc: Kim Peeti; Louise Rusan; David Jeanes; Andrew Veres; Andrew Chesterman
Subject: RE: ROL005912 Clay Gully Road subdivision - officer advice following applicant presentation

Dear Councillor,

[redacted]

4. **RCC led consultation on the structure plan prior to making a decision**
   It is my recollection that David Jeanes was referring to consultation that Council would undertake prior to adopting a structure plan not necessarily prior to making a decision on this subdivision. Regardless of this, it's important to clarify that even if the subdivision is approved the structure plan would not be an approved plan, it would not therefore apply over the development area. It has been prepared to demonstrate to Council that the proposal is appropriate and orderly development, that the necessary infrastructure upgrades have been identified and planned for and that the development does not prejudice the appropriate and orderly development of adjoining land. On this basis I do not think it is incumbent on Council to undertake community consultation in order to be in a position to make a decision, however Councillors may wish to. It is important to remember that even if Council does not undertake consultation on the structure plan prior to making a decision on this application, it still has the opportunity to consult the community prior to a structure plan being formally adopted.

If you have any other questions on this application or the Fiteni application at Double Jump and Bunker roads (ROL006166) please let me know.

Kind regards

Emma Martin
A/Principal Planner
City Planning & Assessment

Irrelevant Information

☎ (07) 3829 8556

From: Cr Lance Hewlett
Sent: Tuesday, 20 June 2017 11:42 AM
To: Emma Martin
Cc: Kim Peeti; Louise Rusan; David Jeanes; Andrew Veres
Subject: Clay Gully

HI Emma,

[redacted] Also, we were advised by David Jeanes in a previous workshop that there would be extensive Council run consultation of the entire structure plan. I assume this will be undertaken in due course as the community strongly expects it, especially given the recent purchase of the truck business and adjoining chook farm for a large over 50's resort. It's a large area and needs to be done with community involvement, in my opinion. Thank you.

Kind Regards,

Cr Lance Hewlett
Councillor, Division 4
Victoria Point and Coochiemudlo Island
Redland City Council |
Cnr Middle and Bloomfield Streets, Cleveland QLD 4163 |
PO Box 21, Cleveland QLD 4163 |
Phone: (07) 3829-8603 | Mobile: 0421 880 371 |
Email: Lance.Hewlett@redland.qld.gov.au | Web: www.redland.qld.gov.au
https://www.facebook.com/lance.hewlett

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Proposed Major Amendments to the Redland City Plan

Councillor Workshop: 8 May 2018

Redland
CITY COUNCIL

Conflict of Interests

- For the purposes of this discussion, Councillors are reminded of their obligations in relation to any conflicts of interests (material or perceived) pursuant to the *Local Government Act 2009*.

Agenda

MAKE A DIFFERENCE MAKE IT COUNT

- Review Process
- Major Amendment Process
- Amendment List – Sources
- Potential/Future Major Amendments
- Proposed Amendment Content
- Questions

Review Process

We are here

Councillor briefing outlining proposed major amendments to the draft City Plan

Councillors are given opportunity to nominate additional amendments for consideration

Council Report confirms the proposed scope and sequence of major amendments proposed to be undertaken in the 2018/2019 financial year

Major Amendment Process

Key steps outlined in the 'Minister's Guidelines and Rules':

1. Decide to make an amendment and notify the chief executive of the *Planning Act 2016*
2. Prepare amendment
3. State Interest Review (60 days)
4. Public Consultation (at least 20 days)
5. Review of submissions + preparation of consultation report
6. Notice to Minister requesting approval to adopt
7. Adoption of amendment package+ formal gazettal activities

Estimated minimum timeframe to complete a major amendment is 6-12months.

Proposed Major Amendment Packages 2018/2019

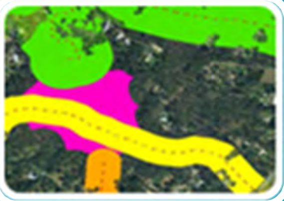
- A series of separate but concurrent amendment packages are proposed to be undertaken in 2018/2019

- Why? To ensure the amendment packages are manageable, transparent and readily understood by the community and avoid potential delays if one package is delayed

Proposed Major Amendment Packages 2018/2019

General Major Amendment Package – <u>the focus of this briefing</u>

- Addresses matters raised during Draft City Plan workshops, Councillor One-on-One meetings, Council officers and external sources (e.g. landowners and the Regional Plan)
- Will also address matters such as development in the canal and lakeside estates, in accordance with the Council resolution on 21 February 2018 (Item 12.2.7)



Wildlife Corridor Plan Package

- Incorporate new provisions to reflect key outcomes espoused in the Wildlife Connections Action Plan 2018 – 2023 in accordance with the Council resolution on 21 February 2018 (Item 12.2.5)
- The package will also incorporate a number of refinements to the Environmental Significance Overlay and the introduction of a significant tree schedule

Victoria Point LDA Structure Plan Package

- Finalisation of a structure plan
- Amendments to incorporate the structure plan into the City Plan
- To be delivered in accordance with the Council resolution on 21 March 2018 (Item 11.2.4)

Local European Heritage Package

- Proposed inclusion of the first tranche of locally significant privately owned heritage properties into the City Plan Heritage Overlay
- Contingent on 18/19 budget allocation for associated incentives package

Pages 42 through 68 redacted for the following reasons:
- -
Irrelevant Information

Recommendations

- Support undertaking a series of separate but concurrent amendment packages:
  - General Major Amendment Package
  - Wildlife Corridor Plan Package
  - Victoria Point LDA Structure Plan Package
  - European Heritage Package (subject to budget approval)
- Generally support the proposed content of the General Major Amendment Package, as contained in this presentation, subject to the below items
- Councillors, within two weeks from the date of this briefing, to submit any other proposed major amendments to the draft City Plan
- Finalise a Council report confirming the scope and contents of the four major amendment packages proposed to be undertaken following adoption of the new City Plan

Questions?

2017 – 2018

Group Plan

City Planning and Assessment

The Operational Plan activities we are a Lead on				How we measure success	
17/18 Significant Activity	**What CPA will deliver**	**Category ***	**Accountable Position**	**Measure/Milestone**	**Target**
2. GREEN LIVING **Our green living choices will improve our quality of life and our children's lives, through our sustainable and energy efficient use of resources, transport and infrastructure, and our well informed responses to risks such as climate change.**					
2.5.1 **Deliver transport planning for the city.**	a) Deliver transport planning activities in the short term under the existing Redlands Transport Plan 2016. b) Develop a new transport plan to replace the existing plan. *Group Partners - CET, CI, CorpS, CS, ESMP, IM*	a) Service Delivery b) Transformation Portfolio *PMO - Redlands Transport Plan Project (71060)* *Strategic Priority – Transport*	Group Manager, City Planning & Assessment	To be finalised pending recruitment of principal transport planner	
5. WISE PLANNING & DESIGN **We will carefully manage population pressures and use land sustainably while advocating and taking steps to determine the limits of growth and carrying capacity on a local and national basis, recognising environmental sensitivities and the distinctive character, heritage and atmosphere of local communities. A well-planned network of urban, rural and bushland areas and responsive infrastructure and transport systems will support strong, healthy communities.**					
5.1.1 **Implement the Local Government Infrastructure Plan.**	a) Ensure that infrastructure necessary to support growth in the city is provided through the development assessment process and capital works program.	a) Service Delivery	Principal Adviser Infrastructure Planning and Charging	Subject to State approval and timing of City Plan ensure that the LGIP is integrated into Asset and Service Management Plans and the Capital works program.	Q3-4
5.1.3 **Commence the Redland City Plan.**	a) Undertake a major amendment following commencement. b) Undertake periodic reviews. *Group Partner - ESMP*	a) Service Delivery b) Service Delivery	Service Manager Strategic Planning	Subject to State approval and timing of City Plan: Finalise drafting of first major amendment package Obtain State Government endorsement of the major amendment package and commence public notification Adoption/commencement of major amendment package and finalise scope of next amendment package	 Q2 Q3 Q4
5.2.3 **Plan for future use of surplus commonwealth land at Birkdale.**	a) Determine preferred land use/s for the site. *Group Partner - RIC*	a) Service Delivery/Transformational Portfolio *PMO – Birkdale Commonwealth Land Review (30562)*	Service Manager Strategic Planning	Subject to the Federal Government selling Council the land: Commence technical planning investigations of the site Finalise draft land use plan for community consultation Undertake community consultation and finalise report on preferred future land use	 Q2 Q3 Q4
5.3.1 **Maintain effective systems and processes that underpin quality, timely decision making for development applications.**	a) Implement the new Redland City Plan and State Planning Act. b) Amend systems and processes as required to ensure effective implementation of planning instruments. *Group Partner - IM*	a) Service Delivery b) Service Delivery	a) Service Manager Engineering and Environment and Planning Assessment b) Principal Adviser Business Planning and Improvement	a) Develop and implement training package in conjunction with City Plan drafting team, subject to State approval and timing of City Plan b1) Amend P&R to accommodate new City Plan requirements b2) Refine Online Lodgement System to i) accept new application types ii) integrate to existing systems iii) take online payment	Q2 Q2 Q2 Q3 Q3

***Categories include Infrastructure Portfolio, Transformation Portfolio, Service Delivery and Strategic Priorities**

2017 – 2018

Group Plan

City Planning and Assessment

The Operational Plan activities that we contribute to as a Group Partner

	Significant Activity	How we contribute
1.1.1	Manage Council owned water bodies for improved environmental outcomes.	Implement environmental outcomes as required under City Plan
1.2.1	Implement the Natural Environment Policy	Ensure City Plan is continually updated to reflect latest environmental data consistent with the Policy
3.3.1	Develop a coastal adaptation strategic plan.	Continue to participate in working groups & provide land use planning advice in the preparation of the coastal adaptation strategic plan
4.1.3	Update Council's Aboriginal and Torres Strait Island Community Policy and Guidelines.	Participate as required in working groups and provide land use planning advice in the preparation of the ATSIC policy and guidelines
4.3.2	Plan and deliver commitments under the ILUA in partnership with QYAC.	Continue to provide land use planning advice as required to deliver commitments under the ILUA agreement
5.1.2	Implement the Netserv Plan.	Support the Netserv plan through the development assessment process
5.2.1	Coordinate a centres master planning and place making program.	Continue to participate in working groups and provide land use planning advice on the revitalisation of Cleveland and place making initiatives
5.2.2	Develop master plan for Redland Aquatic Redevelopment.	Continue to provide land use planning and development assessment advice as the project progresses through stages
5.4.2	Plan and develop cross-boundary transport and infrastructure priorities.	New transport planner to participate in cross boundary transport working group and through development of new Transport Plan identify the transport infrastructure priorities for the City.
6.3.1	Support economic transition for North Stradbroke Island (NSI).	Continue to participate in the NSI Transition Strategy working group and provide land use planning advice on projects where CP&A is an identified project partner.
6.4.1	Develop strategic opportunities for Redland City Council land holdings.	Provide land use planning and development assessment advice as required.
6.6.1	Facilitate process with Economic Development Queensland.	Continue to assist and support ESMP group in facilitating process with Economic Development Queensland.
8.1.1	Transform Council's systems and processes.	Business Intelligence use, improve electronic communication with customers, reduce printing
8.1.2	Improve Council's e-service capability.	Refine the Online Lodgement System for development applications; refine website content
8.2.1	Optimise Redland City Council's asset management governance.	Ensure CPA activities align with the new assess management framework
8.4.3	Align the organisation to meet changing operational requirements.	Actively engage in Leadership programs, meeting and activities as opportunities arise.
8.4.4	Drive innovation and improvement through capable leadership.	On-going review and refinement of work processes to identify opportunities for improvements
8.4.5	Improve organisational performance through employee feedback.	Implement Culture Survey outcomes
8.4.6	Deliver a healthy and safe Redland City Council environment.	Deliver Safety Topic Talks and continue to undertake workplace safety inspections
8.5.1	Review Council's community engagement model and framework.	Use the model for all CPA community engagement activities

2017 – 2018

Group Plan

City Planning and Assessment

SERVICE DELIVERY *(External/Internal services delivered by CPA)*				**How we measure success**	
ID	**Service Description (WHAT)**	**Service Initiative (HOW)**	**Accountable Position**	**Service Measure**	**Service Target**
1	Development application processing	[redacted]	a) Group Leadership Team b) Group Leadership Team c) Principal Adviser Infrastructure Planning and Charging d) Group Leadership Team	% applications decided within legislative timeframes	≥90%
2	Respond to customer requests and enquiries	[redacted]	Group Leadership Team	% enquiries resolved within 5 days	90%
3	Business systems and process improvements	[redacted]	Principal Adviser Business Planning & Improvement	a) Amendments completed b) Intranet site updated c) Special reports created d) Fees and charges review complete	a) Amendments completed b) Intranet site updated c) Special reports created d) Fees and charges review complete
4	Planning for future land use and infrastructure requirements within the City	[redacted] f) Complete a structure plan for the Victoria Point Local Development area, where neccessary [redacted]	e) Service Manager Strategic Planning f) Service Manager Strategic Planning g) Service Manager Strategic Planning h) Service Manager Strategic Planning i) Service Manager Strategic Planning j) Service Manager Strategic Planning k) Service Manager Strategic Planning l) Service Manager Strategic Planning m) Principal Adviser Infrastructure Planning and Charging	a) Amendment program commenced b) Advice provided in accordance with customer service charter c) Review completed and Council resolution made d) Guidelines finalised e) Meetings attended and required submissions made f) Structure plan completed g) Program established h) Represent Council as required i) Recommendations implemented	a) Amendment program commenced b) Advice provided in accordance with customer service charter c) Review completed and Council resolution made d) Guidelines finalised e) Meetings attended and required submissions made f) Structure plan completed g) Program established h) Represent Council as required i) Recommendations implemented

				How we measure success	
ID	**Service Improvement Area (What)**	[redacted]	**Accountable Position**	**Improvement Measure**	**Improvement Target**
1	Customer and stakeholder service	[redacted]	Group Manager CP&A	Meetings held	Actions implemented
2	Fees and charges	[redacted]	Principal Adviser Business Planning & Improvement	ABC costing developed for an aspect of CP&A fees and charges	ABC developed
3	Business systems and process improvements	[redacted]	a) Principal Advisor BP&I b) Principal Adviser BP&I c) Principal Adviser BP&I d) Principal Adviser Infrastructure Planning and Charging e) Principal Adviser Infrastructure Planning and Charging f) Principal Adviser BP&I g) Principal Adviser BP&I h) Principal Adviser BP&I i) Service Manager Planning Assessment	a) Printing reduced b) Act on efficiency opportunities c) Procedures and work instructions update d) Solution procured and implemented e) Controls established f) Dashboards created g) Lean improvements identified and prioritised h) E-planning initiatives identified and implemented i) Resource folder established	a) Q4 b) Q4 c) Q4 d) Q4 e) Q4 f) Q4 g) Q4 h) Q4 i) Q1

Redland CITY COUNCIL

Irrelevant Information

PEOPLE, CAPABILITY & KNOWLEDGE *(CPA has the Right Capability to Deliver our Services - People, Structure, Skills)*	How we measure success
[redacted]	[redacted]

19.2 PAIGE PTY LTD V REDLAND CITY COUNCIL (PLANNING AND ENVIRONMENT COURT APPEAL) 2893/2020

[redacted]

Sch. 3(7)

Pages 76 through 104 redacted for the following reasons:
- -
Sch. 3(7)

[redacted]

OFFICER'S RECOMMENDATION

That Council resolves as follows:

1. To oppose the development application and the request to re-classify the koala habitat designation on the site, for the reasons generally in accordance with those identified in Attachment 2.

2. To authorise the Chief Executive Officer to finalise the reasons for refusal after consultation with the relevant experts and Counsel advice.

3. To instruct its solicitors to notify the parties that it opposes the development application, for the reasons generally in accordance with those identified in Attachment 2.

4. That Council officers and solicitors engage experts and Counsel to assist with the appeal with a view to narrowing the issues and resolve the appeal using delegated authority where appropriate.

5. That this report and attachments remain confidential until the conclusion of the appeal, subject to maintaining the confidentiality of legally privileged and commercial in confidence information.

Sch. 3(7)

[redacted]

Sch. 3(7)

Pages 107 through 114 redacted for the following reasons:

- -

Sch. 3(7)

Matter Costs

Report Print Date 3/06/2022
10:03:04 AM

Request ID (9376)

Matter Title	Matter Description	Supplier	Cost Amount
PE Appeal 39/21 Sutgold Pty Ltd -v- Redland City Council	PE Appeal 39/21 Sutgold Pty Ltd -v- Redland City Council [redacted]	[redacted]	$7,567.00

Contrary to Public Interest

Matter Costs

Report Print Date 3/06/2022
10:08:37 AM

Request ID (9370)

Matter Title	**Matter Description**	**Supplier**	**Cost Amount**
PE Appeal 40/21 Sutgold Pty Ltd -v- Redland City Council	PE Appeal 40/21 Sutgold Pty Ltd -v- Redland City Council [redacted]	[redacted]	$23,117.00

Contrary to Public Interest

Matter Costs

Report Print Date 3/06/2022
10:16:00 AM

Request ID (8789)

Matter Title	Matter Description	Supplier	Cost Amount
PE Appeal 566 of 2020 Clay Gully Pty Ltd v RCC	[redacted] CLAY GULLY PTY LTD ACN 627 052 224 of c/- Cooper Grace Ward Lawyers, Level 21, 400 George Street, Brisbane in the State of Queensland, appeals to the Planning and Environment Court in Brisbane under section 229 and Schedule 1, Table 1, Item 1 of the Planning Act 2016 (Planning Act) against the Respondent's deemed refusal of a development application (Council reference ROL005912) for a development permit for a reconfiguration of a lot by standard format plan (3 into 289 lots over 7 stages, new road and park) (Development Application) made under the Sustainable Planning Act 2009 (SPA) in respect of land situated at 39 Brendan Way, 21 to 29 and 31 Clay Gully Road, Victoria Point in the State of Queensland and more particularly described as Lot 1 on RP72635, Lot 4 on RP57455 and Lot 1 on RP95513 (Land)	[redacted]	$58,500.75

Contrary to Public Interest

Matter Costs

Report Print Date 3/06/2022
10:11:08 AM

Request ID (9001)

Matter Title	Matter Description	Supplier	Cost Amount
PE Appeal 1612 of 2020 Sutgold Pty Ltd v Redland City Council	PE Appeal 1612 of 2020 Sutgold Pty Ltd v Redland City Council [redacted]	[redacted]	$175,888.60

Matter Costs

Report Print Date 3/06/2022
10:14:18 AM

Request ID (9243)

Matter Title	**Matter Description**	**Supplier**	**Cost Amount**
PE Appeal 2893/20 Paige Pty Ltd -V- Redland City Council	[redacted] PE appeal 2893/20 PAIGE PTY LTD c/- HWL Ebsworth Lawyers, Level 19, 480 Queen Street, Brisbane in the State of Queensland appeals to the Planning and Environment Court at Brisbane at its next sittings, against the Respondent's deemed refusal of an application for a Development Permit for Reconfiguring a Lot - 1 into 23 Lots and Road (Application) on land located at 152-156 Bunker Road, Victoria Point in the State of Queensland and more particularly described as Lot 23 on RP86773 (Land).		$43,781.30

Contrary to Public Interest

Matter Costs

Report Print Date 3/06/2022
10:05:27 AM

Request ID (8577)

Matter Title	Matter Description	Supplier	Cost Amount
PE Appeal 3829 of 2019 Sutgold Pty Ltd -v- Redland City Council	Sutgold Pty Ltd -v- Redland City Council Appeal No. 3829 of 2019 - 314618 [redacted] Appeal against refusal of development permit for reconfiguration over land located at 72, 74, 78, 80 & 82 Double Jump Road.	[redacted]	$234,047.50

Contrary to Public Interest

Matter Costs

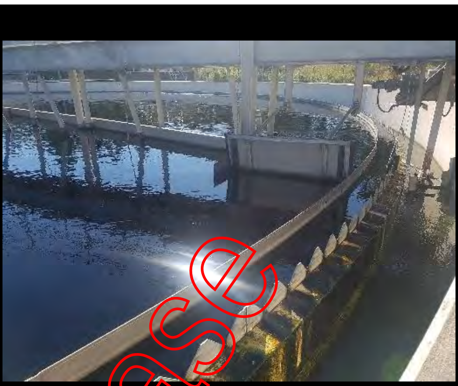
Report Print Date 3/06/2022
10:14:42 AM

Request ID (8650)

Matter Title	Matter Description	Supplier	Cost Amount
PE Appeal 4300/19 PPV Victoria Point Land Pty Ltd -V- Redland City Council	PE Appeal 4300/19 PPV Victoria Point Land Pty Ltd -V- Redland City Council [redacted] Appeal against Respondants deemed refusal of an application for a preliminary approval for a material Change of use for retirement facility & relocatable home park on land located 679-689, 687-707 & 711-719 Redland Bay road & 10 double jump road Victoria Point	[redacted]	$431,970.55

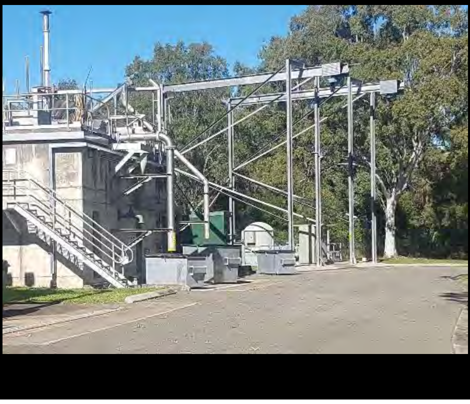
Contrary to Public Interest











Redland City Council

Victoria Point Sewage Treatment Plant

Upgrade Planning for New Developments Planning Study

Revision C

Final for RCC

July 2020

Redland City Council
Victoria Point STP – Upgrade Planning for New Developments
Planning Study

This report has been prepared solely for the benefit of Redland City Council for the Victoria Point STP – Upgrade Planning for New Developments. No liability is accepted by Tyr Group or any employee or sub-consultant of Tyr Group with respect to its use by any other person or in relation to any other project.

This disclaimer shall apply notwithstanding that the report may be made available to other persons for an application for permission or approval or to fulfil a legal requirement.

Revision	Date	Description	Prepared by	Reviewed by
C	July 2020	For RCC	David Fligelman, Ian Fisher, Ryan Schwartz	David Fligelman

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ABBREVIATIONS

AAL	Average Annual Load
ADWF	Average Dry Weather Flow
APT	Activated Primary Tank
BNR	Biological Nutrient Removal
BUA	Beneficial Use Approach
COD	Chemical Oxygen Demand
DES	Department of Environment and Science
DO	Dissolved Oxygen
EBPR	Excess Biological Phosphorus Removal
EP	Equivalent Population
EoW	End of Waste Code
IDM	Infrastructure Demand Model
MLSS	Mixed Liquor Suspended Solids
MML	Maximum Monthly Load
NPC	Net Present Cost
PDWF	Peak Dry Weather Flow
PST	Primary Sedimentation Tank
PWWF	Peak Wet Weather Flow
RAS	Return Activated Sludge
RBCOD	Readily Biodegradable COD
rDON	Dissolved Organic Nitrogen
SBCOD	Slowly Biodegradable COD
STP	Sewage Treatment Plant
SRT	Sludge Retention Time (Sludge Age)
SOUR	Specific Oxygen Uptake Rate
TP	Total Phosphorus
TSS	Total Suspended Solids
VFA	Volatile Fatty Acid
VS	Volatile Solids
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WRR	Waste Reduction and Recycling Act
WWTP	Wastewater Treatment Plant

1 EXECUTIVE SUMMARY

1.1 BACKGROUND AND OBJECTIVES

Council has received development applications that cover the majority of land in the SW Victoria Point local plan area. As a result, Council has needed to prioritise and bring forward detailed land use and infrastructure planning for the local plan area ahead of the City Plan and LGIP timeframe of post 2027. Two proposed developments in the catchment are projected to result in a connected load of 44,312 EP to Victoria Point STP in 2041, with the bulk of this additional load is predicted to be connected between 2022 and 2027. The existing Victoria Point STP operates under a very tight limit for nitrogen mass loads discharged, and the growth in sewage loads has significant implications for the nitrogen removal required to be achieved by the plant in the future.

The projected growth in loads requires the plant's previous upgrade strategy to be reassessed, including:

- Specific consideration of the process and hydraulic capacity of the existing plant, and;
- The scope, costs and timing of works required to ensure ongoing compliance with the Environmental Authority, including the Total Nitrogen Mass Load limit, under the projected increase in sewage loads through to 2041 (44,312 EP).

1.2 BASIS OF PLANNING ADOPTED

The sewage loads from the catchment are expected to be increased supplemented by two developments – Weinam Creek (to an ultimate value of 3000 EP) and South West Victoria Point (to the ultimate connected population of 4215 EP). The majority of the growth associated with these developments is expected to occur between 2022 and 2027. The planning horizon for planning has been adopted as 2041.

Based on visual inspection, the existing plant is generally in good condition. Items requiring renewal comprise:

- Removal (and replacement if required) of the acoustic covers on the oxidation ditch aerators;
- Refurbishment of structural steel and cladding of the dewatering building, and,
- Provision of a replacement sludge dewatering machines.

There have been major structural issues in the existing bioreactor. In the absence of other information, the analysis has assumed that the repairs to this structure undertaken in July 2017 will render it suitable for ongoing use throughout the planning horizon.

The sewage loads and composition applied to the study were drawn from extensive analysis of 12 years of historical operational monitoring data, and intensive monitoring of the plant influent sewage composition and plant operations in November and December 2019. This data was used to calibrate a dynamic process model of the existing plant for use in the estimation of the existing plant capacity, and the selection and concept design of the required upgrade works.

The current effluent quality criteria for the plant requires the mass load of total nitrogen to be maintained at less than 13.5 kg/d on an annual basis. Compliance with this limit requires the effluent total nitrogen concentration to be substantially reduced under the higher flows which will arise due to the new developments. Previous consultation with DES stretching back to 2002-03 has not been successful in amending this limit. Further analysis and modelling of the receiving waterway, Eprapah Creek, is currently underway to identify the potential to increase the mass of nitrogen which can be discharged from the plant. However, pending completion of this analysis, the 13.5 kgN/day limit has been applied to the upgrade planning.

1.3 KEY FINDINGS AND RECOMMENDATIONS

The prevailing capacity of Victoria Point STP is limited to 38,300 EP by the ability of the secondary clarifiers to treat 5 x ADWF. The existing plant's ability to maintain compliance with the Total Nitrogen Mass Load Limit will be compromised at a similar load (38,700 EP).

Based on this analysis, the plant requires upgrades three process areas to treat the additional 7215 EP load from the South West Victoria Point and Weinam Creek developments. Concept designs for the upgrade works required in each of these process areas were developed as a part of this study. The scope, required timing and estimated capital costs of the required upgrades is summarised in Table 1-1.

Table 1-1: Summary of Required Plant Upgrades and Staging to Service New Developments

Upgrade	Estimated Capital Cost	Required from	
Post-Anoxic / Re-Aeration Zone)	$1.289m Direct Job Cost	38,700 EP	2025
1 No. Additional Secondary Clarifier	$2.255m Direct Job Cost	38,300 EP	2024
1 No. Additional Chlorine Contact Tank	$0.296m Direct Job Cost	38,700 EP	2025
TOTAL CAPITAL COST (+/- 30% Accuracy Target)	Total Direct Job Cost (including Preliminaries, Commissioning and Handover): $4.033m Total Project Cost (including 30% Contingency): $8.512m		

The operational costs required to treat the sewage load generated by the South West Victoria Point and Weinam Creek Developments were estimated in detail. The additional electricity consumption and biosolids haulage required to treat the load dominates the additional costs. In 2041 (the planning horizon), the additional annual operating cost is $135,100 p.a. with additional sludge haulage at $65 /wet tonne, increasing to $160,400 p.a. if the rate for sludge haulage rises to $100 /wet tonne

The whole-of-life cost to treat the load from the South West Victoria Point and Weinam Creek Developments is $10.31-10.68m over 40 years, depending on the cost of biosolids management.

The works to treat sewage loads from the developments are required to be completed and in service by 2024-25. This suggests the upgrades should be undertaken under a single contract with procurement and design commencing in 2020-21.

2 BACKGROUND AND OBJECTIVES

The Victoria Point Sewage Treatment Plant (STP) was originally constructed in 1977, then upgraded to an oxidation ditch-based process in 2003. The sewage received by the plant is primarily residential in origin, with some light trade waste. The plant consistently achieves excellent nitrogen removal, with the annual median effluent total nitrogen ranging from 1.40 mg/L to 1.90 mg/L over the last five years of operation.

The existing Environmental Authority for the plant includes a stringent requirement for total nitrogen mass loads not to exceed 13.5 kgN/d on a long-term median basis. This requirement constrains the effluent total nitrogen limit to lower values as the flow to the plant increases, and there is a risk of non-compliance with this limit at the current sewage flows and effluent nitrogen performance. While issues in the initial calculation basis applied to this limit have been referred to the regulator on a number of occasions (including 2003 and 2010), Redland City Council's case to raise the limit to 21.3 kgN/d has not been accepted to date.

The projected load for 2041 is 44,312 EP based on two proposed developments in the catchment – *South West Victoria Point* and *Weinam Creek*. The bulk of this additional load is predicted to be connected between 2022 and 2027.

The loads from these developments will result in substantial exceedance of the existing plant's capacity in the near term, and prevent compliance with the existing effluent quality criteria. On this basis, Redland City Council requires the plant's previous upgrade strategy to be reassessed in detail, including specific consideration of:

- The actual sewage loads currently received by the plant (based on both long term monitoring data, and an intensive monitoring program undertaken in November-December 2019);
- Projection of the sewage loads for the two proposed developments through to a planning horizon of 2041;
- The hydraulic capacity of the existing plant;
- The process capacity of the existing plant (based on dynamic process modelling);

The development of concept designs, cost estimates, and required timing for the upgrade works required to ensure ongoing compliance with the Environmental Authority, including the Total Nitrogen Mass Load limit, under the increased loads associated the Weinam Creek and South West Victoria Point developments through to 2041.

3 BASIS OF ASSESSMENT, PLANNING, AND DESIGN

3.1 CONTRIBUTING POPULATION

Redland City Council has received development applications that cover the majority of land in the SW Victoria Point local plan area. As a result, Council has needed to prioritise and bring forward detailed land use and infrastructure planning for the local plan area ahead of the City Plan and LGIP timeframe of post 2027.

The projected contributing population to Victoria Point STP catchment is shown in Table 3-1 and Figure 3-1 overleaf. The figures shown are based on the Infrastructure Demand Model (IDM) outputs provided by Redland City Council.

The contributing population of the Weinam Creek development was originally provided to an ultimate value of 3377 EP. Based on advice from Redland City Council, this project has assumed a linear growth rate through to a reduced ultimate load of 3000 EP in 2036.

In the absence of detailed projections for the South West Victoria Point development (formerly known as Clay Gully), the projection has been developed based on connections commencing in 2022-23, and linear growth over the subsequent five years. The planning has applied an ultimate connected population of 4215 EP for this development.

It is important to note that the "ultimate" connected population, as shown in Table 3-1, does not refer to a particular year. Rather, the ultimate refers to the connected population when the catchment is "fully developed".

The planning horizon for this report has been set as 2041, corresponding to a connected sewage load of 44,312 EP, with the majority of this growth occurring between 2022 and 2030.

Table 3-1: Victoria Point STP - Projected Connected Population

Year	South West Victoria Point Development	Weinam Creek Development	Total (incl. Developments)
2020		0 EP	32,496 EP
2021		434 EP	33,374 EP
2022	0 EP	677 EP	33,992 EP
2023	843 EP	921 EP	35,453 EP
2024	1,686 EP	1,164 EP	36,914 EP
2025	2,529 EP	1,408 EP	38,375 EP
2026	3,372 EP	1651 EP	39,836 EP
2027	4,215 EP	1,839 EP	41,153 EP
2028	4,215 EP	2,027 EP	41,627 EP
2029	4,215 EP	2,216 EP	42,102 EP
2030	4,215 EP	2,404 EP	42,576 EP
2031	4,215 EP	2592 EP	43,050 EP
2032	4,215 EP	2,674 EP	43,211 EP
2033	4,215 EP	2,755 EP	43,373 EP
2034	4,215 EP	2,837 EP	43,534 EP
2035	4,215 EP	2,918 EP	43,696 EP
2036	4,215 EP	3000 EP	43,857 EP
2037	4,215 EP	3000 EP	43,948 EP
2038	4,215 EP	3000 EP	44,039 EP
2039	4,215 EP	3000 EP	44,130 EP
2040	4,215 EP	3000 EP	44,221 EP
2041	4,215 EP	3000 EP	44,312 EP
Ultimate	4,215 EP	3000 EP	51,613 EP

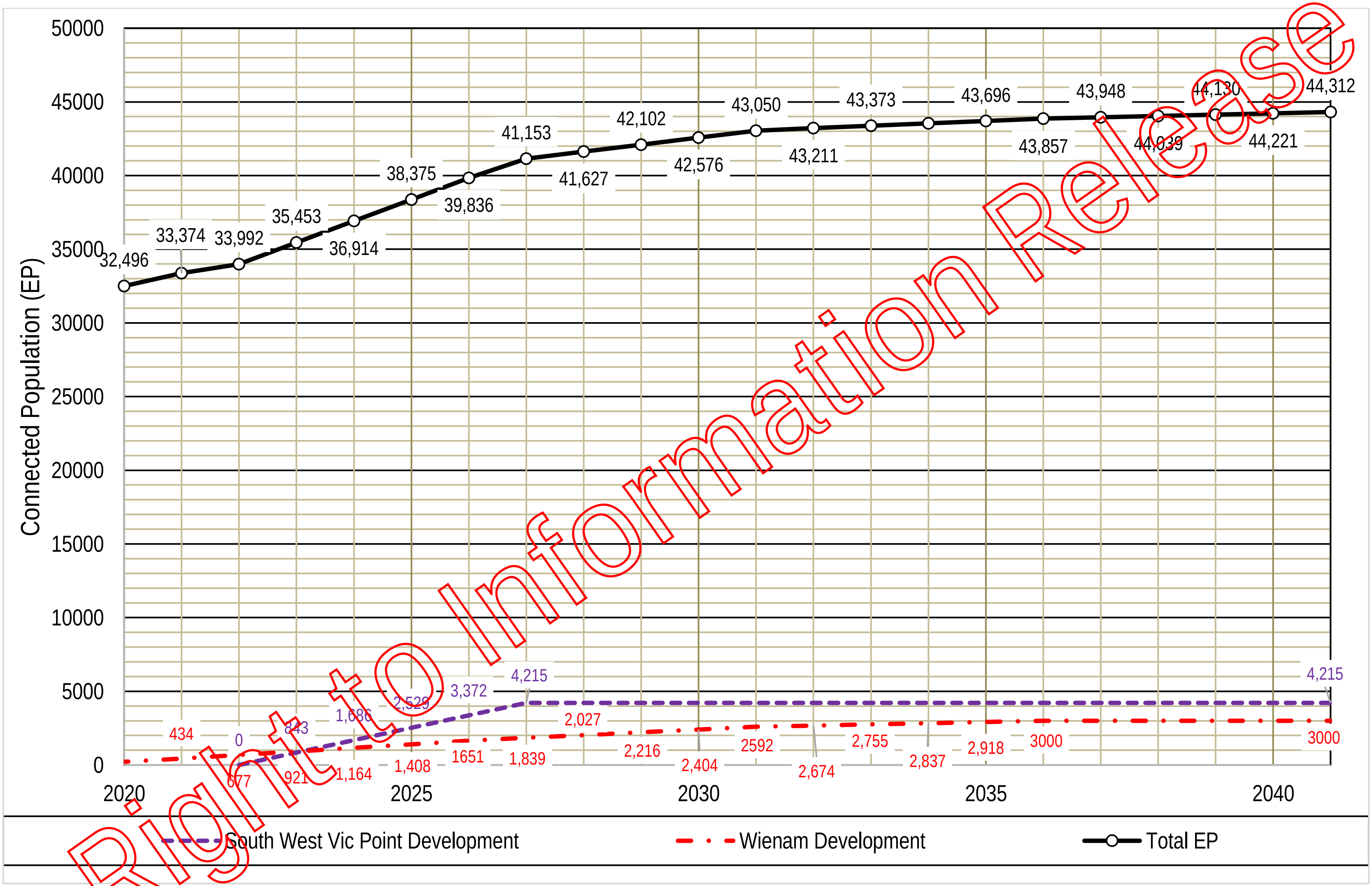

Figure 3-1: Victoria Point STP –Projected Contributing Population

3.2 INFLUENT SEWAGE FLOWS

The dry weather flows to the plant are critical to quantifying the plant loading, but additionally for Victoria Point STP, determine the maximum acceptable effluent total nitrogen (see Section 3.6.3).

The influent flows to the plant have been analysed for the period January 2007 through June 2019, and estimated on a per capita basis (using the IDM population projection) for the last six years. The following two criteria were applied to exclude wet weather days from the dataset:

Criteria 1: Exclusion of days on which the recorded rainfall exceeded 4mm, or the rainfall in the preceding 4 days exceeded 10mm. This criterion is focussed on reducing the influence of even modest levels of sustained infiltration on the analysis by excluding days immediately following relatively minor rainfall.

Criteria 2: Exclusion of days on which the recorded rainfall exceeded 1mm, or the rainfall in the preceding 4 days exceeded 50mm. This criterion is identical to that used to define a “dry weather day” in the Environmental Authority for all Redland STPs. This criterion will exclude inflow to the sewerage system more than Criteria 1, but retain more days which are influenced by the sustained infiltration which occurs after heavy rainfall.

The results of this analysis are shown in Figure 3-2 and Figure 3-3, and indicate:

- The average flow tracks very strongly with total rainfall on a 365 day rolling average basis. This indicates the impact of sustained infiltration after wet weather events on the flows to the plant.
- There does not appear to have been any substantial increase in the baseline dry weather flow to the plant over the last 10 years. That is, for a given annual rainfall, the calculated dry weather average flows do not appear to have increased when considered on a 365 day average basis.
- There is a small discrepancy between influent sewage flows and the flows discharged from the plant. This is likely due to inaccuracies with the effluent flowmeter, which is calculated from the height of flow of a weir. The overall magnitude of this error is not significant.
- The per capita dry weather sewage flows over the last four years have averaged 180 L/EP/d (Influent, Criteria 1) to 191 L/EP/d (Effluent, Criteria 1), but all of these years were below the average annual rainfall.
- The maximum recorded flows per capita during the analysis period, calculated on an annual basis, were in 2011 (212 L/EP/d Influent, 1584mm) and 2012 (216 L/EP/d Influent, 1384mm). Since then, the maximum per capita flows, were 219 L/EP/d estimated for 2013 and 2015 under Criteria 2 for the effluent. Both of these years recorded comparable (or higher) rainfall than the 2011 and 2012 years. This suggests that a moderately wet year may see a per capita flow in the order of 220 L/EP/d (calculated under Criteria 2).
- The dry weather flow calculated for the characterisation period of November 29 – December 19, 2019 was 153 L/EP/d. As the characterisation period followed on from a prolonged period of low rainfall, this is likely to represent the minimum per capita flow at Victoria point.

Based on the analysis of the data, a maximum dry weather average per capita inflow of 220 L/EP/d has been carried forward as the basis of planning. For reference, it is worth noting that the 2003 plant upgrade was based on a per capita flow of 220 L/EP/d, but the Strategic Planning Review (2009) applied a per capita flow of 190 L/EP/d increasing to 230 L/EP/d by 2025.

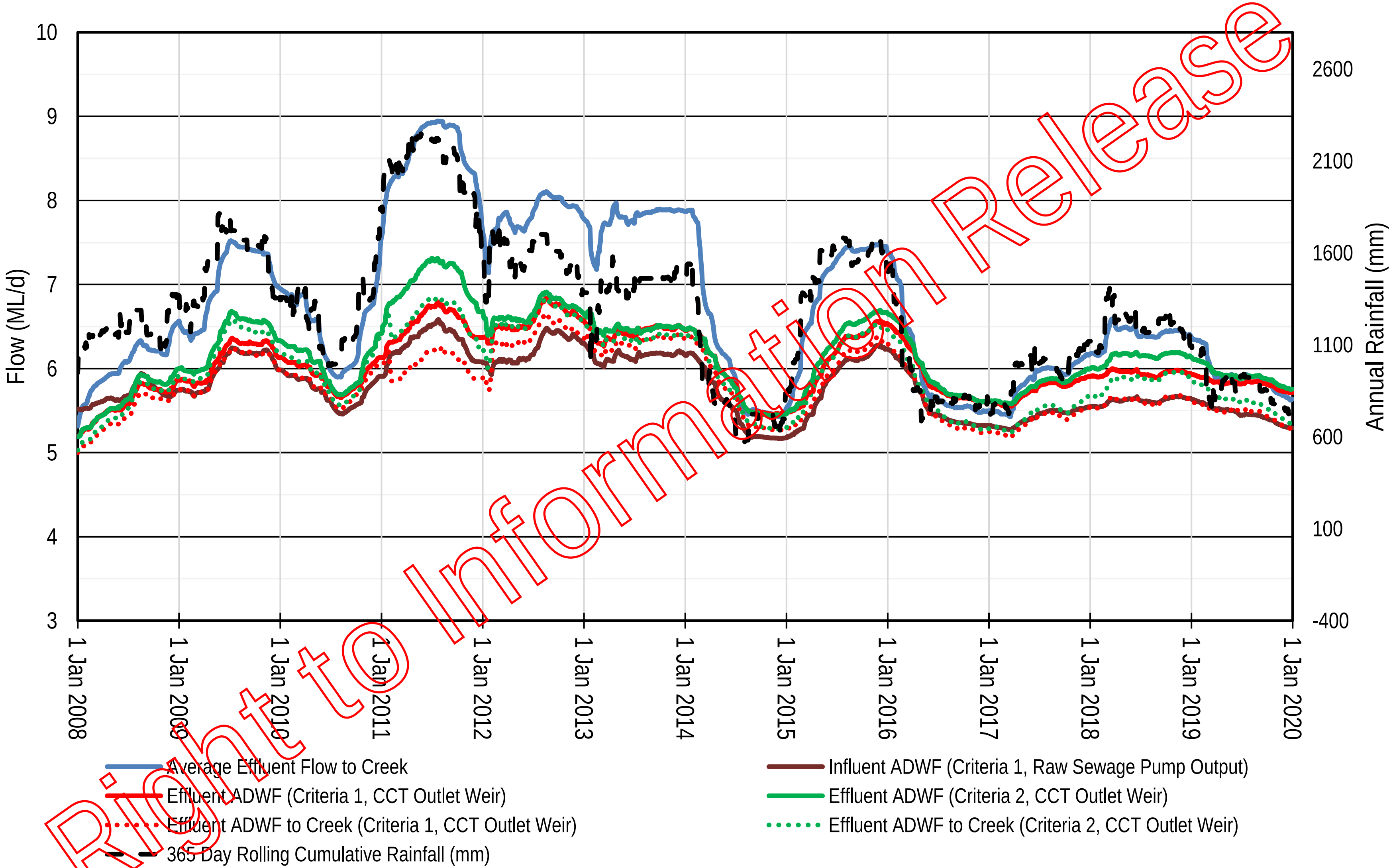

Figure 3-2: Victoria Point – 365-Day Rolling Dry Weather Average Flows, January 2007- December 2019

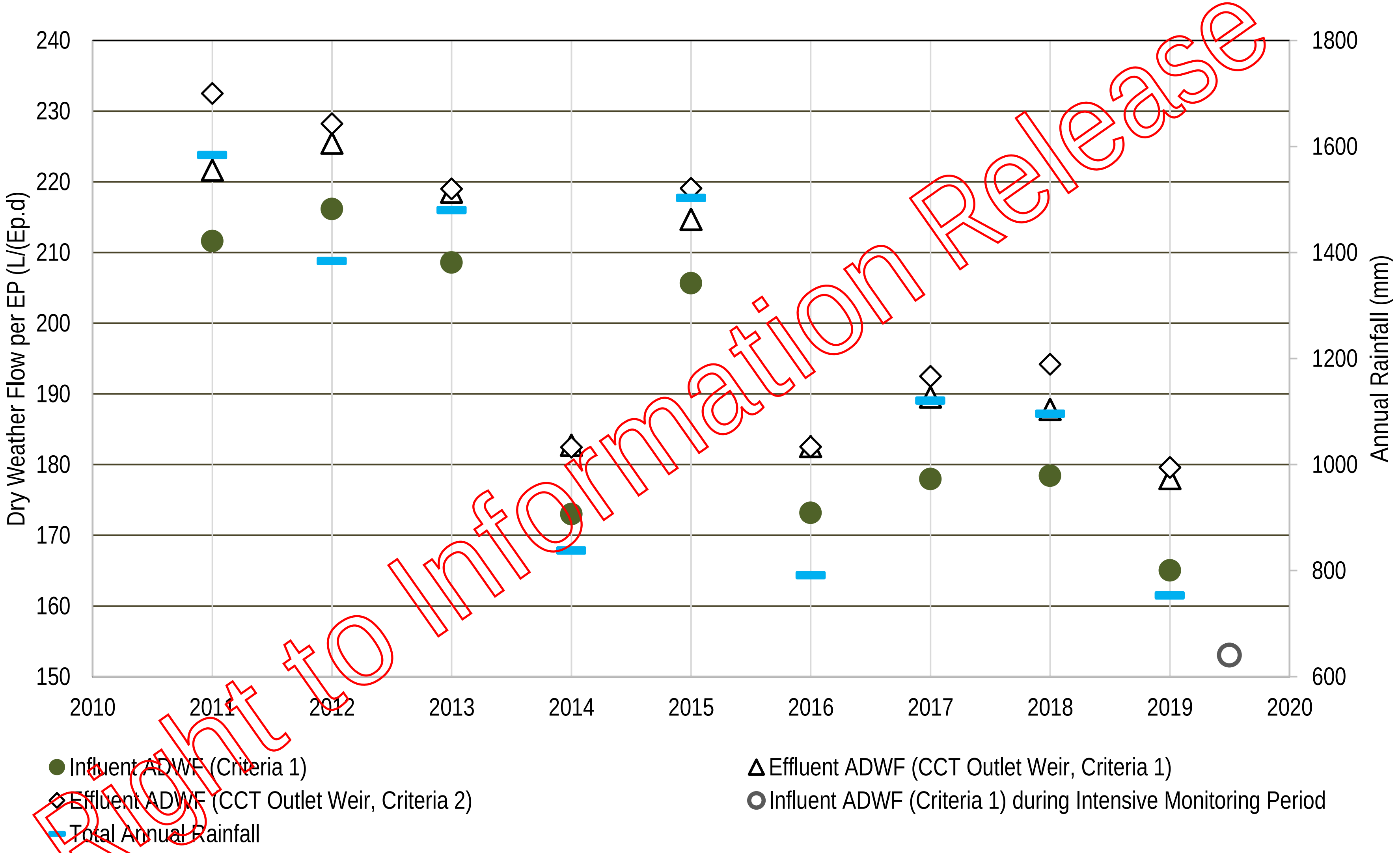

Figure 3-3: Victoria Point – Annual Dry Weather Average Per capita Flow, 2007- 2019

3.2.1 Dry Weather Diurnal Influent Sewage Flows

The typical dry weather diurnal sewage flow pattern was derived from 30-minute SCADA data drawn from the intensive monitoring period (November 27 through December 20, 2019). No filtering of this data for wet weather was required as the plant was operating under a sustained period of dry weather at this time.

Average diurnal flow patterns were derived from this data based on a 30-minute averaging are summarised in Figure 3-4. As the averaging of daily flow patterns serves to attenuate the diurnal profile (reducing the magnitude of the peaks and the troughs), a “typical” diurnal profile was derived from the SCADA data and adopted for analysis of the plant capacity. To this end, the profile from November 30, 2019, showing a diurnal peak of 1.95 x ADWF, was applied to the concept design.

The typical dry weather diurnal peaking factor recorded during the monitoring period was 1.8 x ADWF on weekdays, and 1.9 x ADWF on weekends. This ratio is typical for sewage catchments of this scale.

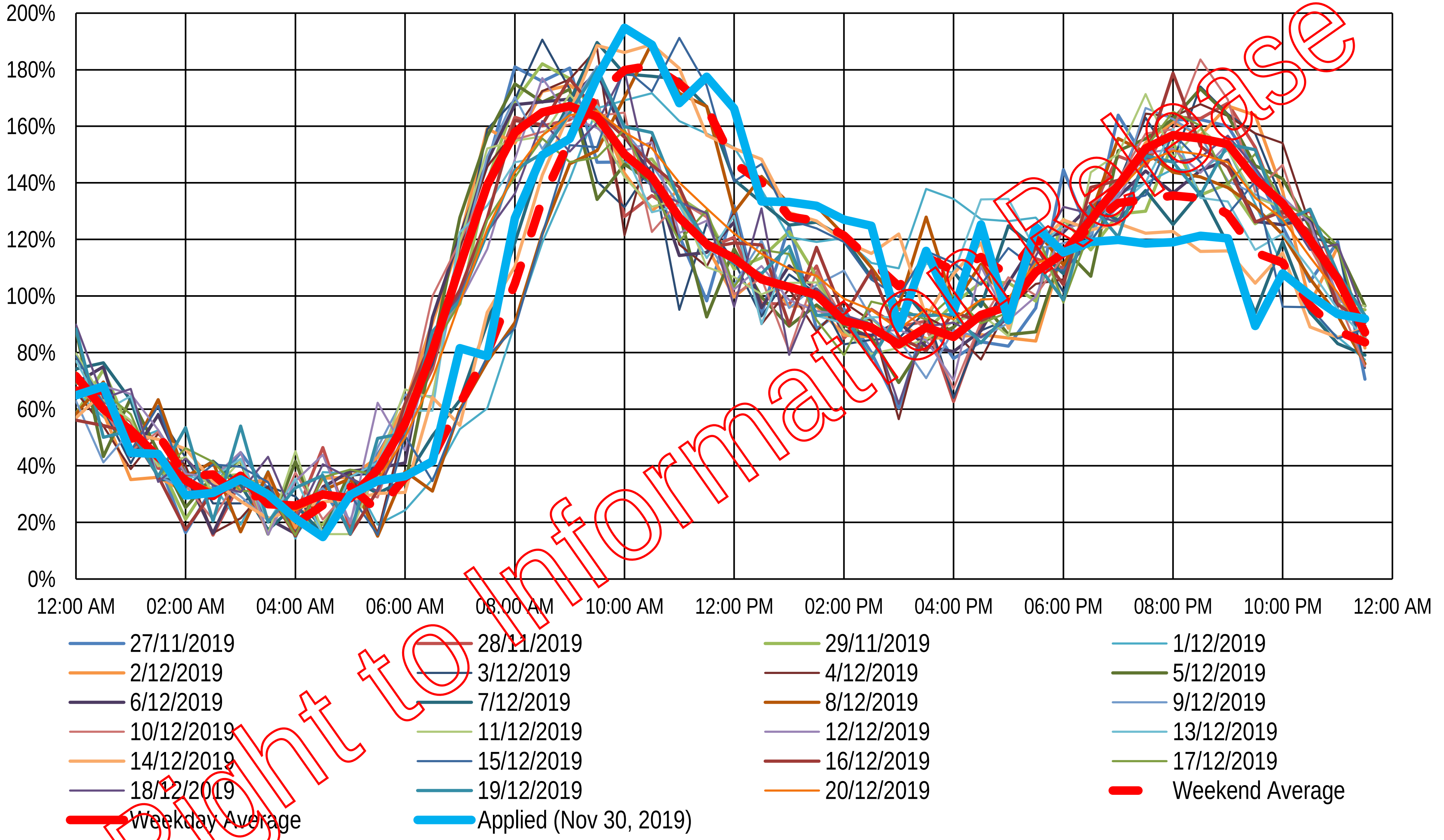

Figure 3-4: Diurnal Influent Flow Pattern – 30 Minute Average Flow, Nov 27-Dec 20, 2019, and Typical Pattern Applied to Planning

3.3 INFLUENT SEWAGE COMPOSITION

3.3.1 Available Influent Sewage Monitoring Data

The ongoing sampling and composition monitoring of Victoria Point STPs influent sewage was limited in recent years. Key limitations in the available long-term influent characterisation data included:

- Limited valid sampling events for bulk pollutants (COD, BOD_5, TKN, Total Phosphorus, Suspended Solids): There were a total of just 61 dry weather influent sampling events over the last 10 years – all since 2014. However, this data set is reduced further by inconsistencies and anomalies observed for almost all sampling events prior to October 2015. These issues, likely related to non-representative sampling, appear to have been resolved around this time, resulting in a total of 30 dry weather sampling results over the period from October 2015 through to May 2019. All but one of these 30 influent sampling events occurred in the 2015-2017 years. These results have been used to support estimates of average annual pollutant loads, but were not sufficient for estimation of the extent of variation around the average (e.g. Maximum Monthly Load, Maximum Weekly Load etc.).
- Limited valid sampling to support COD fractionation (e.g. sCOD, FFCOD, BOD_5, $sBOD_5$, TSS, VSS): While there is substantial data to support estimation of the COD fractions for periods well prior to 2014, there was little or no valid data from the last five years. The limited monitoring through to May 2019 included BOD_5 results which were inconsistent with the remainder of the results. Further, there was only one sampling result with a direct measurement of inert suspended solids.

Due to these gaps in the influent sewage composition, intensive monitoring of the plant influent sewage and operational performance was undertaken from November 28 through to December 18, 2019. This program included sampling to support estimation of the bulk pollutant load, COD fractionation, and diurnal pollutant variations. As outlined in the following sections, the intensive monitoring period provided suitable information for derivation of the influent sewage fractions, and calibration of a dynamic model of the plant's secondary treatment process. However, the results from both the long-term sampling and characterisation program are not sufficient to support accurate estimation of maximum monthly loads (relative to average annual loads). As such, the typically observed ratios of maximum monthly loads to average annual loads (MML/AAL) for Municipal STPs of comparable scale have been applied (1.18 for COD, and 1.15 for TKN, TP and ISS).

The influent parameters measured during the intensive monitoring period are summarised in Table 3-2.

Table 3-2: Intensive Monitoring Period (Nov 28-Dec 18, 2019) – Summary of Influent and Effluent Results

Date of Collection	Units	Range	Median	No. of Results
Influent Sewage – 24 Hour Composite Results				
Flow from Log (6am to 6am)	ML/d	4.7 - 5.4	5	21
pH - Field	pH units	7.33 - 8.15	7.56	8
Conductivity - Field	µS/cm	1170 - 1710	1410	8
Total Alkalinity	mg/L as $CaCO_3$	293 - 403	299	8
BOD 5 days @ 20°C	mg/L	220 - 420	280	11
BOD_5 Mass Load	kg/d	1104 - 2192	1372	11
Soluble BOD (1.2um)	mg/L	59 - 130	85	11
sBOD/BOD	ratio	0.24 - 0.46	0.29	11
BOD-Uninhibited	mg/L	300 - 400	355	6
cBOD/Total BOD	ratio	0.8 - 1.05	0.95	6
COD	mg/L	610 - 1100	790	13
COD Mass Load	kg/d	3020 - 5126	3863	13
Soluble COD (1.2um)	mg/L	200 - 300	250	13
sCOD/COD	ratio	0.24 - 0.41	0.30	13
Flocculated Soluble COD	mg/L	140 - 180	165	6
F_{bs} (at average effluent sCOD)	ratio	0.144 - 0.176	0.152	5
Total Oil & Grease	mg/L	47 - 100	65.5	4
Suspended Solids	mg/L	300 - 540	360	13
VSS/TSS	ratio	0.88 - 0.97	0.94	13
Inert Suspended Solids	mg/L	10 - 60	20	13
Calcium as Ca	mg/L	36 - 37	36.5	4
Magnesium as Mg	mg/L	17 - 21	17.5	4
Ammonia N	mg/L	49 - 80	51	13
Nitrate N (Calc)	mg/L	0.026 - 0.54	0.052	7
Nitrite+Nitrate as N	mg/L	0.026 - 0.54	0.042	8
Total Kjeldahl Nitrogen as N	mg/L	64 - 84	69	13
Total Nitrogen as N	mg/L	64 - 85	69	13
TN Mass Load	kg/d	308 - 414	334	13
Ammonia/TKN	ratio	0.66 - 0.80	0.74	12
Ortho Phosphorus as P	mg/L	4.2 - 8.1	4.6	13
Total Phosphorus as P	mg/L	6 - 11	8.6	13
TP Mass Load	kg/d	29.2 - 53.6	43.6	13

Table 3-2: Intensive Monitoring Period (Nov 28-Dec 18, 2019) – Summary of Influent and Effluent Results (continued)

Date of Collection	Units	Range	Median	No. of Results
Effluent – 24 Hour Composite Results				
pH - Field	pH unit	6.88 - 7.86	6.91	6
Total Alkalinity	mg/L as CaCO$_3$	114 - 115	115	2
BOD 5 days @ 20°C	mg/L	<5	<5	6
COD as O$_2$	mg/L	21 - 31	25.5	6
F_{us} (based on Effluent COD)		0.022- 0.040	0.027	6
Soluble COD (1.2um)	mg/L	14 - 28	20.5	6
F_{us} (based on Effluent sCOD)		0.018 - 0.032	0.023	6
Suspended Solids	mg/L	<5	<5	6
VSS	mg/L	<5	<5	6
Ammonia N	mg/L	0.02 - 0.37	0.044	6
Nitrate N (Calc)	mg/L	0.6 - 0.9	0.79	6
Nitrite+Nitrate as N	mg/L	0.61 - 0.9	0.8	6
Total Kjeldahl Nitrogen as N	mg/L	0.72 - 1.4	1.05	6
Total Nitrogen as N	mg/L	1.5 - 2.2	1.7	6
Ortho Phosphorus as P	mg/L	0.81 - 1.7	1.1	6
Total Phosphorus as P	mg/L	0.84 - 1.7	1.2	6

3.3.2 COD Fractionation

The fractions of influent COD which are biodegradable, non-biodegradable, particulate, and soluble are crucial to effective estimation of plant capacity and performance. The fractionation of the COD has been derived from the intensive monitoring period data, and where possible, validated against the available long term information.

3.3.2.1 Readily Biodegradable COD (RBCOD, F_{bs})

The readily biodegradable fraction of the COD determines the extent to which biological phosphorus removal can be achieved with a given influent sewage, and in some configurations has a bearing on the extent of denitrification. The intensive monitoring period data indicated that the readily biodegradable COD was consistently around 15% of the influent COD (range 14.4-17.5%). This gives an F_{bs} of 0.15, which is around the midpoint of the typical range for municipal sewage in South East Queensland. As no long-term records of this parameter are available, data from the monitoring period has been applied to the analysis without modification.

3.3.2.2 Unbiodegradable-Soluble COD (F_{us})

The fraction of the influent COD which is unbiodegradable and soluble (F_{us}) has been directly estimated using the influent and effluent data from the monitoring period. The F_{us} was found to be in the range of 0.02 to 0.04, with an average value of 0.03. This value is lower than the 0.05 typically observed in Australian municipal sewage.

3.3.2.3 Unbiodegradable-Particulate COD (F_{up})

Given the importance of the unbiodegradable-particulate COD fraction in determining plant capacity based on solids settling, the unbiodegradable particulate fraction of the COD (F_{up}) has been estimated through calibration of a steady-state process model to the sludge production observed within the existing secondary treatment process. This analysis is summarised in Section 3.3.4.

Due to the significant data gaps in the long-term data to inform this calibration, the F_{up} derived from the intensive monitoring period is considered to be more reliable and representative. To this end, the unbiodegradable-particulate COD fraction (F_{up})

of 0.26 derived from the intensive monitoring period has been applied to the planning. This is marginally higher than the 0.20 to 0.25 typically observed in Australian municipal sewage

3.3.2.4 Slowly Biodegradable COD which is Particulate (F_{xsp})

Influent COD which is neither unbiodegradable (F_{up} or F_{us}) nor readily biodegradable (F_{bs}) is classified as slowly biodegradable. The slowly biodegradable fraction is important in driving denitrification, and also determines the potential for fermentation to convert slowly biodegradable COD to readily biodegradable COD.

The colloidal (F_{xsc}) and particulate (F_{xsp}) slowly biodegradable COD is determined by balancing the COD fractions, and relies on measurement of soluble COD and soluble BOD. Based on the intensive monitoring period result, the F_{xsp} value derived from the data was 0.75, which is in line with the default value applied in the model.

3.3.3 Suspended Solids Load

The mass of inert suspended solids can vary substantially between catchments, and its accurate determination is vital to an accurate solids production estimate. Results for this parameter are limited in the historical influent monitoring results for the plant. However, even where influent monitoring results for inert suspended solids are available, accurate measurement often proves challenging due to:

1. Difficulties in obtaining a representative concentration of solids within sewage samples – particularly given the settling of solids in the inlet works and sewage mains in between pumping events and as a function of flow velocity.
2. The relatively low mass of inert suspended solids which are typically filtered from influent sewage in comparison to error imposed by the testing methodology (e.g. residual moisture or ash associated with filter papers). The typical reported uncertainty in measurements of total suspended solids (~5%) and volatile suspended solids (~15%) stems from these challenges.

To assist in generating the most accurate estimate of this parameter possible, the volatile and total suspended solids measured in the bioreactor have been used to calibrate the sludge production within the secondary treatment process, then compared with figures contained in the plant log.

The steady-state analysis is summarised in Section 3.3.4, and identified average inert suspended solids concentrations consistently in the range 32-35 mg/L through the periods of study. This is within the typical range for Australian municipal sewage.

3.3.4 Secondary Treatment Steady-State Model Calibration to Support Influent Characterisation

A steady-state process model has been calibrated to 2018 and 2019 operating data for sludge production and composition, and separately calibrated for the intensive monitoring period of November-December 2019. The specific function of the calibration was to estimate the key sludge production parameters which cannot be adequately estimated from direct measurement of the influent sewage stream (e.g. Unbiodegradable-Particulate COD Fraction (F_{up}), and Inert Suspended Solids (ISS)).

The steady-state model calibration analysed operations for each quarter of 2018 and the calibration period, by drawing on:

- The extensive operations data in terms of sewage flow, waste sludge flow, mixed liquor solids concentration, alum dose rate, and effluent phosphorus concentrations.
- Biosolids haulage records (as an independent measure of sludge production and solids capture). Due to intrinsic uncertainties in biosolids haulage records (particularly due to variations in dry solids content of the dewatered biosolids cake), the application of these records have been limited to their use as a general check.
- Two filtrate sampling results from 2018 (Jan and Dec),which indicated solids capture of 87% in dewatering. This figure was applied to calculation of true sludge age from the model. This result was broadly in line with analysis of the biosolids haulage records over a 12 month timestep, and indicated a solids capture rate in dewatering of approximately 90%. Note that the dewatering filtrate sampling data from the intensive monitoring period was highly

variable, with suspended solids results ranging between 12 and 1600 mg/L. This variability rendered the filtrate data largely unusable in the estimation of dewatering solids capture.

- Eight sampling results for mixed liquor VSS/TSS ratio from 2013-2019. These results, while few in number, indicated a VSS/TSS ratio consistently in the range of 79-80%.
- Three mixed liquor VSS/TSS results measured in the intensive monitoring period, which ranged from 83.8 - 85.3% (average 84.5%).

Using this processed data, unbiodegradable-particulate COD and inert suspended solids in the influent were then estimated for each year of the analysis periods using the following methodology:

- Step 1: Estimate the mass of sludge in the secondary treatment process using the plant log data.
- Step 2: Estimate the sludge age by dividing the sludge inventory by the mass wasted each day.
- Step 3: Develop a steady state model of the process using the influent sewage load (COD, TKN, TP, F_{bs}, F_{us} etc.), the average sludge age (estimated in Step 2), and the average temperature for the relevant period.
- Step 4: Calibrate the model to balance the total sludge production and mixed liquor VSS/TSS ratio through adjustment of the unbiodegradable-particulate COD fraction (F_{up}) and inert suspended solids (ISS).

The results of this analysis are summarised in Table 3-3, and show an excellent fit to the available monitoring and operating data. Overall, while the intensive monitoring period was relatively short (and therefore may have indicated to shorter-term variations influent quality), the data from this period were more comprehensive and internally consistent. As a result, the intensive monitoring period monitoring is considered to be more reliable, and has been given greater weighting in the influent characterisation adopted for planning.

Table 3-3: Victoria Point STP – Steady-State Model Solids Production Calibration to 2018 and 2019 Operating Data

Parameter	Units	Q1 2018	Q2 2018	Q3 2018	Q4 2018	Nov-Dec 2019 (intensive)
Input Operational Parameters (Measured or estimated from data)						
Influent ADWF	ML/d (L/EP/d)	5.80 (183)	6.10 (193)	5.7 (180)	5.95 (188)	4.91 (153)
Influent COD	g/EP/d	122.4	122.4	122.4	122.4	122.6
Mixed Liquor Temp	˚C	25.9	22.3	20.5	24.4	26.0
True SRT (87% solids capture)	days	20.2	19.7	18.8	20.0	
Effluent PO_4-P	mgP/L	0.45	0.40	0.31	0.40	1.22
Alum Dose	mg/L as alum powder	31	31	31	31	0
VSS/TSS in Mixed Liquor (calibration target)	%	79.5%				84.5%
VSS/TSS in Mixed Liquor (model output)	%	79.5%	79.3%	79.3%	79.5%	84.1%
Calibration Error – VSS/TSS	% Error	0.0%	0.3%	0.3%	0.0%	0.5%
MLSS (calibration target)	mg/L	3219	3255	3544	3556	3433
MLSS (model output)	mg/L	3222	3260	3211	3336	3377
Calibration Error – MLSS	% Error	-0.1%	-0.2%	9.4%	6.2%	1.6%
Average Haulage	*t/d*	*11.9*	*11.7*	*12.3*	*12.2*	*11.9*
Average Dryness (%)	*%*	*14.7*	*14.7*	*14.3*	*14.1*	*13.7*
Haulage Sludge Production (target)	*kg/d*	*1747*	*1719*	*1758*	*1728*	*1624*
Sludge Production (model output)	*kg/d*	*1515*	*1572*	*1622*	*1585*	*1483*
Calibration Error – Sludge Haulage	*% Error*	*13.3%*	*8.6%*	*7.7%*	*8.3%*	*8.7%*
Calibration Outputs						
Fup	ratio	0.240	0.244	0.25	0.25	0.26
Inert Suspended Solids	mg/L	32	32	36	36	35

3.3.5 Nutrient Loads

3.3.5.1 Total Nitrogen

The loads of influent nitrogen are generally on the lower end of those normally observed for Australian municipal sewage. An average value was selected based of 10.8 g/EP/d was adopted based on the intensive monitoring period result. This is 3% below the average estimated from the long term data. In the absence of long term nitrogen load data, the maximum monthly nitrogen load has been applied as 15% higher than the average annual result.

3.3.5.2 Total Phosphorus

The average Phosphorus load on the plant is slightly lower than value expected for typical Australian Municipal Sewage, at 1.4 g/EP/d. This result is consistent with the general reduction in influent total phosphorus observed across Australia over the last 8-10 years. Similarly to the nitrogen loads, due to the absence of long term phosphorus load data, the maximum monthly phosphorus load has been applied as 15% higher than the average annual result.

3.3.6 Diurnal Variations in Influent Sewage Composition

The Intensive Monitoring Period included three days of monitoring of diurnal variations in influent and effluent composition. The monitoring was based on 2-hourly composite samples, tested for the major pollutants such COD, suspended solids, nitrogen, and phosphorus. The influent monitoring results are summarised in Figure 3-5 and Figure 3-6 overleaf. Note that these plots have been simplified to represent a continuous 12am to 12pm profile, but are based on stitching the 12am-8am results from the second day of each monitoring event to the 8am-12pm results from the first day of each monitoring event.

- Substantial variations in influent suspended solids within the diurnal pattern – particularly for the December 18-19 monitoring. This may be the result substantial settling of solids in the network upstream of the plant during periods of lower or average flow, and resuspension of the solids with the onset of the morning and evening peak flow periods.
- Relatively large diurnal variations in the influent concentrations of COD and Total Phosphorus, with the peak in concentration coinciding with the peak flow period. The peak in concentrations is higher than often observed in municipal sewage catchments, and may be due in part to the peak in suspended solids.
- The peak in influent nitrogen concentration commencing a little prior to the peaks in COD and TSS. This is frequently observed in municipal sewage catchments (due to a greater proportion of the influent nitrogen being soluble rather than particulate), and can have implications for denitrification performance in secondary treatment processes.

The average of the diurnal profiles from each of the three days of monitoring (as shown in Figure 3-6) were applied to the calibration and planning.

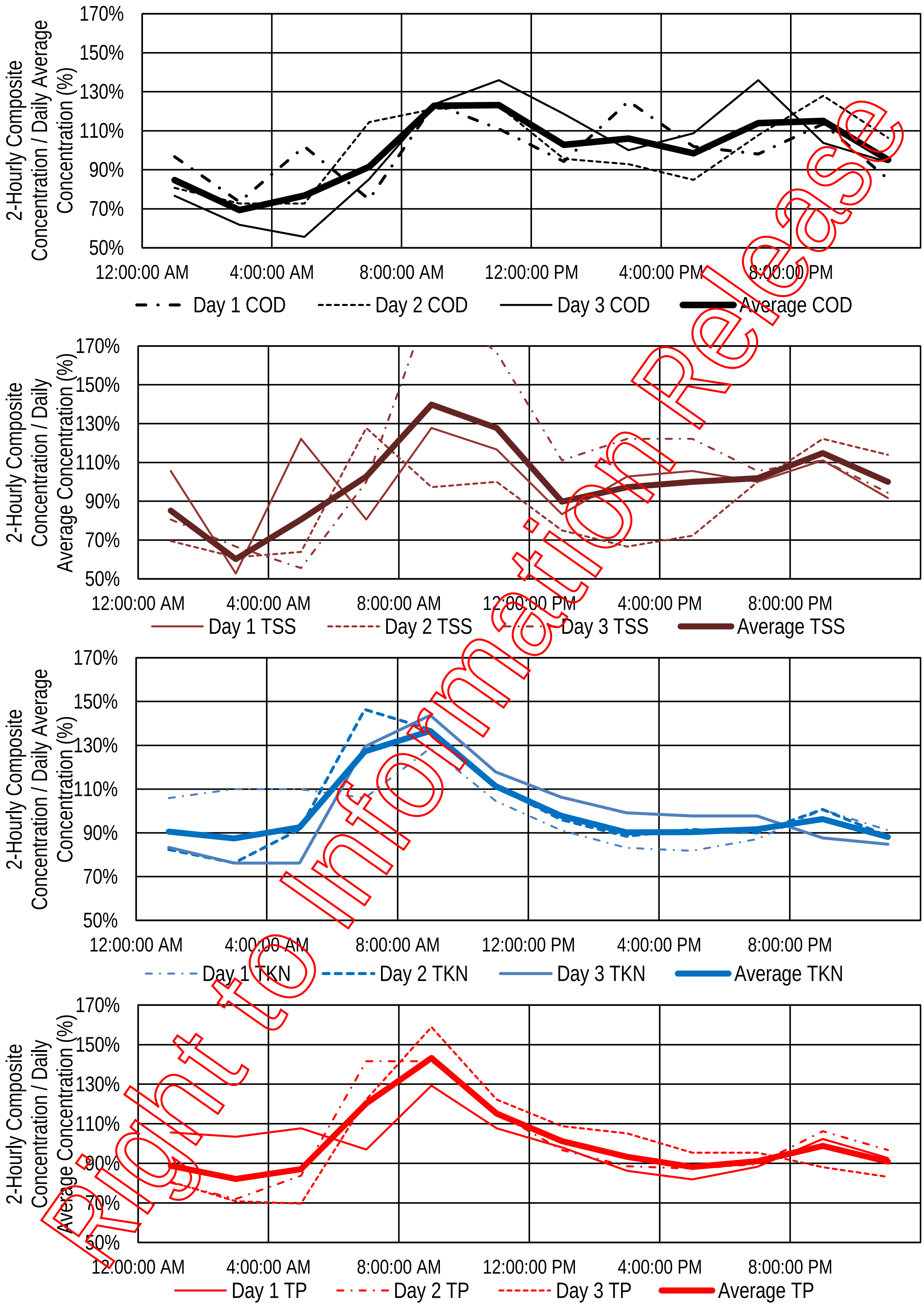

Figure 3-5: Diurnal Influent Pollutant Concentration Profiles (Nov 29-30, Dec 2-3, Dec 18-19, 2019)

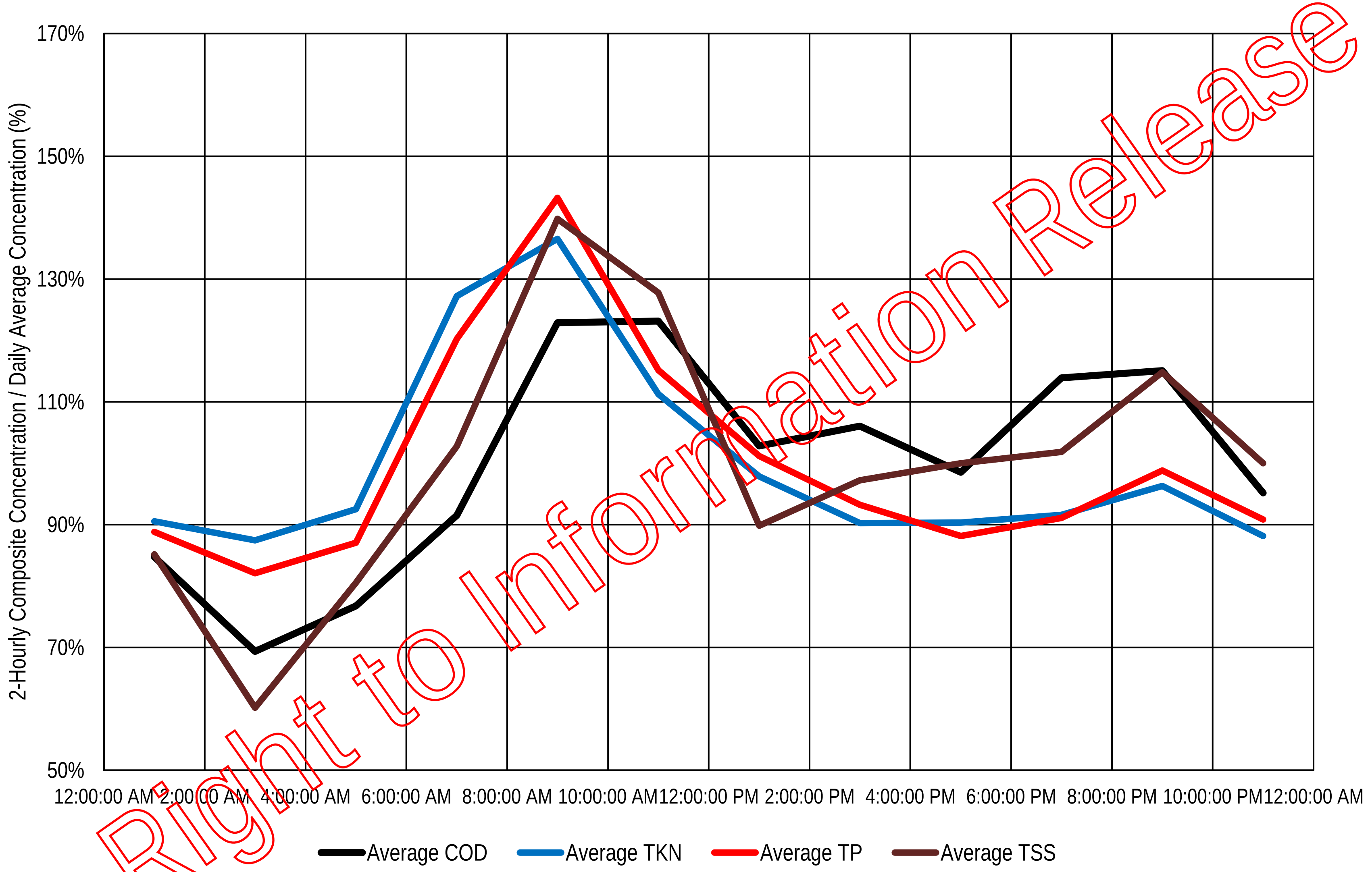

Figure 3-6: Average Diurnal Influent Pollutant Concentration Profiles (Nov 29-30, Dec 2-3, Dec 18-19, 2019)

3.4 INFLUENT LOADS ADOPTED FOR CAPACITY ASSESSMENT

The influent characteristics adopted for planning, as derived as described in the previous sections, are summarised in Table 3-4. As outlined in the preceding sections, a number of key assumptions have been applied to the generation of these estimates.

Table 3-4: Influent Per Capita Flows and Loads

Parameter	Original Design (2001-2002)	Strategic Planning Review (2009) (“**Future Case Conservative**”)	Applied as Basis of Planning (from Long-Term Data)	November-December 2019 Intensive Monitoring Period	Applied
Flows and Loads					
Average Dry Weather Flow	220 L/EP/d	190 L/EP/d increasing to 230 L/EP/d by 2025	220 L/EP/d	153 L/EP/d	220 L/EP/d (153 L/EP/d also considered)
Peak Wet Weather Flow to Secondary Treatment Process	5 x ADWF		5 x ADWF		5 x ADWF (1100 L/EP/d)
COD	115 g/EP/d (MML 138 g/EP/d)	126.5 g/EP/d	122.4 g/EP/d at AAL (Ave Oct 2015- 2018)	122.6 g/EP/d	122.6 g/EP/d at AAL 144.7 g/EP/d at MML
Total N	11 g/EP/d	15 g/EP/d	11.1 g/EP/d at AAL (Ave Oct 2015- 2018)	10.8 g/EP/d	10.8 g/EP/d at AAL 12.4 g/EP/d at MML
Total P	2.5 g/EP/d	3.2 g/EP/d	1.7 g/EP/d at AAL (Ave Oct 2015- 2018)	1.4 g/EP/d	1.4 g/EP/d at AAL 1.54 g/EP/d at MML
Inert Suspended Solids	Back-calc from sludge production: 26 mg/L at AAL 30 mg/L at MML		36 mg/L at AAL (calibration) 41.4 mg/L at MML	35 mg/L	35 mg/L at AAL 40 mg/L at MML
COD Fractions					
Unbiodegradable Particulate (Fup)	Back-calc from sludge production: 0.21		0.25	0.26	0.26
Readily Biodegradable (F_{bs})	0.15		0.15	0.157	0.157
Unbiodegradable Soluble (F_{us})	Not stated		0.05	0.03	0.03

3.5 SLUDGE AGE, SLUDGE SETTLEABILITY AND CLARIFIER DESIGN PARAMETERS

The settleability of the mixed liquor generated within the secondary treatment process is critical to establishing the plant's existing capacity and upgrade requirements. The settleability of the plant sludge, measured as Dilute Sludge Volume Index (DSVI), has been routinely monitored during operations. Under the DSVI test methodology, the settling cylinder needs to have a sludge volume of 150-250 mL/L at the end of 30 minutes. As shown in Figure 3-7, many of the monitoring results exceeded this range – especially prior to 2013. Fortunately, there remain an average of more than 150 valid DSVI test results for the last 5 years, providing ample data for analysis.

The settleability is plotted with mixed liquor suspended solids and sludge age in Figure 3-7 and Figure 3-8 respectively. Within this data, it is worth noting that settleability is a complex function and not strongly correlated to recorded parameters. For example:

- The data suggests that higher mixed liquor concentrations tend to correlate a more favorable (lower) DSVI. However, the correlation appears to be minor, and may be an artefact of the test methodology rather than process conditions.
- There are anecdotal reports that alum dosing improves settleability. The results for Victoria Point STP are somewhat consistent with this observation. In 2013-14, the plant operated without alum dosing, and achieved an average settleability of 212 mL/g DSVI. From 2015 to early 2019, an alum dose of approximately 40 mg/L was applied, and a lower average DSVI of 182 mL/g achieved. However, as the average DSVI in 2018 was 217 mL/g with an alum dose of 43 mg/L, this improvement was not consistent enough to make a substantial material impact on the "unfavorable" settleability which should be adopted for planning.
- There does not appear to be any strong correlation between sludge age and settleability. The gradual decline in the plant's sludge age over the last 12 years of operation does not appear to have a marked impact on the settleability (or the range of settleabilities) observed.

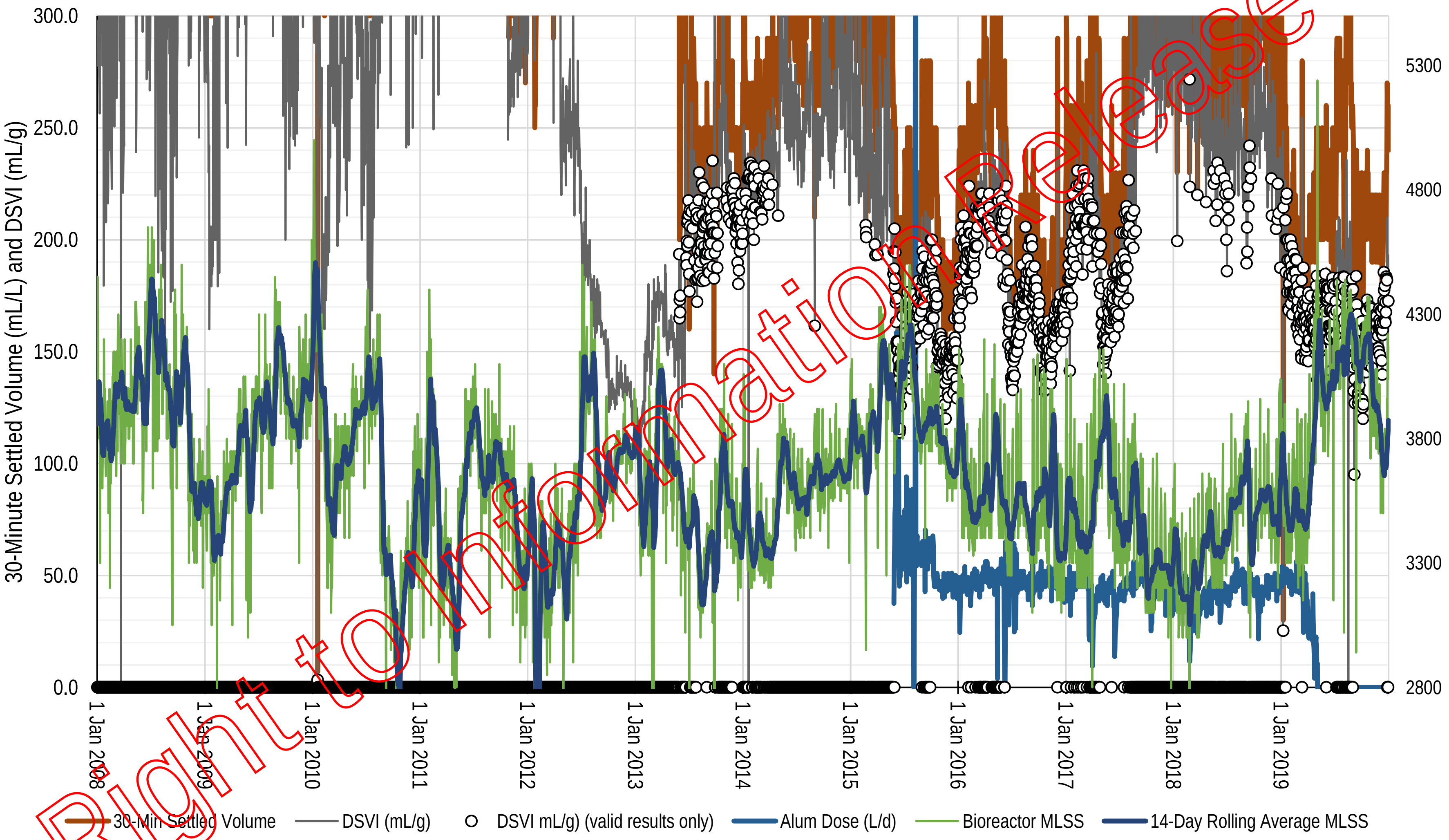

Figure 3-7. Victoria Point STP – Settleability, Alum Dose and Mixed Liquor Suspended Solids

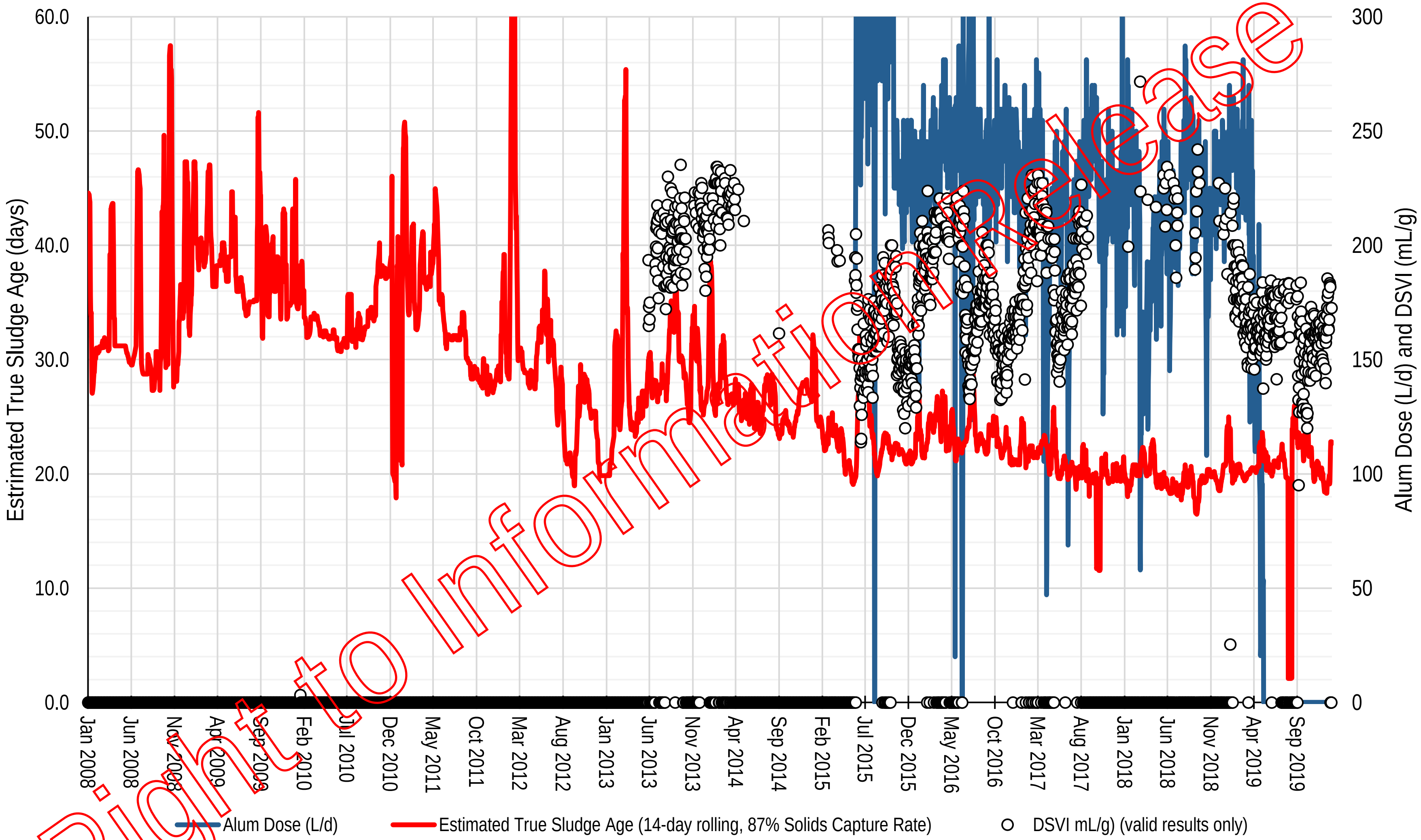

Figure 3-8. Victoria Point STP – Settleability, Alum Dose and Sludge Age

Table 3-5 summarises sludge age and settleability applied to the 2003 upgrade design, 2009 Strategic Planning Review, and adopted for upgrade planning.

Table 3-5: Clarifier Design Parameters for Previous Upgrades and Applied for Upgrade Planning

Parameter	Original Design (2001-2002)	Strategic Planning **Review (2009) ("Future Case Conservative")**	Applied as Basis of Planning Estimated from Monitoring, Solids Calibration and IDM (2018)
Sludge Age	25 days	25 days	15 days (see Section 3.8)
Mixed Liquor Solids	90th%ile MLSS = 1.2 x AAL (applied to design)	90th%ile MLSS = 1.2 x AAL (applied to design)	Maximum Monthly MLSS = 1.17 x AAL Based on: • MML/AAL peaking factors (1.18 for COD, 1.15 for solids and nutrients) • 20°C Minimum Temperature
Settleability	185 mL/g DSVI (90th %ile) Vo: 5.81 m/h n: 0.34 m^3/kg FST Design Factor = 1.0		205 mL/g DSVI (80th %ile 2013-June 2019) Vo: 5.47 m/h n: 0.492 m^3/kg FST Design Factor = 0.8

The key clarifier design parameters differ markedly between the 2003 upgrade design and the values adopted for the upgrade planning. Key differences are as follows:

Sludge Age: The reduction in sludge age from 25 days to 15 days effectively increases clarifier capacity (by reducing mixed liquor solids concentration). The capability of the plant to achieve the effluent quality requirements at the reduced sludge age of 15 days has been verified by process modelling (see Section 5). Further, operating experience from other oxidation ditches in South East Queensland, and operation of Victoria Point STP at a true sludge age of less than 20 days over recent years, indicates that the lower sludge age represents a sound basis of planning.

Peak Mixed Liquor Solids: The peaking factor of 1.2 applied to the 2003 upgrade design is comparable to the peaking factor derived through application of the adopted maximum monthly sewage loads and the impact of minimum operating temperature (1.17).

Settleability: The settleability adopted for the upgrade planning is substantially inferior to that applied to the 2003 Upgrade Design in three key respects:

1. The settleability (as DSVI) measured on site is consistently inferior to that applied to the 2003 design. The 80th percentile DSVI has been applied to the upgrade planning as adoption of the 90th percentile is considered excessively conservative (given the other design factors applied).
2. Clarifier designs undertaken using the Vesilind Flux model rely on published correlations between settleability (e.g. DSVI) and the model parameters V_o and n. The n-value applied to the 2003 upgrade design (0.34) is much more favourable than that derived from the IAWQ correlation (0.47, (Ekama, et al., 1997)), and suggests a settling rate of approximately 1.21 m/h (compared to 0.66 m/h for the IAWQ correlation at the design maximum monthly mixed liquor concentration). In spite of this figure, the clarifiers appear to have been sized based on a settling rate of 0.90 m/h in the 2003 upgrade design. This is equivalent to a DSVI of 142 mL/g under the IAWQ correlation and the maximum solids concentration – a very favourable settleability compared to the measured 80th percentile of 205 mL/g DSVI.
3. It has become part of sound clarifier design practice to de-rate the peak flux and surface overflow rate for design by a factor of 0.8 to account for the typical non-idealities found when comparing the outputs of the Vesilind Flux theory with the results of stress tests on full scale clarifiers (Ekama, et al., 1997). This approach has been adopted for the upgrade planning.

Sludge Storage in Clarifiers: The upgrade planning has included provision for the storage of sludge in the clarifiers. Sludge storage in the clarifiers serves to increase the clarification capacity by reducing the mixed liquor solids concentration in the clarifier feed. The depth of sludge applied to the analyses comprised:

For Calibration: Up to a depth of 0.82m up the side wall - based on the measured sludge level in the existing plant, and,

For upgrade planning: Up to 0.3m (upgraded plant) up the side wall.

The TSS concentration in the clarifier blanket was assumed to the same as the concentration in the mixed liquor. It does not appear that any provision for clarifier sludge storage was included in the 2003 upgrade design.

Overall, the clarifier design parameters applied to the planning result in:

- A comparable maximum surface overflow rate of approximately 0.86-0.93 m/h for the current plant (cf. 0.9 m/h under 2003 design).
- A lower maximum surface overflow rate with addition of a further clarifier or additional reactor volume (primarily due to higher mixed liquor solids concentrations at higher loads).

3.6 ENVIRONMENTAL LICENCE LIMITS FOR DISCHARGE AND EXISTING PLANT EFFLUENT QUALITY

3.6.1 Effluent Quality Criteria

The effluent quality criteria required under the current Victoria Point STP Environmental Authority (EPPR00874613) are summarised in Table 3-6.

Table 3-6: Surface Water Release Limits from Victoria Point STP to Eprapah Creek (Release Point W1)

Parameter	Min	Long Term 50th %ile	Short Term 50th %ile	Long Term 80th %ile	Short Term 80th %ile	Max
Design ADWF (ML/d)						8.5 (98.4 L/s)
Max Inflow (ML/d)						42.5 (491 L/s)
BOD_5(mg/L)				10 mg/L	15 mg/L	30 mg/L
Suspended Solids (mg/L)				10 mg/L	15 mg/L	30 mg/L
pH	6.5					8.5
Dissolved Oxygen (mg/L)	2					
Total N (mgN/L) Note 1		3 (2 @ St 2)	5 (3@ St 2)			9 (6@ St 2)
	Mass Load must not exceed 13.5 kgN /d					
Total P(mgP/L)Note 1		5 (4@ St 2)	10 (6@ St 2)			15 (12@ St 2)
Free Cl (mg/L)						0.7
Faecal Coliforms	150 cfu/100ml (median of 5 samples), 600 cfu/100ml (4 out of 5 samples)					

Note 1: The existing Environmental Authority states "*Second stage Nitrogen limits shall come into effect when the long term 50th percentile Nitrogen load from the plant reaches 13.5 kgN/d. The long term 50th percentile total effluent Nitrogen load from the plant must not exceed 13.5 kgN/d. Second stage Phosphorus limits are based on blend of 6.9 mgP/L from the existing plant and 2 mgP/L from the new plant*". However, the plant is required to achieve better than the Stage 2 concentration limits to comply with the 13.5 kgN/d mass load limit (see Figure 3-10).

The 13.5 kgN/d limit for total nitrogen has been the subject of substantial consultation with the regulator, stretching back to 2002. The limit was derived as an estimate of the prevailing mass load of nitrogen discharged to the Eprapah Creek by the plant prior to the 2003 upgrade. Under analysis undertaken by GHD at the time of the upgrade (de Haas, 2003), it is understood that the mass load of 13.5 kg/d was estimated based on grab samples of effluent collected at approximately 8am each day. As the effluent total nitrogen concentration was much lower at 8am than at other times of day, the actual nitrogen mass load during this period was likely to be substantially higher, and was estimated to be 21.3 kgN/d. This figure was not reflected in the plant's Environmental Authority at the time. Subsequent efforts to have DES modify the limit to 21.3 kg/d (including in 2003, 2010, and 2017) have not been successful.

As background to future development of the plant, and discussions with DES, the assimilative capacity of Eprapah Creek is currently being modelled. To this end, specific areas of investigation within this project include:

- Ability to tolerate total nitrogen loads (for example, loads exceeding 13.5 kgN/day);
- Potential benefits (in terms of acceptable nitrogen loads) of relocation of the STP's discharge location closer to the mouth of Eprapah Creek;
- Potential benefits (in terms of acceptable nitrogen loads) of confining the STP's effluent discharge to ebb tide, and,
- The scope to deliver reduced nitrogen loads to Eprapah Creek through nutrient reductions from other sources (offsets).

Preliminary advice from the specialists undertaking the modelling suggests that nitrogen discharges will remain the key pollutant of concern for Eprapah Creek in the future. By contrast, the STP dry weather flow, and phosphorus loads are not expected to be the critical parameters impacting the creek's health.

The environmental modelling is scheduled for completion in July 2020. Pending completion of this analysis, the upgrade planning has assumed that the concentration and mass load limits within the current licence will be retained into the future – including the critical limit for the existing mass load limit of 13.5 kgN/day of total nitrogen.

The upgrade planning has been based on:

1. Maintaining effluent total nitrogen mass loads at less than 13.5 kgN/day under average annual loading conditions with temperature at or above the annual average of 23.9^{o}C. Application of this criteria means the Stage 2 long-term median total nitrogen limit of 2 mg/L will be met.
2. Meeting the Stage 2 short term median total nitrogen concentration limit of 3 mg/L at the critical loading conditions of maximum monthly sewage loads and a minimum operating temperature of 19.5^{o}C. While the wording of the existing Environmental Licence is ambiguous in relation to the transition from Stage 1 to Stage 2 limits, the Stage 2 nutrient limits have been applied as they appear to be most consistent with the planning applied to the original 2003 plant design. Additionally, within this second criteria, the predicted level of exceedance of the maximum TN mass load limit of 13.5 kg/d under these "worst case" operating conditions must be minor to be consistent with the need for median concentration limits to accommodate short-term process disruptions due to equipment outages or other issues.

3.6.2 Historical Effluent Total Nitrogen

The long term median effluent total nitrogen of the plant has been analysed on an annual basis for 2014 through 2018, and for the period of January through May of 2019. As shown in Table 3-7, the results range between 1.40 mg/L and 1.90 mg/L. The data also suggests no significant correlation between effluent TN concentration and annual rainfall.

Table 3-7: Victoria Point STP – Long Term (Annual) Median Effluent Total Nitrogen and Annual Rainfall

Year	Annual Rainfall (mm)	Annual Median Effluent Total Nitrogen (mg/L)
2011	1584	
2012	1384	
2013	1480	
2014	838	1.40
2015	1503	1.60
2016	791	1.90
2017	1121	1.40
2018	1096	1.40
2019 (January to May)	456	1.90

Note: Time weighted composite effluent samples.

The mass load limit of 13.5 kgN/d effectively reduces the acceptable long-term median effluent total nitrogen concentration which can be discharged from the plant. As the mass of effluent nitrogen is also a function of flow, the prevailing annual per capita flow (which in turn is strongly influenced by annual rainfall) is also critical.

As shown in Figure 3-9, the compliance of the plant with the total nitrogen mass load limit has been robust over the last 5½ years. This has been the result of:

- Low annual rainfall (and Dry weather per capita flows of less than 220 L/EP/d) for all years except 2015.
- Long term median effluent total nitrogen of substantially less than 1.90 mg/L in 2014 (1.40 mg/L), 2015 (1.60 mg/L), 2017 (1.40 mg/L) and 2018 (1.40 mg/L).
- Some effluent reuse at the Redland Bay Golf Club (2.4-5.3% of average flow).

3.6.3 Effective Total Nitrogen Limit

Figure 3-10 shows the maximum effluent total nitrogen concentration based on the projected connected populations and per capita flows. This analysis effectively assumes that wet weather flow results are excluded from the data set under the wet weather criteria applied in the Cleveland STP licence (see Criteria 2 under Section 3.2). The chart additionally shows the required nitrogen concentrations at the average per capital flow under Criteria 1 for the last four years (191 L/EP/d), which represents an upper bound which would be acceptable in years of lower rainfall. Alternative calculation methodologies which directly consider wet weather flows would require lower effluent total nitrogen to be achieved.

The horizontal blue line on Figure 3-10 the shows the upper end of the range of annual median effluent total nitrogen limits achieved in the last 5½ years of operation (1.90 mg/L). As shown on Figure 3-10, the existing plant would be at risk of exceeding its mass load limit for total nitrogen where:

- *The long-term median effluent total nitrogen concentration is at the upper end of the range achieved by the plant over the last 5 years;*
- *The per capita flow is at 220 L/EP/d or more.* Analysis of flows over the last 6½ years suggests that the current catchment is likely to deliver per capita flows at or above this value in years where the total rainfall is approximately 1500mm. Long term rainfall records for Redland Bay (41 years) and Mt Cotton (86 years) indicate that annual rainfall is at or above this level for one out of every three years, AND,
- *Effluent reuse is negligible or not substantially increased from that achieved in recent operations.* The Redland Bay Golf Club reuse flows have historically ranged between 2.4% and 5.3% of the average effluent flows over the last 5 ½ years, with the lowest usage of recycled water coinciding with wet years.

However, subsequent analysis (Section 5.2.2) indicates that the high effluent total nitrogen is likely at low per capita flows, with the risk of exceedance of the nitrogen mass load limit substantially increased from 2025 under the projected increase in sewage flows.

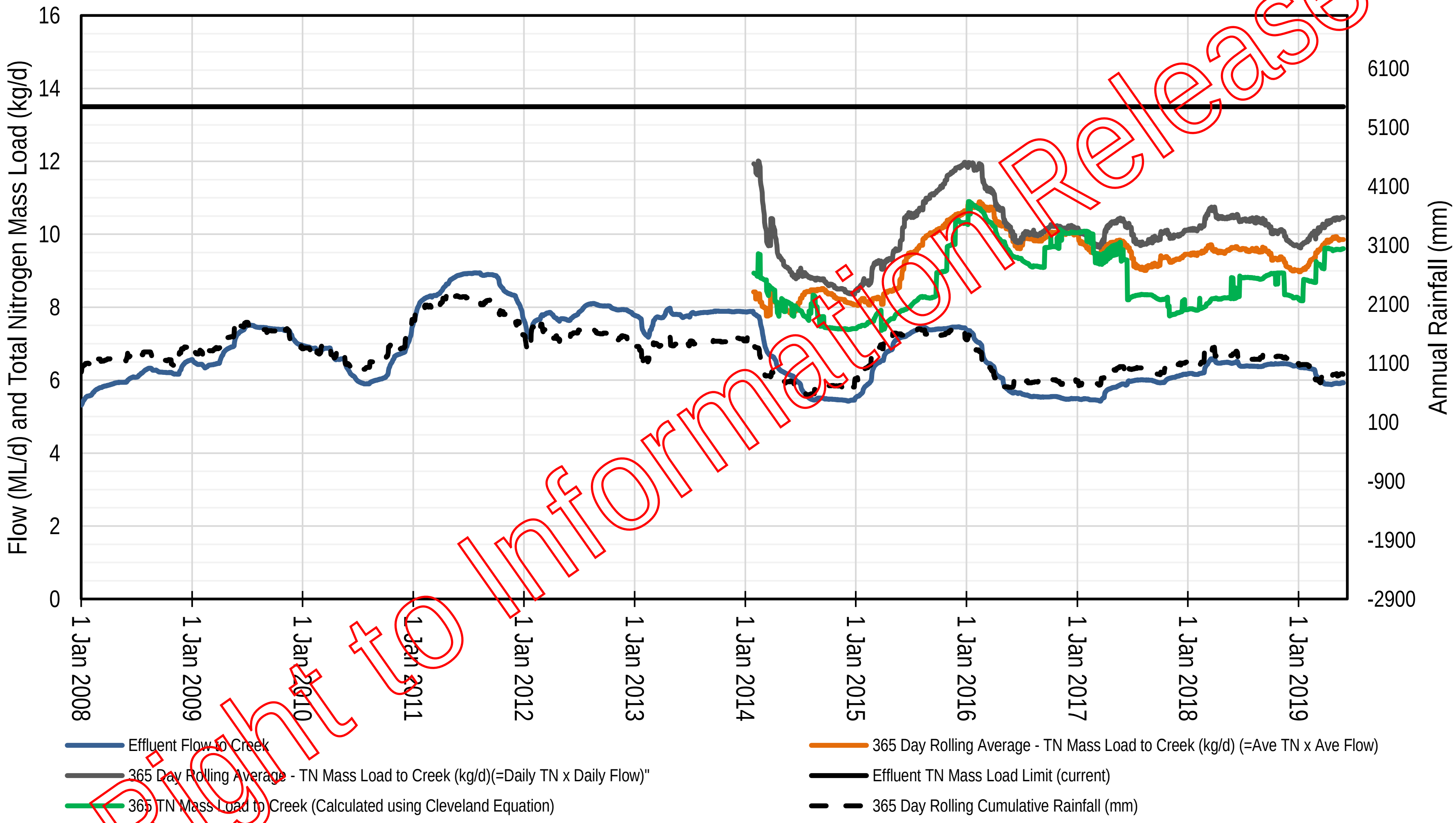

Figure 3-9: Victoria Point STP – Historical Performance Against Total Nitrogen Mass Load Limit

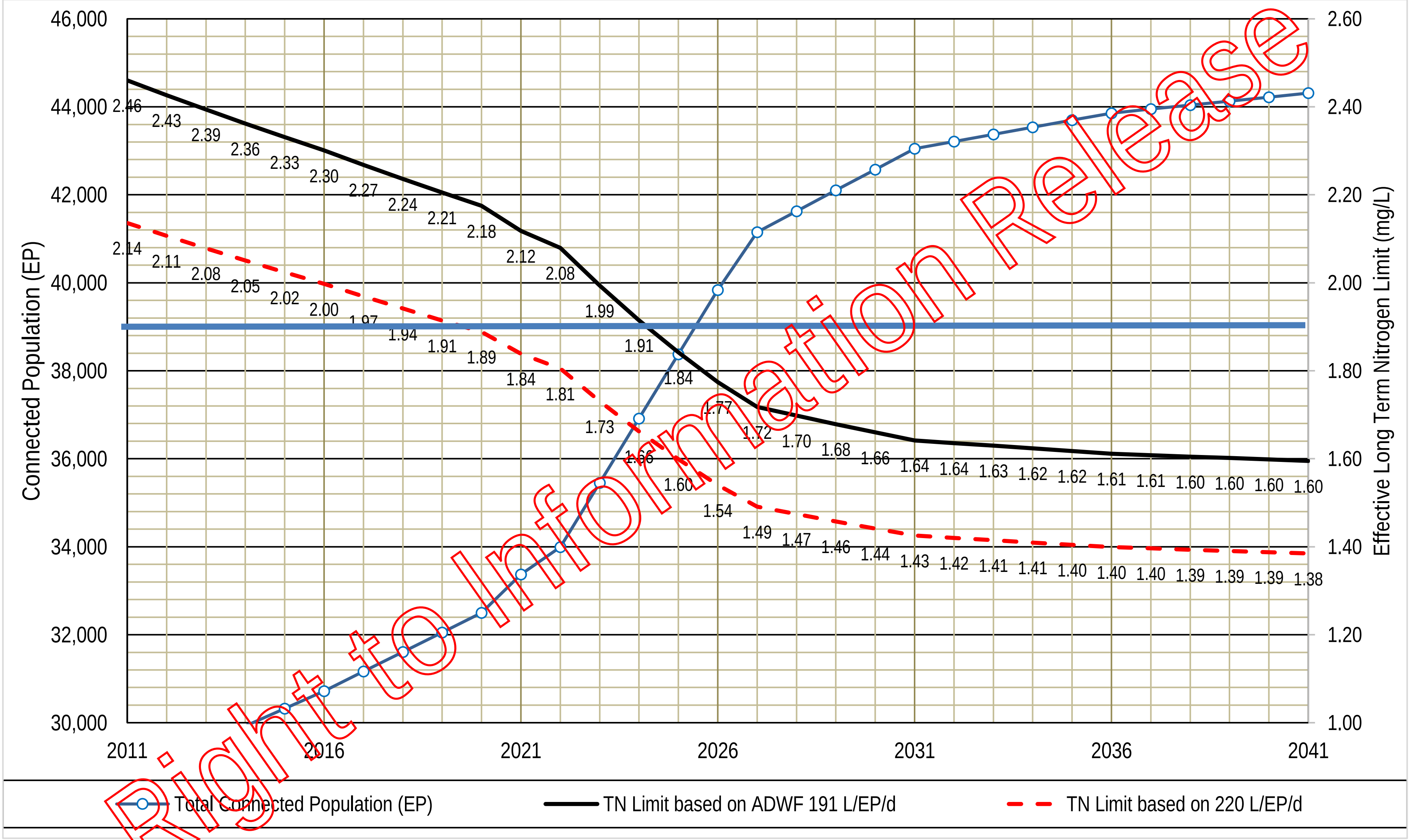

Figure 3-10: Victoria Point STP – Projected Maximum Effluent Total Nitrogen Concentration required for Mass Load Limit Compliance

3.6.4 Effluent Nitrogen Components and Refractory Dissolved Organic Nitrogen

Refractory Dissolved Organic Nitrogen (known as rDON, F_{nus} or TKN_{us}) passes directly through conventional biological treatment processes without modification, and is also generated in activated sludge. As rDON emerges in the plant effluent, its concentration has a direct bearing on the maximum inorganic nitrogen which can be permitted in the plant effluent without exceeding the licence limits. This is critical for establishing the ability of the plant to achieve lower effluent total nitrogen concentrations using conventional biological processes in the future.

The rDON in the effluent, as estimated from the effluent TKN, ammonia and suspended solids results, is listed in Table 3-8. Based on this analysis, a maximum median rDON of just under 0.7 mg/L was applied to the Phase 1 upgrade planning. This value is at the lower end of the long-term median values typically observed in South East Queensland.

However, the average rDON estimated from 24-hour effluent composite samples during the intensive monitoring period was substantially higher at 0.91 mg/L (range 0.68-1.04). While it is important to note that the low per capita flows (153 L/EP/d) during the intensive monitoring period may have contributed to the higher rDON recorded during this period, the potential impacts of a higher rDON concentration of 0.91 mg/L has been considered in the upgrade planning.

Table 3-8: Victoria Point STP – Long Term (Annual) Median Effluent Nitrogen by Species

Period	Ammonia (as N) (mg/L)	Oxidised N (mg/L)	Total Nitrogen (mg/L)	Estimated rDON (assuming nil solids)
2014	0.032	0.78	1.4	0.63
2015	0.02	1.10	1.60	0.54
2016	0.018	1.2	1.9	0.7
2017	0.013	0.76	1.4	0.68
2018	0.021	0.71	1.4	0.68
2019 (to May)	0.018	1.1	1.9	0.69
2014-May 2019	0.02	0.85	1.50	0.67
Intensive Monitoring Period	0.097	0.77	1.78	0.91

Note: Flow weighted composite effluent samples.

Importantly, the oxidised nitrogen concentrations shown in Table 3-8 indicate that the effluent ammonia concentrations are very low on average, but that there is substantial scope to reduce the effluent Total Nitrogen achieved by enhancing oxidised nitrogen removal in the secondary treatment process.

3.6.5 Ammonia Removal through Breakpoint Chlorination

The presence of chlorine in a substantial excess to ammonia (Cl:N ratio of ~9 to 1), which may currently be occurring for substantial periods of time in the chlorine contact tanks of Victoria Point STP, can result in further ammonia oxidation through 'breakpoint chlorination'. In an effort to understand the likely extent of ammonia removal via this mechanism in the existing plant operations, grab samples of filtered effluent were collected during the intensive monitoring period, and compared to the final effluent (post chlorination). The results of this analysis are summarised in Table 3-9. While the sampling results are not conclusive, they suggest that breakpoint chlorination may be having a minor impact on the effluent ammonia concentrations.

Table 3-9: Filtered and Final Effluent Ammonia Sampling Results – Nov-Dec 2019.

Ammonia Nitrogen Result	Units	Nov 29 ~9am	Dec 4 8:35am	Dec 9 ~9am	Dec 11 9:05am	Dec 13 9:00am	Dec 16 9:00am
Filtered Effluent Grab	mg/L	0.038	0.027	0.150	0.026	0.034	0.41
Final Effluent 24h Composite	mg/L	0.034	0.054	0.076	0.020	0.027	0.037
Final Effluent 2h Composite	mg/L	0.027	0.25 (8-10am, Dec 2.)		0.067 (8-10am, Dec 18)		

3.6.6 Maximum Effluent Flow

The existing Environmental Licence for Victoria Point STP states that *"Inflows must not exceed the peak design capacity of 5 times the Design Average Dry Weather Flow (DADWF) of 42.5 ML/d (DADWF = 8.5 ML/d)"* (Condition No. G4-1). Considered in isolation, the wording of this condition is somewhat ambiguous in relation to:

- Whether the average dry weather flow to the plant must not exceed 8.5 ML/d, or,
- Whether it is acceptable to treat peak flows less than 5 times the average dry weather flow - particularly where the average dry weather flow exceeds 8.5 ML/d.

A conservative interpretation of the existing licence would mean that new licence would potentially be required:

1. Once the average dry weather flow to the plant exceeds 8.5 ML/d, or;
2. To augment the plant capacity to more than 8.5 ML/d ADWF capacity.

Under this interpretation, a per capita flow of 220 L/EP/day may require a new discharge consent from DES once the connected population exceeds approximately 38,600 EP. The projected growth associated with the South West Victoria Point and Weinam Creek developments would see this limit exceeded in 2025.

Counter to this interpretation, DES may consider the view that no new licence will be required as the proposed upgrades are not intended to increase the plant's capacity above the range of the current Environmentally Relevant Activity (63-1(e) Sewage Treatment 10,000-50,000 EP). This would also be in line with preliminary expectation that increases in effluent flows to Eprapah Creek (in the absence of additional pollutant loads) are not expected to have an adverse impact on the health of the waterway (Pers. Comm., T. McAllister, December 2019).

In the absence of specific information on what new conditions might be applied, the upgrade planning has considered that the current effluent quality and mass load limits in the existing Environmental Authority would continue to apply under a new approval.

3.6.7 Peak Wet Weather Flow to Treatment

In line with the design basis applied to the 2003 upgrade, the upgrade planning has been based on transfer and full treatment of all flows up to five times the average dry weather flow (at 220 L/EP/d).

3.7 Recycled Water Quality

In the absence of details of the existing effluent reuse to the Redland Bay Golf Club, the design has assumed that no further treatment of the effluent is required to meet the requirements of the Recycled Water Management Plan.

3.8 End of Waste Code

The End of Waste (EoW) code for Biosolids was issued by the Queensland Government under the *Waste Reduction and Recycling Act 2011*(WRR Act), and became effective on January 1, 2020 (Department of Environment and Science, 1 Jan 2020). The code defines the requirements and conditions under which biosolids can be beneficially used as a resource in urban and rural land applications. Biosolids which do not meet the requirements of the code will need to be managed as a waste stream (which would generally be an inferior environmental outcome and attract much higher costs).

The issued EoW code includes the "Barrier options" for achieving Grade B biosolids stability using practices where:

- Biosolids are injected below the surface of the land, or,
- Biosolids applied to the land surface are incorporated within six hours of application on the land.

These stabilisation options are included in the USEPA and NSW Guidelines for Biosolids Reuse, and are directly relevant to the planning of Victoria Point STP's upgrades by enabling reuse of biosolids generated within secondary treatment processes with sludge ages as short as 12 days without further processing - provided the solids do not represent an "undue risk" associated with high pathogen concentrations or excessive unstabilised solids. The code identifies undue risk to be processes which are achieving less than 1-log pathogen reduction compared to primary sewage for the relevant indicator organisms.

This enables the upgrade planning to be based on the minimum sludge age required for robust nitrogen removal, and does away with the need for additional biosolids stabilisation (e.g. through digestion or composting).

3.9 REDUNDANCY

3.9.1 General

Redland City Council applies Duty/Assist redundancy as a general approach to all mechanical equipment. This principal has been applied to the development of the plant, under the interpretation that the capacity to treat or pass the peak loading of any process unit is met with all parallel elements in service.

The redundancy of the oxidation ditch aerators is based on a duty/duty/standby configuration (as per the current operations). As the positions of the three installed aerators are fixed, Aerator No.1 and Aerator No. 2 are normally operated, with Aerator No. 3 only operating at times when one of Aerator No.1 or No. 2 are out of service. An alternative feed location is provided for periods when Aerator No. 1 is out of service.

In relation to secondary clarification, the redundancy criteria applied has been expanded to consider:

- Treatment of peak wet weather flows up to 5 x ADWF (see Section 3.6.7) with all clarifiers in service, and,
- Treatment of peak dry weather flows with one clarifier out of service.

The new blowers for the Re-Aeration Zone have been configured in a duty/standby arrangement. This approach has been adopted as the failure of a single blower under a duty/assist configuration would not have sufficient capacity to treat the peak diurnal load at the planning horizon.

3.9.2 Bioreactor Redundancy

The upgrade planning has been based on retention of a single bioreactor (as per the existing plant). As an additional reactor is not required to achieve the projected process capacity, provision of a second reactor unit would add substantial costs. This means that the existing reactor will not be able to be taken out of service for repairs or maintenance through to the planning horizon (at least). Given the known structural issues in the oxidation ditch structure, this represents a risk to Redland City Council.

A high level cost estimate has been developed for duplication of the existing Victoria Point STP oxidation ditch. Based on the key unit rates, mark-ups, and contingency applied in this investigation (see Section 7.1), the estimated cost to duplicate the existing oxidation ditch has been estimated as $18.7m. As the reactor volume in the existing plant does not directly constrain the plant capacity, this considered to be a high cost for resolution of the issues in the existing structure.

Previous investigations by Redland City Council considered use of the existing, disused 'old plant' to provide treatment while the Oxidation Ditch is taken out of service for repairs. While the studies indicated that effluent TN levels <10mg/L may be achievable, extensive additional analysis would be required to verify the viability of this option. Use of the existing disused plant structures (either as temporary liquid stream treatment, or permanently as part of the sludge stream), would require a detailed structural assessment in order to ascertain viability, and to determine the scope and costs of required refurbishment measures.

3.10 CONDITION OF EXISTING PLANT INFRASTRUCTURE

The initial existing plant visual condition review (which was limited in scope to general condition observation without detailed or invasive inspection) noted the following elements of concern:

- Oxidation Ditch – Visual evidence of concrete deterioration and limited cover to reinforcement. Cracking resulting in loss of containment, which was under repair during the site visit of June 2019. In the absence of additional information, the study has assumed that the repaired oxidation ditch will be suitable for ongoing use through to the planning horizon. As noted in the previous section, the cost to duplicate the existing reactor is very high compared to the likely repair costs.
- Oxidation Ditch Aerator Covers – Severely corroded, require removal and/or replacement (depending on noise).
- Dewatering Building – Extensive corrosion to both structural steel and cladding. Repair and/or replacement of key elements required.
- Existing Gravity Drainage Decks / Belt Filter Presses – The existing TEMA GDD/BFP appears to be in reasonable condition, but is at risk of becoming obsolete within the next 5 years. The existing AJM belt press is in poor condition, and is largely obsolete (creating difficulties in maintenance). Both machines require extensive maintenance to remain operational. They also perform relatively poorly, achieving a relatively poor dry solids concentration in the dewatered biosolids product of only 12-14%. Due to the condition of the existing dewatering system, the options for upgrading the dewatering system are currently under investigation as a part of the separate project.

In general, metalwork within the existing disused plant's bioreactors and clarifiers is in very poor condition. The concrete structures, however, appear to be generally intact, and potentially suitable for ongoing service with refurbishment.

4 DYNAMIC PROCESS MODEL DEVELOPMENT AND CALIBRATION

In order to accurately assess the capacity of the existing secondary treatment process and inform the concept design of the upgrades, a dynamic BioWIN model of the existing plant was developed and calibrated. Given the very low effluent total nitrogen currently achieved by the plant, and the need to further enhance nitrogen removal in the future, the process model calibration pursued a high degree of accuracy. To generate the most accurate model possible, the following approach was applied:

- Whenever possible, actual plant operating data was used to calibrate the plant model, including:
    - Flow rates for Influent Sewage, RAS, and WAS.
    - Aerator speeds.

  For each of these parameters, 30-minute average values were derived from the SCADA historian.
- The 19-day period of December 1st to 19th, 2019 was selected for the calibration as it coincided with the characterisation program, providing the most accurate influent and operating data on which to base the model.
- The average sewage characteristics and diurnal influent sewage pollutant concentration patterns for COD, TKN, and TP derived from the characterisation period were applied. Diurnal changes in the influent total suspended solids were not applied, as this has been consistently shown to not be required to achieve a dynamic model calibration.
- As discussed in Section 3.3.4, the available samples for the solids concentration in the dewatering filtrate were highly variable. On this basis, the capture in the belt press during the calibration period was estimated using the limited historical filtrate monitoring data (which gave an estimated solids capture of 87%), as validated using the sludge haulage records and the steady-state process model calibration.
- The oxidation ditch was modelled as a series of thirty bioreactor cells to represent the plug flow nature of the Victoria Point reactor configuration (see Figure 4-1). This configuration also allows for relatively accurate comparison of key parameters, such as dissolved oxygen) at specific points in the bioreactor. A ditch velocity of 0.20 m/s, which is at the lower end of typical values was applied based visual observation of the surface flow within the bioreactor during site visits.
- Two model clarifiers were used, each with dimensions to represent the units installed at Victoria Point. As a part of this approach, the total volume of sludge in the model clarifier was compared to values reported onsite to ensure that it was an accurate representation of the plant for the period of study.
- On-site measurements of aerator power and current draw as a function of speed were collected for each aerator. This data was used to establish the relationship between power input and aerator speed in the model. Table 4-1 summarises the collected data from site and applied to the modelling of aeration.

Table 4-1: Victoria Point STP Aerator Power Consumption, Recorded March 17, 2020

Aerator	Speed (Hz)	Speed (%)	Power Consumed (kW)
No. 1	30	60	27
	40	80	54
	50	100	103
No. 2	30	60	27
	40	80	59
	50	100	100

- Two very small reactor cells were added to the model represent the additional aeration from the bioreactor weir outlet and the RAS screen.

- As no alum dosing was undertaken during the period selected for calibration, it was not included in the model.

Calibration Method

- The calibration was performed to achieve the best match possible to the monitoring results for suspended solids, total nitrogen, ammonia, nitrate, total phosphorus, and phosphate both in the bioreactor and final effluent.
- In the first instance, the efficiency of the surface aerators was adjusted in the model to provide a match to the measured dissolved oxygen concentration. Unfortunately, the configuration of the aerators and dissolved oxygen instruments leads to an unstable model configuration where very small changes in the aerator efficiency resulting in large changes DO (i.e. from 0 to 5 mg/L), or the model outputs are unstable (and unrepeatable). To overcome this limitation, control logic was developed in the BioWin Controller add-on to accurately mimic the aerator speed control in the plant.
- Even with the actual measured DO accurately met by the model, the fit of ammonia, nitrate, and nitrite was initially relatively poor, with results suggesting insufficient nitrification and excess denitrification compared to the observed plant performance. On this basis, a review of the DO profile within the oxidation ditch was carried out by Redland operations personnel using a handheld instrument. While not conclusive, the monitoring confirmed that is substantial variation in the DO concentration achieved at various locations both along the path length of ditch, and across the channel. On this basis the measured DO reported from the site data was increased to achieve the observed performance. The total fit to the observed aeration input power remained excellent even with this change.
- The calibration philosophy was based on minimising the number of kinetic and stoichiometric parameters modified from the BioWIN default values. Despite some known divergences between the BioWIN model and BNR microbiological processes, it is our experience that making a large number of poorly or partially supported changes reduces the applicability and confidence in the final model. For this calibration, the plant operating conditions, coupled with the high degree of accuracy demanded by the stringent licence requirements, a relatively large number of changes to default parameters was required. These were:
    - AOB Substrate Half Saturation reduced to 0.3 mg/L (from 0.7 mg/L) to provide the low level of Ammonia observed in the final effluent. Modifications to substrate half saturations are not typically required.
    - PAO Anoxic Growth Factor reduced to 0 from 0.33 to eliminate anoxic P uptake – to better match the level of denitrification and effluent phosphate. Modification of the anoxic growth factors is infrequently required, but was necessary to reduce the extent of phosphorus removal reported by the model in this case.
    - NOB Max Specific Growth rate increased to 1.5 /d from 0.7 /d and Substrate Half Saturation increased to 0.05 from 0.1 to reduce the nitrite and increase the nitrate in the final effluent as reported by the model. More recent model calibrations have sometimes required amendment of this parameter to prevent nitrite levels in the effluent far exceeding those observed in practice.
    - AOB DO Half Saturation and NOB DO Half Saturation decreased to 0.05 mg/L from 0.25 and 0.5 respectively. Modifications to these parameters are typical for processes where the dissolved oxygen is not uniformly maintained outside the concentration where simultaneous nitrification and denitrification is known to occur, such as oxidation ditches or intermittent processes.

Calibration Results

Given the available information, the fit of the model to the observed plant performance is considered reasonably good as shown in Figure 4-2 through Figure 4-14. More specifically:

- The model's fit with respect to effluent ammonia, nitrate and total nitrogen is considered excellent (see Figure 4-5 through Figure 4-7, and Figure 4-10 through Figure 4-11). The accuracy for these parameters far exceeds the recommended thresholds for this type of modelling, but was vigorously pursued due to the very low levels of nitrogen required at Victoria Point.
- The fit with respect to effluent phosphate and total phosphorus (see Figure 4-8, Figure 4-9, and Figure 4-12) is not as good the nitrogen species, but is still considered acceptable. Previous projects have demonstrated that BioWin

may overpredict excess biological phosphorus removal under low or transient DO conditions (such as those which occur at Victoria Point). Given that the phosphorus removal requirements are relatively lenient compared to the nitrogen removal requirements, and that additional phosphorus removal can be readily achieved with chemical dosing, this is not considered a significant limitation.

- The average solids inventory predicted by the model was within 2% of the results of the characterisation period (see Figure 4-4), and 8% of the values reported in the plant log. Both of these figures are well within the recommended 10% error range (Rieger, et al., 2013).

Table 4-2: Dynamic Process Model Calibration Evaluation

Parameter	Mean of Residuals	Absolute Mean of Residuals	Root Mean Square Error	Target Value
Effluent Ammonia	0.05	0.07	0.12	1.0 mg/L Note 1
Effluent Nitrate	-0.07	0.12	0.16	1.0 mg/L Note 1
Effluent TN	0.10	0.18	0.19	1.0 mg/L Note 1
Effluent Phosphate	0.82	0.82	0.90	N/ANote 2
Effluent TP	0.85	0.85	0.91	N/ANote 2

Note 1: Recommended target for assessing plant capacity for nitrogen removal using dynamic modelling. Monthly or annual average (Rieger, et al., 2013)

Note 2: No recommended target for assessing phosphorus removal using dynamic modelling (Rieger, et al., 2013)

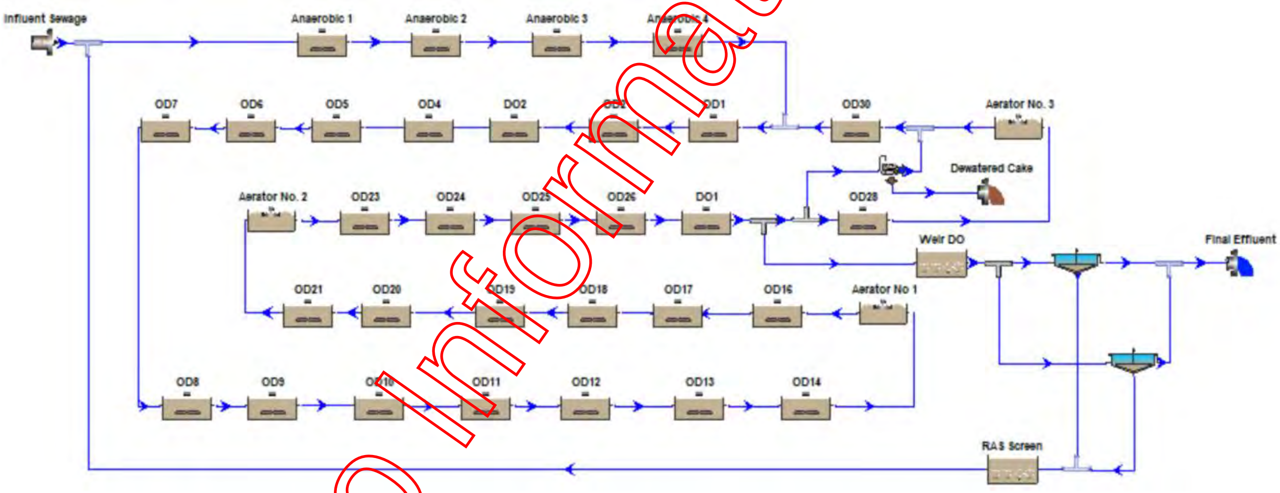

Figure 4-1: BioWIN Process Model Configuration – Existing Victoria Point STP

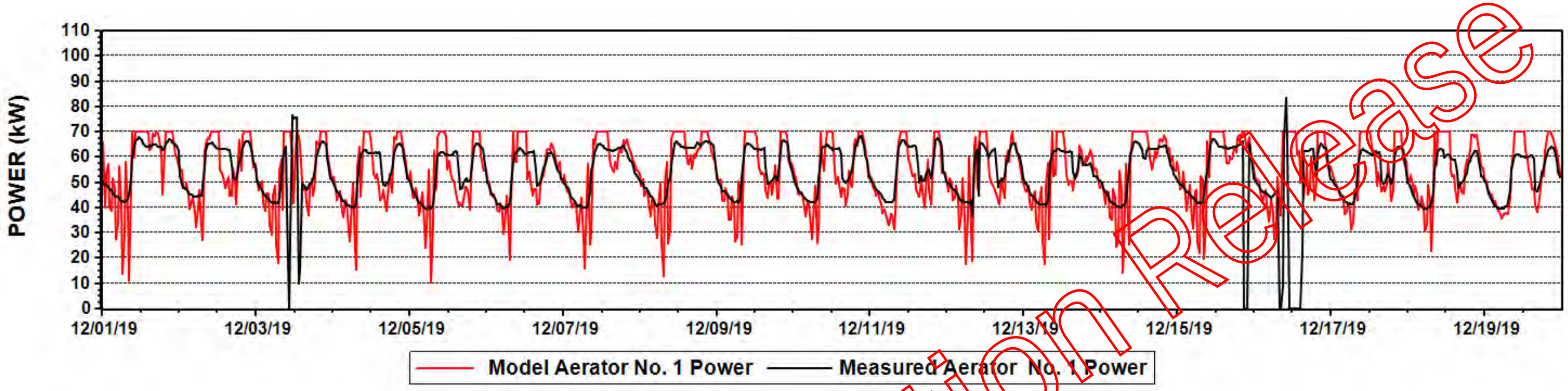

Figure 4-2: Victoria Point STP Dynamic Model Calibration - Aerator No.1 Power

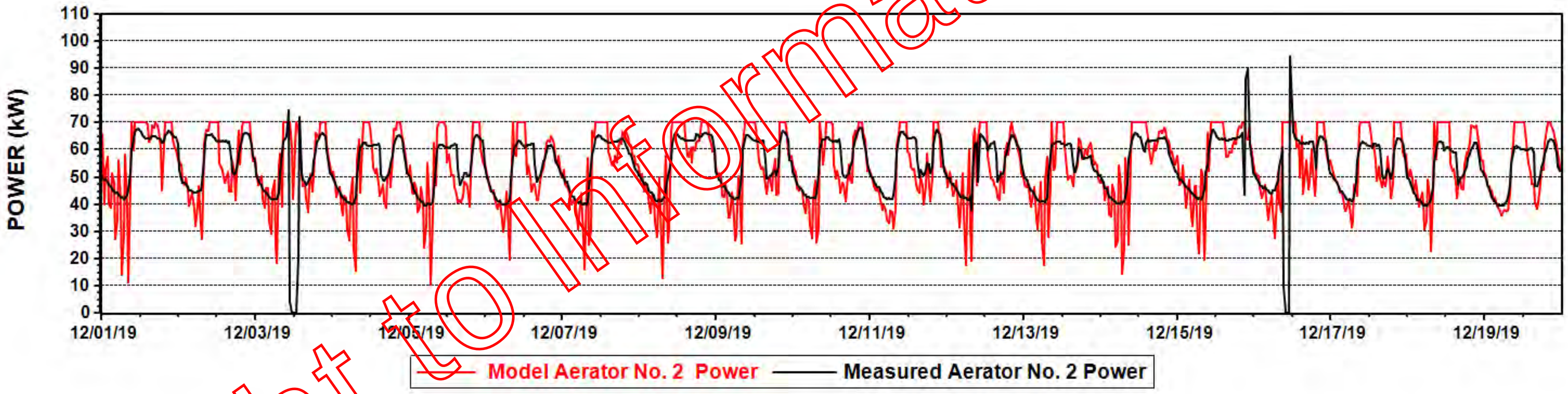

Figure 4-3: Victoria Point STP Dynamic Model Calibration - Aerator No. 2 Power

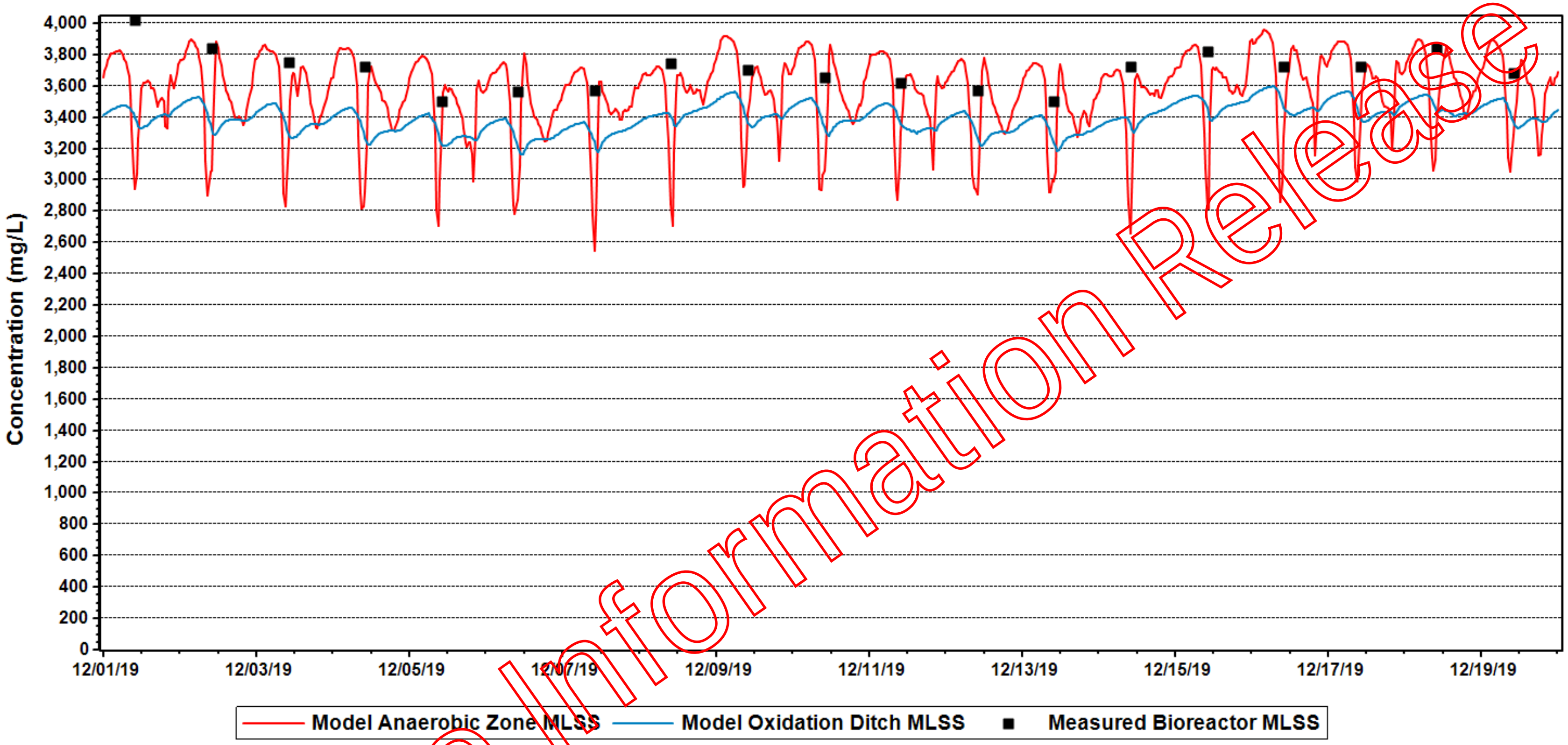

Figure 4-4: Victoria Point STP Dynamic Model Calibration - Bioreactor Mixed Liquor Suspended Solids

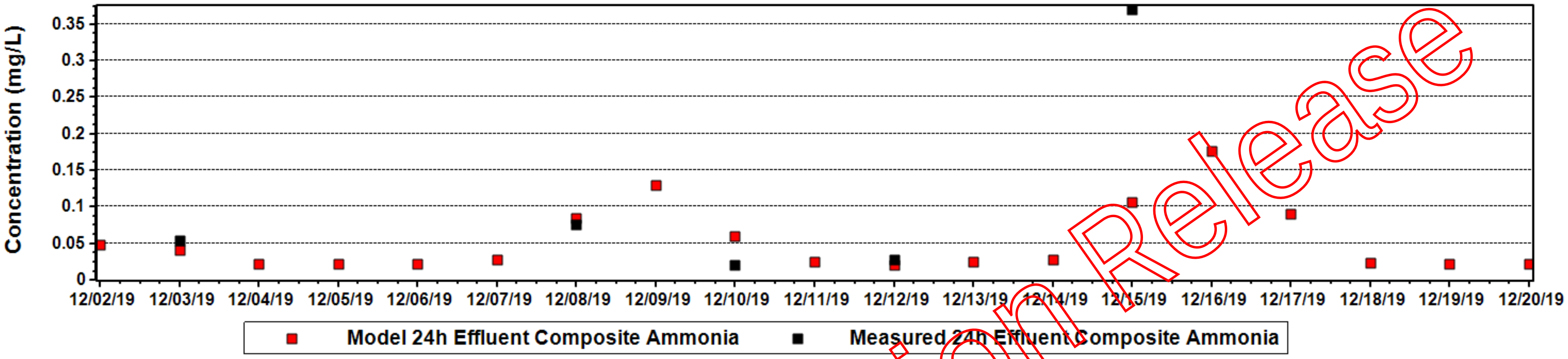

Figure 4-5: Victoria Point STP Dynamic Model Calibration - Effluent Ammonia (as N)

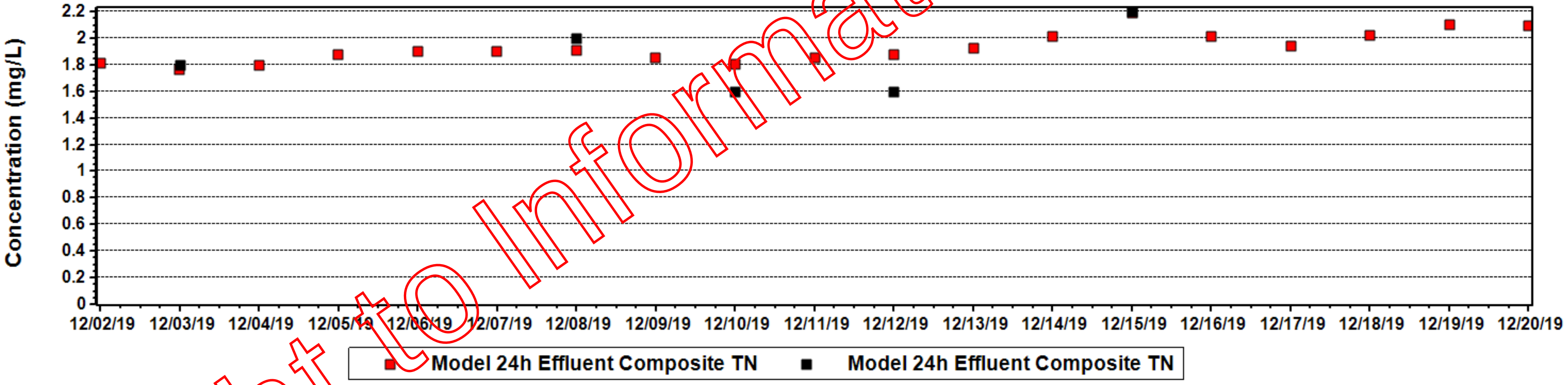

Figure 4-6: Victoria Point STP Dynamic Model Calibration - Effluent Total Nitrogen

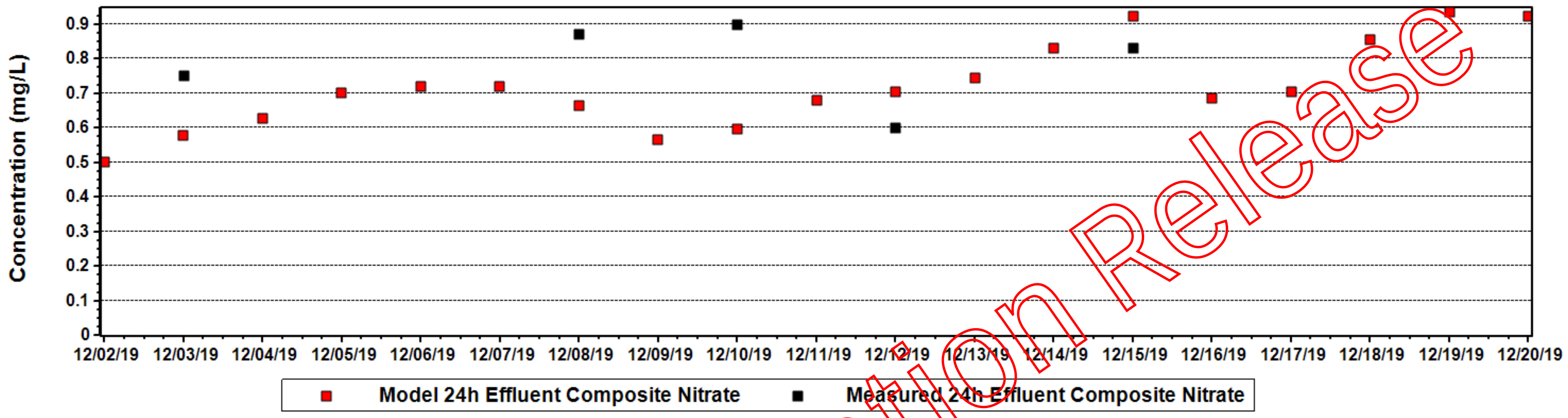

Figure 4-7: Victoria Point STP Dynamic Model Calibration - Effluent Nitrate (as N)

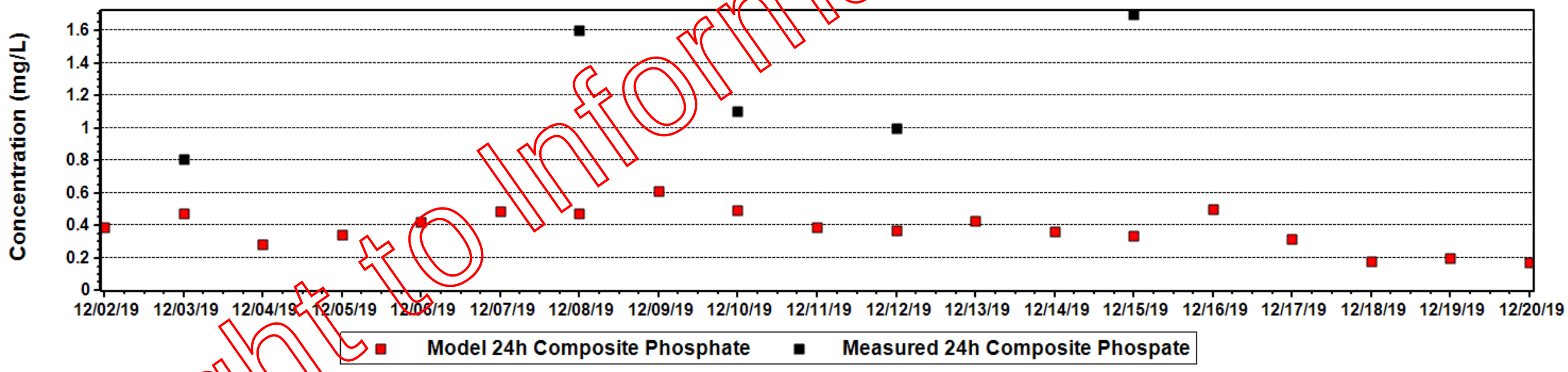

Figure 4-8: Victoria Point STP Dynamic Model Calibration - Effluent Phosphate (as P)

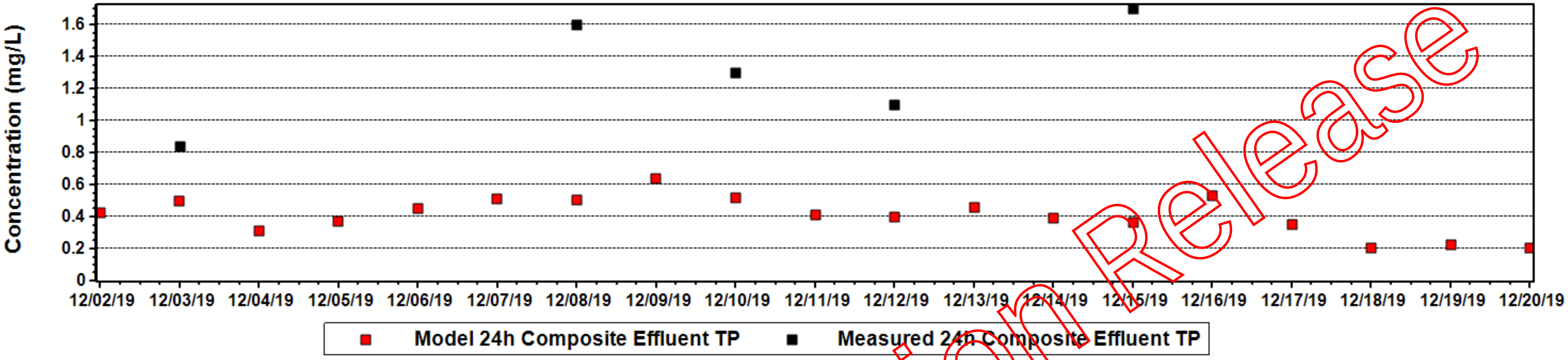

Figure 4-9: Victoria Point STP Dynamic Model Calibration - Effluent Total Phosphorus

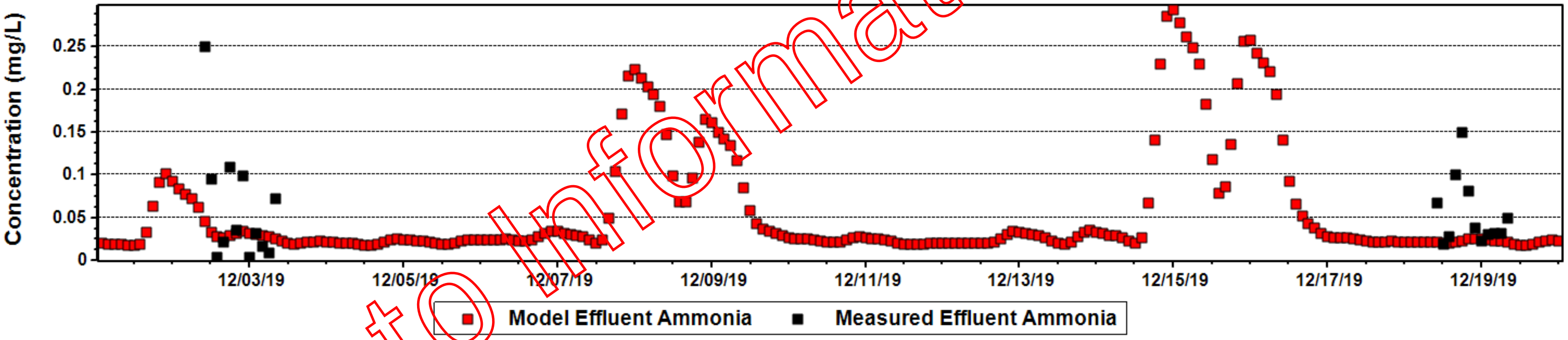

Figure 4-10: Victoria Point STP Dynamic Model Calibration - Diurnal Effluent Ammonia (as N)

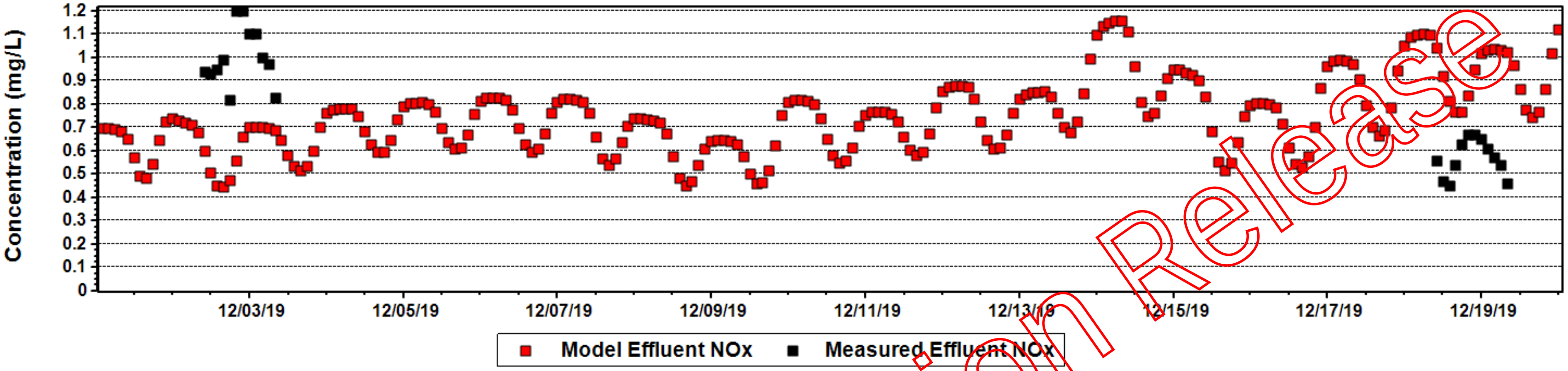

Figure 4-11: Victoria Point STP Dynamic Model Calibration - Diurnal Effluent Oxidised Nitrogen (as N)

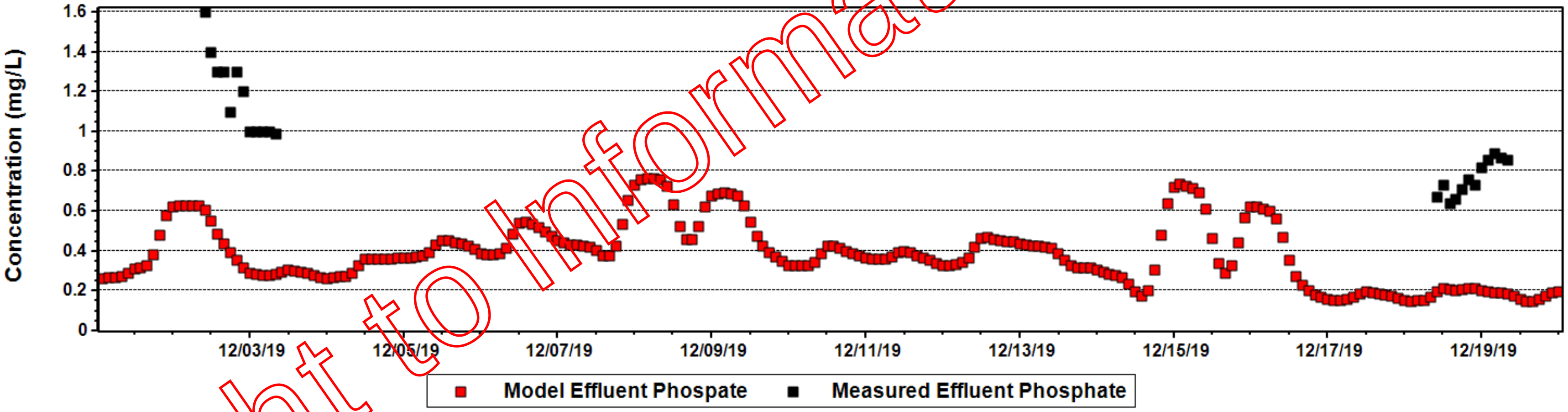

Figure 4-12: Victoria Point STP Dynamic Model Calibration - Diurnal Effluent Phosphate (as P)

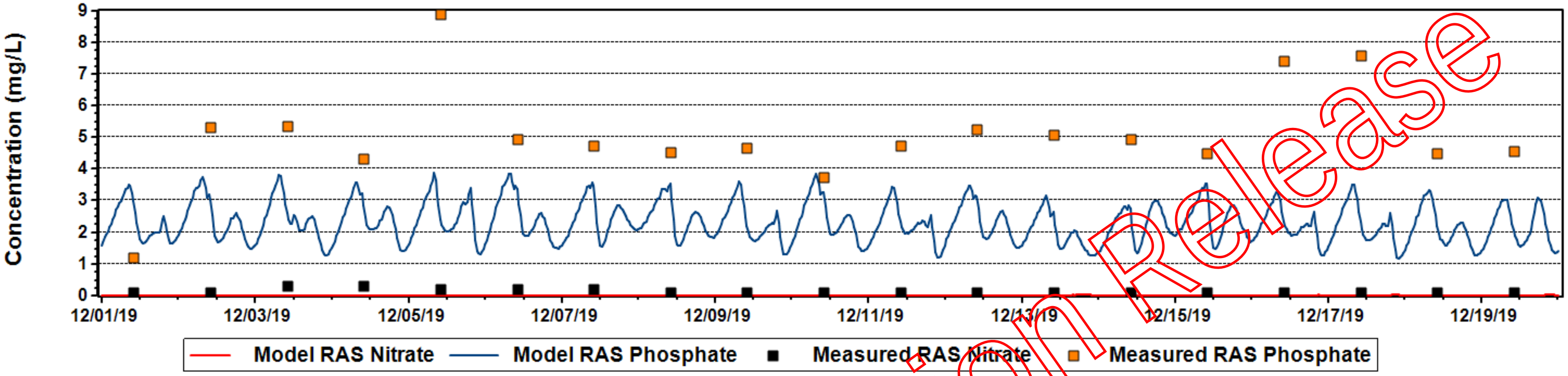

Figure 4-13: Victoria Point STP Dynamic Model Calibration – RAS Stream Nutrients

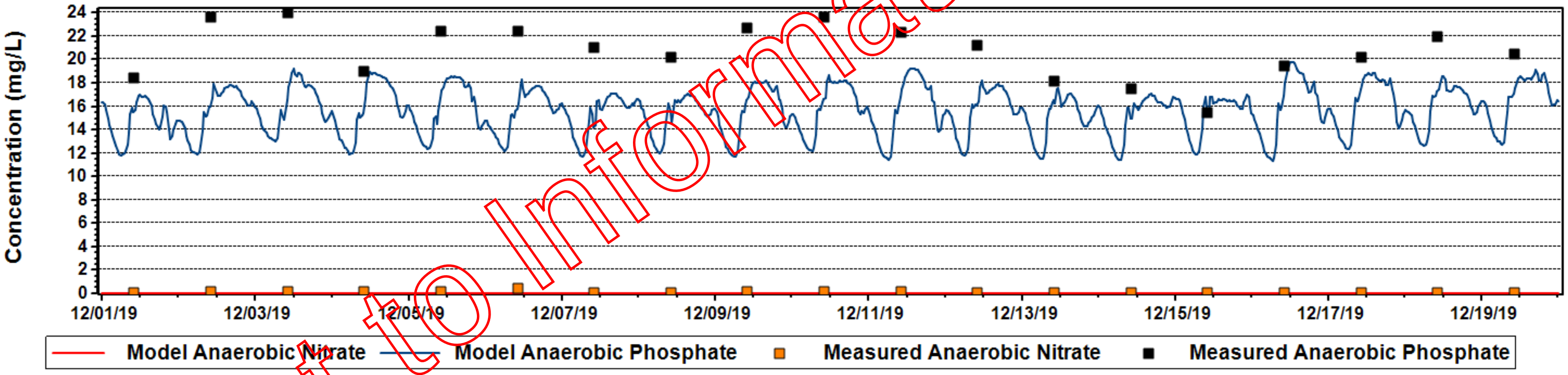

Figure 4-14: Victoria Point STP Dynamic Model Calibration - Anaerobic Zone Nutrients

5 EXISTING PLANT CAPACITY

5.1 HYDRAULIC CAPACITY

The existing plant has been modelled to identify the hydraulic capacity of the installed infrastructure. A report summarising the inputs, outputs, assumptions, and limitations of the hydraulic analysis is provided in Appendix A.

The assessment was based on the requirement of the Plant to pass 565 L/s (plus an additional 400L/s RAS in the relevant units), based on the following key assumptions:

- Per capita flow of 220 L/EP/d
- Design connected population of 44,398 EP, approximately equal to the 44,312 EP projected for 2041.
- Peak wet weather flow condition of 5xADWF
- Minimum freeboard of 300mm

Note that minimum freeboard of 500mm is routinely applied as the hydraulic design criteria for aerated vessels, but the 2003 design of the oxidation ditch (which features enclosed aerators) applied a minimum freeboard of 300mm which has been carried forward to this analysis.

Limitations on the system to meet the above requirements, as listed in the hydraulic report are:

- Inlet pumps - The existing pumps operating in a duty/assist configuration have an estimated peak capacity of 525 L/s. This is substantially less than the 565 L/s required to meet the design criteria adopted for upgrade planning. Additionally, it is anticipated that the existing pumps will suffer from cavitation under this operating condition.

  However, two new pumps have been ordered for the Victoria Point WWTP inlet pump stations, and are expected to be delivered and installed in August 2020[1]. The new pumps have been sized to deliver 300 L/s with a single duty unit, and 550 L/s with both units operating at the nominated top water level in the pump stations.

  However, RCC have advised that the selected inlet pumps will theoretically deliver the 565 L/s required once the water level in the well increases to 0.5m above the normal top water level. This level would still be 1.0m below surcharge. At 1.5m above the normal top water level (i.e. the level at which surcharge commences), the pumps are expected to deliver a combined flow of approximately 590 L/s. As such, the upgraded pumps will be sufficient for the projected 2031 load if the South West Victoria Point and Weinam Creek developments proceed.
- Inlet channel – The limited availability of information concerning the losses through the step screen, grit screw and grit trap, has prevented verification of their capacity in the hydraulic model. However, experience during extreme wet weather events indicates the inlet works has sufficient capacity for the peak influent sewage flow delivered by the existing raw sewage pumps (~525 L/s). Further, the change in raw sewage screens identified under this project will provide scope in increase hydraulic capacity through inlet screening channels.
- Filter feed pumps – The performance data from the existing pumps provided does not match the analysis for single pump duty. Due to continuous and variable rate of discharge of flows to the filter feed tank, and the lack of flow measurement on the filter inflow or outflow, it has not been feasible to independently verify the actual flow delivered by the filter feed pumps in operation, or their capacity.
- Filters – The existing filters may not be sufficient to meet the entire 3xADWF capacity applied to the 2003 upgrade design. However, it is noted that filtration of flows to 3 x ADWF is not specifically required for licence compliance, and acceptance of a lesser peak throughput is anticipated to be sufficient for this process unit based on the licence requirements and frequency of wet weather events.

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[1] The new pumps are Wilo 55 kW 6 pole FA25.93T pumps with FK34.1-6/33 motors.

Subsequent to the hydraulic analysis, the RAS flow achieved by the existing pump stations was measured on-site. With two of the three pumps in each pump station operating simultaneously at 100% speed (4 pumps in total), RAS pump station 1 delivered 190 L/s, and RAS Pump Station 2 delivered 193 L/s, giving a total RAS flow of 383 L/s. The RAS channel and screen adjacent to the anaerobic zone managed this flow without issue or exceedance of freeboard limits.

Table 5-1: Victoria Point STP – Summary of Existing Process Unit Hydraulic Capacity

Unit	Hydraulic Assessment Flow Capacity of unit at minimum freeboard (L/s)
Raw Sewage Pump Capacity	525 (Existing Pumps), 550 L/s (from August 2020)
Inlet channel to Anaerobic Reactor pipe	727 L/s
Pipe oxidation ditch to Mixed Liquor Distributor	1460 L/s
Mixed Liquor Distributor Weir	1400 L/s
Mixed Liquor Distributor to Clarifier	Including RAS: 517 L/s (per clarifier) Total required: 489 L/s (per clarifier)
Pipe from Clarifier to Filter Feed Tank	754 L/s
Filter Feed Tank to Filters	Unable to be confirmed.
Filter Hydraulic Capacity (estimation)	442 L/s
Filtered Water holding tank to chlorine contact tank inlet	1012 L/s
Chlorine contact tank outlet weirs	1610 L/s
RAS Pump Capacity	188 L/s (per pump station in original design) From Site Measurements: RAS Pump Station 1 155 L/s (one pump at 100% speed) 193 L/s (two pumps at 100% speed) RAS Pump Station 2: 120 L/s (one pump at 100% speed) 190 L/s (two pumps at 100% speed)
WAS Pump	1-8.3 L/s (depending on stator condition)
Dewatering filtrate return	73 L/s (Derived from SCADA Data for Pump Station)

5.2 SECONDARY TREATMENT PROCESS CAPACITY

5.2.1 Capacity Based on Sludge Production / Clarification

The nominal clarification capacity of the existing secondary treatment process was initially quantified using steady state process modelling and the Vesilind 1-D flux model. The following criteria and conditions were applied to the analysis:

1. Pollutant loads at Maximum Monthly Load (see Sections 3.3 and 3.4)
   As the maximum monthly influent load will correspond to the maximum sludge inventory within the system, this loading condition has been applied to the analysis. This is in line with typical process design practice.

2. Sludge age of 15 days (see Section 3.5)
   To maximise the capacity of the system while maintaining adequate nitrification and denitrification, an operating sludge age of 15 days has been applied. This sludge age was determined based on analysis of the performance of the existing plant and confirmed with the calibrated dynamic process model. This sludge age exceeds the minimum required for application of the "barrier option" under the end of waste code.

3. Mixed liquor temperature of 19.5°C (see Section 3.6.1)
   The maximum sludge inventory corresponds to the minimum mixed liquor temperature. This figure was drawn directly from the plant log, and represents the typical sustained minimum value during the winter months.

4. Settleability at 80th percentile of Valid Monitoring Results (see Section 3.5)
   The 80th percentile of the valid settleability monitoring results measured on-site from 2013-19, 205 mL/g DSVI, has been applied to the capacity assessment.

5. De-rating of Clarifier Peak Surface Overflow Rate to account for non-idealities in full scale clarifiers
   The peak surface overflow rate has been de-rated by 20% to account for typical impact of non-idealities in the Vesilind Flux theory compared to full-scale stress test results (Ekama G. A., et al., 1997).

6. Sludge Storage in Secondary Clarifiers
   The steady state modelling included provision for the storage of sludge in the clarifiers up to a depth of 0.3m to the side wall. This depth of sludge blanket is somewhat less than measured under recent operations, but is considered a suitably conservative basis for analysis. Sludge storage in the clarifiers serves to increase the clarification capacity by reducing the mixed liquor solids concentration in the clarifier feed. The solids concentration in the clarifier blanket was assumed to the same as the concentration in the mixed liquor.

7. Treatment of Flows up to 5 x ADWF (see Section 3.6.7)
   In line with the design basis applied to the 2003 upgrade, the upgrade planning has been based on transfer and full treatment of all flows up to five times the average dry weather flow (at 220 L/EP/d).

8. Peak Capita Flows at 220 L/EP/d (see Sections 3.2 and 3.4).
   In line with the design basis applied to the 2003 upgrade, the upgrade planning has been based on transfer and full treatment of all flows up to five times the average dry weather flow (at 220 L/EP/d).

The solids removal capacity of the existing Victoria Point secondary treatment process based on these conditions is summarised in Table 5-2.

Table 5-2: Secondary Treatment Process Capacity based on Solids Clarification

Parameter	Units	AAL	MML
Capacity	ML/d ADWF	9.42	8.43
	L/s PWWF	545	488
	EP	42,800	38,300
Maximum Surface Overflow Rate (including derating for non-idealities)	kL/m²/h	1.080	0.969
Minimum RAS Ratio from Vesilind Flux Model	Ratio	0.54	0.61
Minimum RAS Flow Required	L/s	295	298
RAS Flow Available in Existing Plant (2 No. RAS Pump at 100% Speed in each RAS Pump Station)	L/s	383	

5.2.2 Ability of Existing Plant to Meet Effluent Total Nitrogen Mass Load Limit

The calibrated dynamic process model has been used to assess the ability of the existing plant to achieve the nitrogen removal requirements at the planning horizon. The results of this analysis are summarised in Table 5-3.

In considering the results (and validating against actual plant performance), it is important to note that the maximum per capita flow is effectively the most stringent assessment criteria for annual compliance (as it results in the lowest effluent total nitrogen requirements). By contrast, the compliance with the less-stringent short-term concentration limit has been assessed at both the minimum and maximum per capita flows.

Table 5-3: Existing Secondary Treatment Process Nitrogen Removal Performance Limits

Loading Condition	Connected Population (EP)	Per Capita Flow (L/EP/d)	Temperature (°C)	Ammonia as N (mg/L)	NO_3 as N (mg/L)	rDON (mg/L)	Total N(mg/L)	Required TN (mg/L)
AAL	44,312	220	23.9	0.36	0.40	0.67	1.43	1.38 Note 1
MML	44,312	220	19.5	0.46	0.70	0.91	2.07	3 mg/L (Short term median @St.2) Note 2
MML	44,312	153	19.5	0.45	0.69	0.91	2.05	

Note 1: See Figure 3-10 in Section 3.6.3.
Note 2: See Section 3.6.1 for additional discussion of the exceedance of the mass load limit for periods much less than 12 months.

Key conclusions of this analysis include:

- Under the projected sewage loads imposed by the South West Victoria Point and Weinam Creek developments, the modelling predicts an increase of just 0.09 mg/L in effluent total nitrogen under AAL conditions. However, due to increased flows, the reduction in the effective total nitrogen limit to stay under 13.5 kgN/day pushes the plant into non-compliance. Effectively, the additional load imposed by the South West Victoria and Weinam Creek developments are very likely to result in the plant exceeding its mass load discharge limit for Total Nitrogen.
- The per capita influent sewage flows have only a marginal impact on the predicted effluent quality under the MML loading scenario. The existing plant is capable of meeting the short-term concretion limits for total nitrogen under these critical loading conditions.
- As rDON represents a significant portion of the effluent TN limit, any sustained increase in the rDON concentration represents a risk to licence compliance under every operating scenario.

Analysis of the existing plant operations indicates median effluent nitrogen of up to 1.9 mg/L has been observed under recent operations. However it is important to note that operations under dry conditions effectively increases the permissible effluent nitrogen concentration. In wet years the observed effluent nitrogen concentration decreases to 1.5 mg/L or less, but the discharge requirements become more stringent due to the increased flow and mass load licence.

Based on the analysis undertaken in Section 3.6.3, the highest 365-day average mass load discharged by the plant under recent operations was 10.7 kg/d, which occurred in January 2016. Based on this figure and the estimated connected population in 2016, it is anticipated that the "real-world" nitrogen removal capacity of the plant is approximately 38,700 EP, which is broadly consistent with the overall conclusions of the dynamic process model.

5.2.3 Capacity Based on Aeration

The aeration system must provide sufficient dissolved oxygen to oxidise the influent COD and TKN, and maintain the dissolved oxygen concentrations required for proliferation of the organisms which undertake these processes. An analysis of the modelling undertaken for Section 5.2.2 was undertaken to establish the likely capacity of the existing aeration system. To consider the aeration limitations within the dynamic process model, the maximum power for each aerator was directly specified as a part of the model development.

In assessing the aeration capacity of the secondary treatment process, it is important to differentiate between the total installed aerator capacity, and that which can be used while meeting overall nitrogen removal requirements. At Victoria Point the Dissolved Oxygen concentration near the end of the aerobic zone must be relatively low to enable adequate denitrification performance – both in terms of denitrification within that portion of the bioreactor itself, and in reducing the oxygen discharged to the anoxic zone portion of the ditch.

Key conclusions of this analysis included:

- At a load of 44,312 EP (and MML), as projected for 2041, the target dissolved oxygen concentration within the oxidation ditch is not maintained throughout the day, with Aerators No. 1 and 2 operating at the maximum output for most of the daytime period. Under this scenario the total effluent nitrogen increases, but the model predicts it will remain compliant with the Short-Term median total nitrogen concentration limit of 3 mg/L on a 24-hour composite basis at MML. This suggests that at this load, the plant is essentially operating at (or marginally above) its aeration capacity, with absolutely no reserve.
- Subsequent model runs demonstrated that nitrogen removal performance could be maintained by operating the third aerator at very low output for a portion of the day. This operating strategy relies on simultaneous nitrification-denitrification throughout the bulk of the ditch to meet the nutrient removal requirements, which is likely to be difficult to robustly replicate under real-world operating conditions. Further, operational regimes which rely on operation of all three aerators would not necessarily provide a suitable operating risk given criticality of aeration to effluent quality.

5.3 SUMMARY OF EXISTING PLANT CAPACITY

The overall process capacity of the Victoria Point STP, as compiled from the analyses in Sections 5.1 and 5.2, is summarised in Table 5-4. As noted in the table, the prevailing plant capacity, pending the upcoming upgrade of the raw sewage pumps and dewatering system, is limited to 38,300 EP by the ability of the secondary clarifiers to treat 5 x ADWF. The ability of the process to maintain compliance with the Total Nitrogen Mass Load Limit will be compromised at a similar load (38,700 EP).

Table 5-4: Victoria Point STP – Summary of Capacity by Process Unit

Process Unit	Value	Notes
Loading Scenario	Maximum Monthly Load	
Per Capita Flow	220 L/EP/d	Nominal maximum of range in Basis of Planning
PWWF / ADWF	5.0 x ADWF	As defined in plant licence for entire plant liquid stream.
Inlet Works - Overall		41,240 EP, 9.07 ML/d ADWF
Existing Raw Sewage Pumps	*41,240 EP*	Capacity based on existing combined pump capacity of 525 L/s (Duty/Assist) (see Section 5.1)
New Raw Sewage Pumps	43,200 EP	Capacity based on new pumps of 550 L/s (Duty/Assist) to be installed in August 2020 (see Section 5.1)
Influent Sewage Screening	43,910 EP	559 L/s
Grit Removal	69,120 EP *36,520 EP*	880 L/s based on manufacturer rating 465 L/s based on 1.5 m/minute rise rate (conservative)
Secondary Treatment - Overall		38,300 EP, 8.43 ML/d ADWF
Clarification	38,300 EP	At 15 days sludge age, MML loading conditions
Nitrogen Removal	38,700 EP	Based on Total Nitrogen Mass Load Limit of 13.5 kg/d
Aeration	~44,300 EP	Based on nitrogen removal capacity with two aerators operating at 100%.
Hydraulic Capacity	>44,312 EP	
Effluent Disinfection and Discharge- Overall		38,300 EP, 7.48 ML/d ADWF
Tertiary Filters	*35,350 EP to 3.0 x ADWF* *37,100 EP at 2.8 x ADWF* *44,310 EP at 2.4 x ADWF*	270 L/s Capacity. Filtration of all flows not required for licence compliance with the retention of chlorination.
Effluent Disinfection	38,700 EP	Required for Residual Chlorine <0.7mg/L when secondary effluent ammonia must be reduced to maintain compliance with effluent Total Nitrogen Mass Load Limit (see Section 6.3).
Biosolids Handling - Overall		44,300EP, ML/d ADWF
Existing GDD/BFPs	*>44,300 EP*	Duty/Assist, 5 hours/day, 6 days/week
New Dewatering Machines (procurement in progress)	44,300 EP	Duty/Standby, 11.2 hours/day, 5 days/week (Duty Only)
Overall Plant Capacity		38,300 EP, 7.48 ML/d ADWF
Overall Existing Plant Capacity	38,300 EP	Limited by secondary treatment clarifier capacity, noting that nitrogen removal capacity (and chlorine contact tank capacity by corollary) is only marginally higher.

Note 1: Italicised figures are not considered to limit overall plant capacity.

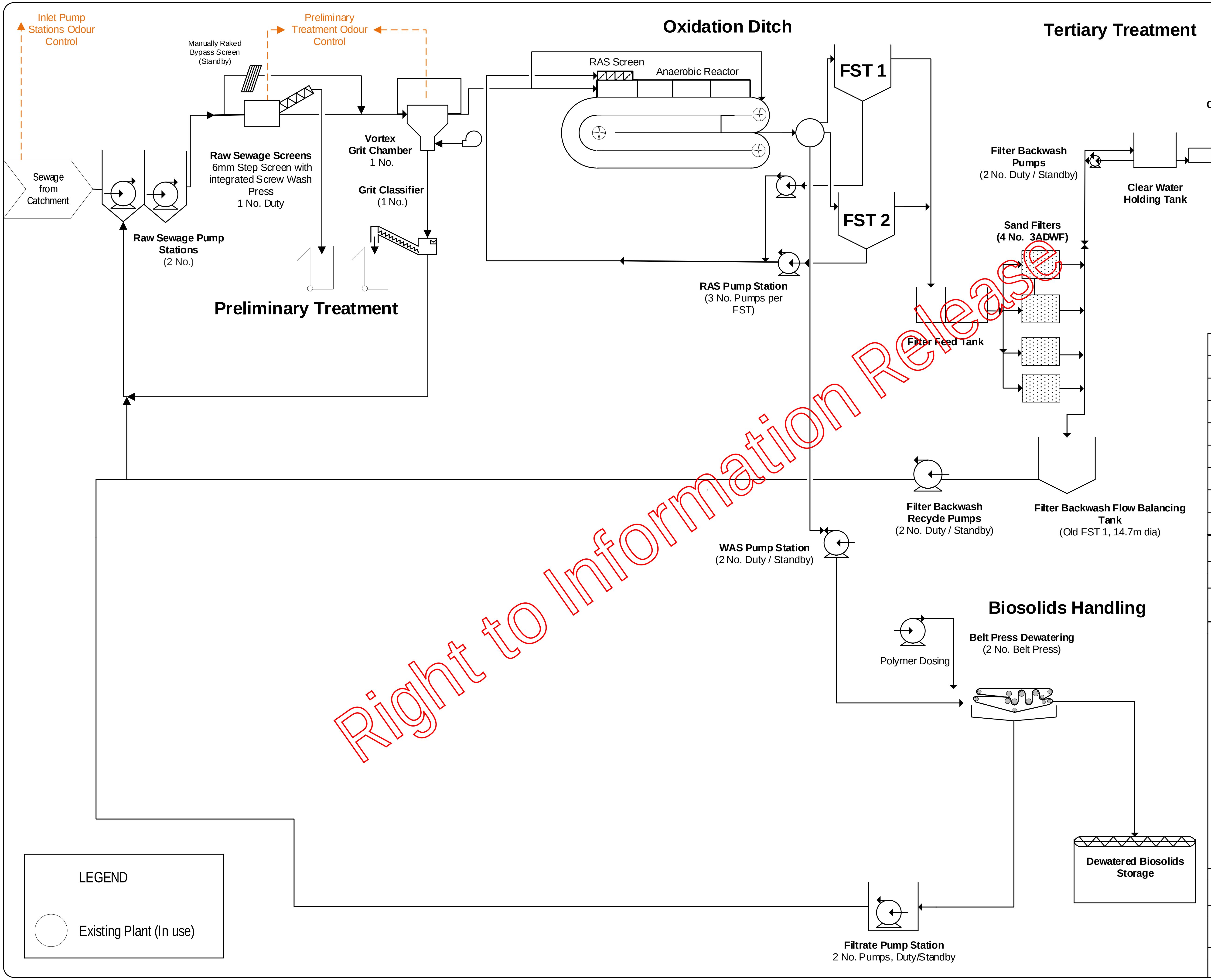
5.4 SUMMARY OF REQUIRED UPGRADE WORKS

The planning investigations and concept design have identified a suite of additional works required to manage the additional loads associated with the South West Victoria Point and Weinam Creek development (44,312 EP). The works required, and the associated staging of works, are summarised in Table 5-5 and Figure 5-2 overleaf.

Table 5-5: Summary of Required Plant Upgrades and Staging

Upgrade	Infrastructure	Required from	
Increased Nitrogen Removal	Post-Anoxic / Re-Aeration Zone)	38,700 EP	2025
Additional Solids Settling Capacity	1 No. Additional Secondary Clarifier	38,300 EP	2024
Additional Disinfection Capacity	1 No. Additional Chlorine Contact Tank	38,700 EP	2025

Completion of the works to service the developments is required to be completed and in service by 2024-25. This suggests the works should be undertaken as a single stage and under a single contract, with procurement and design commencing in 2020-21.



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DRAWN	CHECKED	REVIEWED
RAS	DJF	DJF

APPROVED: DJF

DATE: 08/05/2020

SCALE: Not To Scale

TITLE:
Victoria Point Treatment Plant – Existing Plant
Process Flow Diagram

TYR GROUP TREATMENT AND REUSE

Redland WATER

DRAWING STATUS: For Upgrade Planning

DRAWING NUMBER: VP-STP-PFD-001

REVISION: A

ORIGINAL SHEET SIZE: A3

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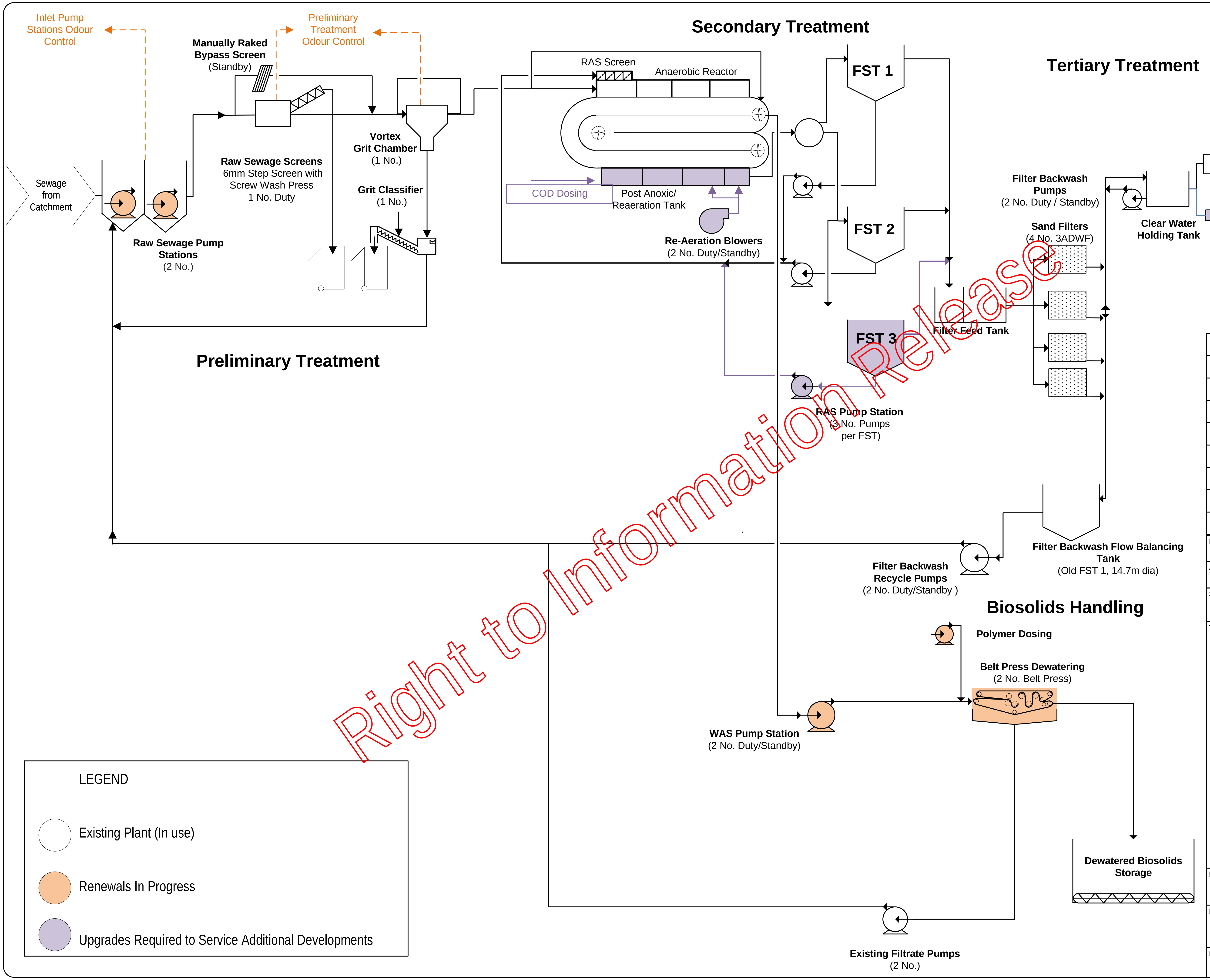

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No	REVISIONS	DRAWN	APPRVD	DATE

DRAWN	CHECKED	REVIEWED
RAS	DJF	DJF

APPROVED: DJF
DATE: 08/05/2020

SCALE: Not To Scale

TITLE
Victoria Point Treatment Plant – Option 2, 20 day Sludge Age, Additional Clarification
Process Flow Diagram

TYR GROUP
TREATMENT AND REUSE

Redland WATER

DRAWING STATUS: For Upgrade Planning

DRAWING NUMBER: VP-STP-PFD-002

REVISION: A
ORIGINAL SHEET SIZE: A3

HTTPS://TYRGROUP.SHAREPOINT.COM/SITES/ARCHIVEDPROJECTS2/SHARED DOCUMENTS/1904 - VICTORIA POINT UPGRADES/DRAWINGS/VICTORIA POINT STP - PROCESS FLOW DIAGRAM - REV A - FOR NEW DEVELOPMENTS ONLY.VSD

6 PLANT UPGRADES REQUIRED TO SERVICE NEW DEVELOPMENTS

The process selection and concept design of the upgrades required to treat the increased sewage loads associated with the new developments are summarised in the following sections.

6.1 INCREASED NITROGEN REMOVAL

6.1.1 Options Identification and Short-Listing

The concept design includes augmentation to reduce effluent total nitrogen concentrations to meet the mass load limit at loads in excess of 38,700 EP. Should both the South West Victoria Point and Weinam Creek developments proceed, these works are projected to be required by the end of 2025.

The options to enhance the nitrogen removal process within the existing plant were the subject of an identification and short-listing process to identify the preferred solutions to be carried forward for more detailed analysis. As detailed in Section 3.6.4, the ammonia, oxidised nitrogen, and refractory nitrogen fractions of the total nitrogen in the plant effluent indicate that there is substantial potential for the nitrogen concentrations to be reduced further using conventional processes. The long-list of options considered is summarised in Table 5-3.

In addition to the treatment options, it is important to note that compliance with the licence could also be achieved through a number of alternative options which accommodate higher effluent total nitrogen concentrations. As discussed in Section 3.6.1 the assimilative capacity of Eprapah Creek is currently being modelled as a background to the future development of the plant. Depending on the results of the modelling, and subsequent negotiations with the DES, potential solutions include:

- Renegotiation of the Stage 2 Nitrogen Mass Load Limit based on the impacts of nitrogen loads (see Section 3.6.1);
- Increased effluent reuse to reduce the volume of flow discharged to Eprapah Creek;
- Relocation of the effluent discharge point closer to the mouth of Eprapah Creek (where dilution with tidal flow is increased);
- Installing effluent storage to enable effluent discharge to be limited to the ebb-tide periods (during dry weather).

The viability of these options depends on the results of the environmental assessments, and if feasible, have the potential to deliver greater value. They remain outside the scope of the upgrade investigations. It is recommended that the development and assessment of these alternative options be pursued if their viability is confirmed through the current environmental investigations.

Table 6-1: Summary of Options Identification and Short-Listing for Enhancing Nitrogen Removal

Option	Advantages	Disadvantages	Carried Forward
Treatment Plant Options			
Dry Weather Flow and Load Attenuation (Influent Balance Tank)	• Proven, well understood technology applied at multiple STPs in SEQ to target very low effluent total nitrogen. • Provides opportunity to optimise operations through reducing plant dynamics and shifting power demand from peak to off-peak periods.	• Plant already achieves very low effluent ammonia • Does not provide additional wet weather treatment capacity • High Capex due to large tankage (2.5 to 3 ML) and odour control (15,000-24,000 m³/h) required • Not likely to be as effective as other solutions within Victoria Point STP's existing configuration.	✗
Post-Anoxic / Re-Aeration Tank	• Proven, well understood technology. Reliably provides supplemental nitrification and denitrification. • Existing oxidation ditch has been configured specifically to enable post-anoxic tankage to be readily added. • Denitrification performance can be efficiently supplemented with chemical substrate (e.g. sugar), eliminating the risk in influent characteristics • Provides a minor increase in solids removal capacity through increasing bioreactor volume • Enables structural issues in one section of the existing oxidation wall to be resolved.	• Additional access road required for maintenance of new equipment in post-anoxic / reaeration zone.	✓
Ozone and BAC	• Well developed, mature technology • Robust additional nitrogen removal • Small footprint	• High energy and materials consumption • Significant additional process complexity compared to alternatives and existing STP.	✗
Reverse Osmosis	• Well developed, mature technology • Can robustly achieve the required levels of nitrogen removal • Small footprint	• No sink available for the nitrogen removed with the RO system and brine stream • High energy consumption	✗

The two options which were considered in detail are discussed further in the following sections.

Post-Anoxic/Re-Aeration Tank

Additional denitrification can be achieved through addition of further bioreactor tankage at the downstream end of the existing oxidation ditch to provide:

1. A post-anoxic zone, where oxidised nitrogen can be denitrified under anoxic conditions. The substrate to drive this additional denitrification is generated through the death and lysis of organisms within the biomass, and if required, augmented by dosing of additional substrate to drive rapid denitrification.
2. A re-aeration zone, to oxidise any ammonia released through death/lysis of organisms in the post-anoxic zone, drive additional biological P uptake as required, and deliver the mixed liquor to the clarifiers with sufficient dissolved oxygen.

Based on experience in the design of comparable systems, the optimal post-anoxic zone generally comprises a mass fraction of 6-9%, and the optimum re-aeration zone approximately 2-3% mass fraction.

Influent Sewage Dry Weather Balance Tank

Conventional biological nitrogen removal processes generally have a peak in effluent ammonia associated with diurnal peak flow period. As nitrifying organisms are very slow growing, they are unable to respond to large scale increases in nitrogen load above the average (as occur during the diurnal peak). As a result, normal dry weather flows generally see the effluent ammonia increase for a few hours during and after the peak loading period. Additionally, effluent nitrate generally increases for many hours after the peak in effluent ammonia due to the nitrification of the excess ammonia in the absence of the substrate required to denitrify it. The balancing of influent sewage flows during dry weather enables the peaks in both effluent ammonia and effluent oxidised nitrogen to be avoided, reducing effluent total nitrogen (on a 24-hour basis).

Dry weather influent sewage flow balancing is used at a number of sewage treatment plants in South East Queensland, including Murrumba Downs, Cooroy, and Pimpama. These facilities demonstrate the capability of load balancing to deliver very low effluent ammonia and nitrate.

A dry weather balancing tank at Victoria Point would need to be approximately 2.5-3 ML in working volume, and would seek to attenuate the sewage flows to the secondary treatment process to between approximately 80% and 120% of the average. In wet weather, the balance tank would generally fill, and flow attenuation would cease. Flow would be pumped from the tank to the inlet works / secondary treatment process by relatively low head pumps. Due to the configuration of the existing raw sewage pump stations at Victoria Point, a balance tank is likely to be most cost effectively delivered as an additional (very large) wet well for these pump stations.

Due to the odours associated with storage of sewage, it is anticipated that a balance tank at Victoria Point STP would need to be fully enclosed and maintained at a negative pressure by an odour control facility. Due to the large volume of air within the balance tank, and its potential rate of filling, the odour control system required to ensure licence compliance would be of substantial scale. Mixing of the balance tank would also be required to ensure that it balances load (rather than just flow).

A Post Anoxic/Re-Aeration Tank has been adopted as preferred solution for Victoria Point as:

- The need for and potential benefits of an influent dry weather sewage balance tank are limited by the very low effluent ammonia already achieved by the plant. A balance tank can also be used to deliver lower effluent nitrate (as required to reduce overall effluent total nitrogen), but not as efficiently or robustly as a Post Anoxic Zone / Re-Aeration zone (with substrate dosing if required).
- The capital and operating costs associated with a balance tank will be larger due to the need for:
    - An odour control system of substantial capacity;
    - Construction of a 2.5-3.0 ML tank (compared to a 0.85-0.90 ML post-anoxic / reaeration tank), including corrosion protection, and,

  - Additional scope in pipework and existing asset modifications.
- The additional wet weather treatment capacity provided by the post-anoxic /reaeration tank (which is not provided by the balance tank option).
- The potential to use the new post-anoxic / re-aeration tank to provide additional cover over the reinforcement in the eastern side of the existing oxidation ditch wall.

6.1.2 Post-Anoxic / Re-Aeration Tank Concept Design

The Post Anoxic zone will comprise three cells, complete with sugar dosing to the first zone if required. Each cell will contain a high-speed compact mixer to maintain the solids in suspension. The Re-Aeration cell will be located downstream of the oxidation ditch and will be serviced by two blowers, diffused aeration and one DO meter. Additionally, the third post-anoxic cell will be fitted with aeration to enable it to operate under anoxic or aerobic conditions as process requirements vary.

The outlet pipework from the existing oxidation ditch outlet has been specifically configured to enable the future addition of a post-anoxic/re-aeration tank on the eastern side of the existing structure. This tank may be cast against the existing reactor to provide some additional cover to the reinforcement of the oxidation ditch, which is showing surface cracking.

Key considerations in the design of the of the Post-Anoxic / Re-Aeration Tank included:

- A post-anoxic zone that is large enough (and compartmentalised) to deliver efficient substrate utilisation in denitrification, but not so large that all nitrate is exhausted well prior to the end of the zone (which can compromise biological phosphorus removal performance).
- Sufficient aeration capacity to fully oxidise any residual substrate and ammonia in the re-aeration zone.
- The provision to aerate the third Post-Anoxic cell under reduced loading conditions to prevent anaerobic conditions (and associated loss of biological phosphorus removal performance).
- Provision of an overall increase in bioreactor volume to deliver increase in wet weather treatment capacity.

Both the dynamic and steady-state process models have been used to support the development of the design for the Post-Anoxic / Re-Aeration Tank. The revised configuration of the model is shown in Figure 6-1.

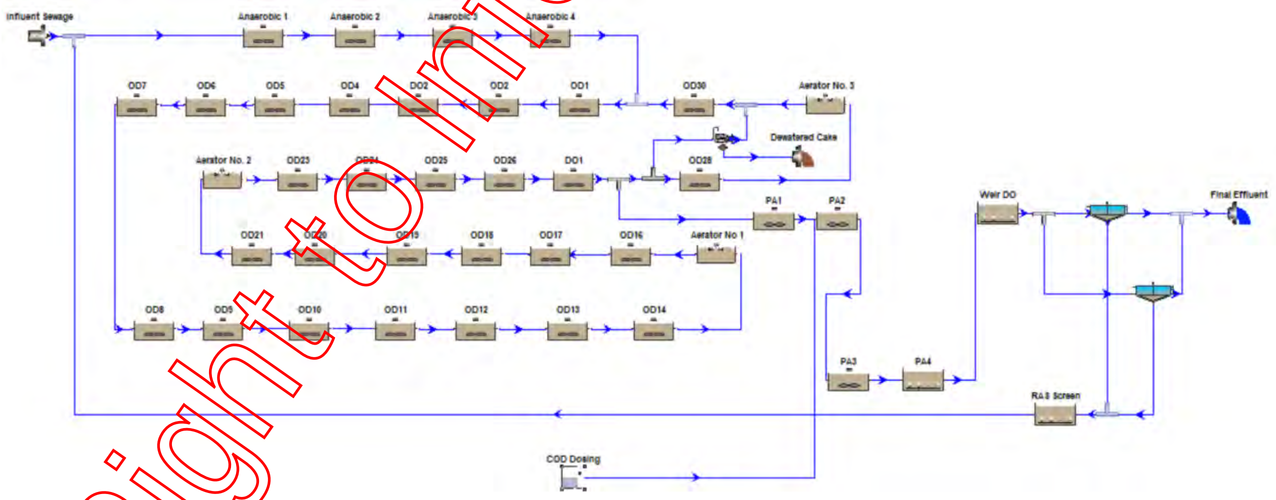


Figure 6-1: Dynamic Process Model including Post Anoxic Zone

The nitrogen removal performance and aeration requirements of the post anoxic zone are summarised in Table 6-2.

The model runs did not include the dosing of additional substrate, and indicated that no additional substrate will be required to achieve compliance with the effluent total nitrogen mass load limits. As a result, no facilities for substrate storage and dosing have been included in the concept design. Should substrate dosing be required in practice to manage operations, changed loading conditions, or drive to lower effluent total nitrogen, the existing Molasses Storage and Dosing Facility could be reconfigured for this purpose.

Table 6-2: Victoria Point STP – Post-Anoxic Zone Design - Dynamic Modelling Results

Loading Condition	EP	Temp (°C)	Effluent NH_3-N[1] (mg/L)	Effluent NO_3-N[1] (mg/L)	Effluent TN[1] (mg/L)	Mass Load (kg/d)	Ditch Setpoint (mg/L)	Re-Aeration Setpoint (mg/L)	Peak AOTR (kgO_2/hr)	Peak SOTR (kgO_2/hr)
AAL	44,312	23.9	0.08	0.28	1.03	10.1	0.6	2.0	10.6	32.2
MML	44,312	19.5	0.41	0.39	1.71	16.7	1.2	2.0	10.7	32.3
MML	44,312	28.0	0.05	0.30	1.26	12.3	0.4	1.2	12.0	32.5

Note 1: Based on 220 L/EP/d, 0.67 mg/L rDON at AAL, 0.91 mg/L rDON at MML

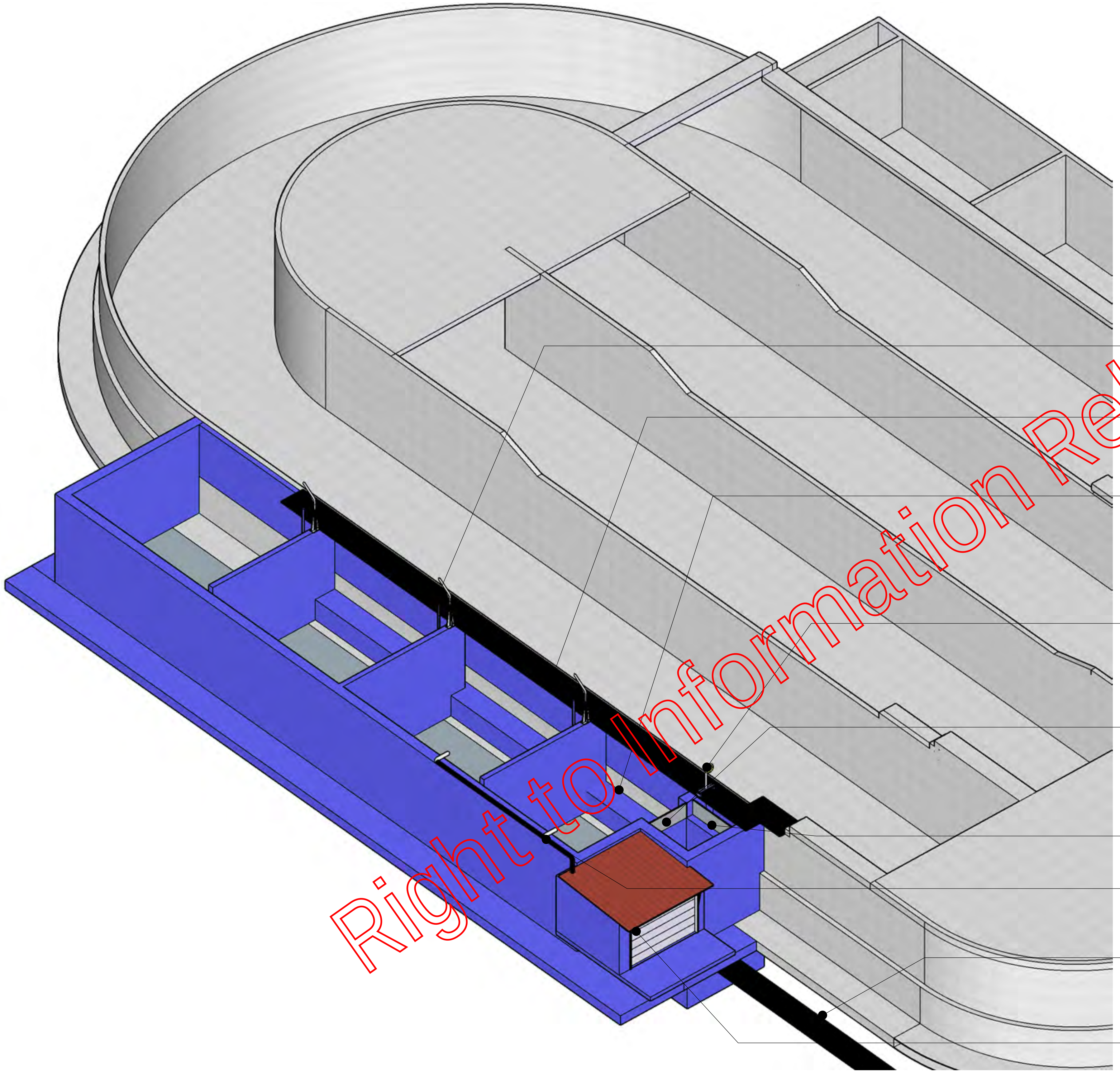
The concept design of the Post-Anoxic / Re-Aeration Tank is outlined in Table 6-3 and shown in Figure 6-2 through Figure 6-5.

Table 6-3: Schedule of Capital Works – Augment Reactor with Post-Anoxic/Re-Aeration Tank

Item	Works required
Civil Structure	• Mixed liquor transfer chamber and re-aeration zone outlet chamber • 3 No. Post Anoxic Cells o 2.6% mass fraction (250 kL) each cell o Internal dimensions 8.46m length x 7.20m width x ~4.1m water depth o Serpentine flow between cells • 1 No. Re-Aeration cell o 2% mass fraction (187 kL) o Internal dimensions 6.37m length x 7.20m width x ~4.1m water depth • Western wall of new tank formed against existing oxidation ditch wall • 500mm external wall thickness, 500mm floor thickness with 1.5m toe. • 250mm baffle wall thickness
Mechanical	• Aeration fitted to Post-Anoxic Cell 3 and Re-Aeration Zone o Fixed-to-floor fine pore membrane diffuser systems o Positive displacement blowers (2 No., Duty Standby, 500 Nm³/h per blower), fitted in dedicated room at corner of outlet chamber for noise control. Roller-door access for maintenance. o DN150mm spiral wound stainless steel aeration pipework o 1 No. actuated butterfly valve for control of air flow to Post-Anoxic Cell 3. • 1 No. high speed compact mixer in each post anoxic zone cell (3.7 kW each)
Instrumentation	• 1 No. DO meter (Re-aeration zone)
Pipework modification	• Modify mixed liquor pipework (chamber attached to ditch or pipework) • 1 No. Penstock / 2 No. Stopboards to bypass new tank as required for maintenance • Submerged duct in tank for mixed liquor transfer to Cell 1
Ancillaries	• New walkway on tank wall for access • Relocation of scum harvester to north of existing location required. • New access road to blower room and apron included in scope.

Key attributes of the design include:

- Construction of a new Mixed Liquor Transfer Chamber and Re-Aeration Zone Outlet Chamber directly over the existing DN960 mixed liquor pipe to the between the oxidation ditch and mixed liquor flowsplitter. The chambers extend to below the floor slab level of the existing bioreactor and enable the pipe to be encapsulated into walls of the new chambers around the existing 90-degree bend. Following completion of construction and wet commissioning of the Post-anoxic / Re-aeration tank, process commissioning of the system can be undertaken through:
    1. Isolation of influent sewage and RAS flow from the oxidation ditch;
    2. Raising of the existing outlet weir of the oxidation ditch;
    3. Emptying the existing DN960 mixed liquor pipe (through closing the penstocks and temporary pumping from the mixed liquor distribution chamber);
    4. Cutting the existing bend at the inlet and outlet of the new transfer chamber;
    5. Returning penstocks and weirs to their normal positions, and re-establishing normal flows to the oxidation ditch.
- A submerged square duct (constructed in concrete) is used to transfer mixed liquor from the transfer chamber to Post Anoxic Cell 1 (through the Re-Aeration Zone, and Post-Anoxic Cells 3 and 2). On discharge to the anoxic zone,
- The inlet to the duct is fitted with a normally-open penstock within the transfer chamber. Isolation and drainage of the Post-Anoxic / Re-aeration tank can be facilitated by closing this penstock, and opening a normally closed stopboard at the top of the transfer chamber to direct mixed liquor from the oxidation ditch direction to the outlet chamber of the new tankage. The design also includes a stopboard on the outlet weir of the Re-Aeration Zone to prevent backflow to the Re-Aeration Zone under this maintenance condition.



High Speed Submersible Mixers (1 in each Post-Anoxic Cell)

Grid Mesh Walkway for Mixer Access.

850mm x 850mm Square Duct to Post-Anoxic Cell 1

Penstock for isolation of Post-Anoxic Zone (normally open) - Face-mounted to 850x850 Duct

Stopboard for isolation of Post-Anoxic Zone on Re-aeration Zone Outlet Weir (normally open)

Stopboard for bypass of Post-Anoxic Zone (normally closed)

Aeration pipework (Re-Aeration Zone and Post-Anoxic Cell 3/Swing)

Existing DN960 Pipe to Mixed Liquor Flowsplitter

Blower Room with 2 No. Post-Anoxic Blowers

CLIENT
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A	For Phase 2 Planning	DJF	DJF	5/5/2020
B	For Phase 2 Planning - Final Report	DJF	DJF	25/6/2020

DRAWN	CHECKED	REVIEWED
DJF	IAF	IAF

APPROVED	DATE
DJF	25/6/2020

SCALE
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TITLE
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Post-Anoxic Zone
General Arrangement - Isometric

REVISION	ORIGINAL SHEET SIZE	DRAWING NUMBER
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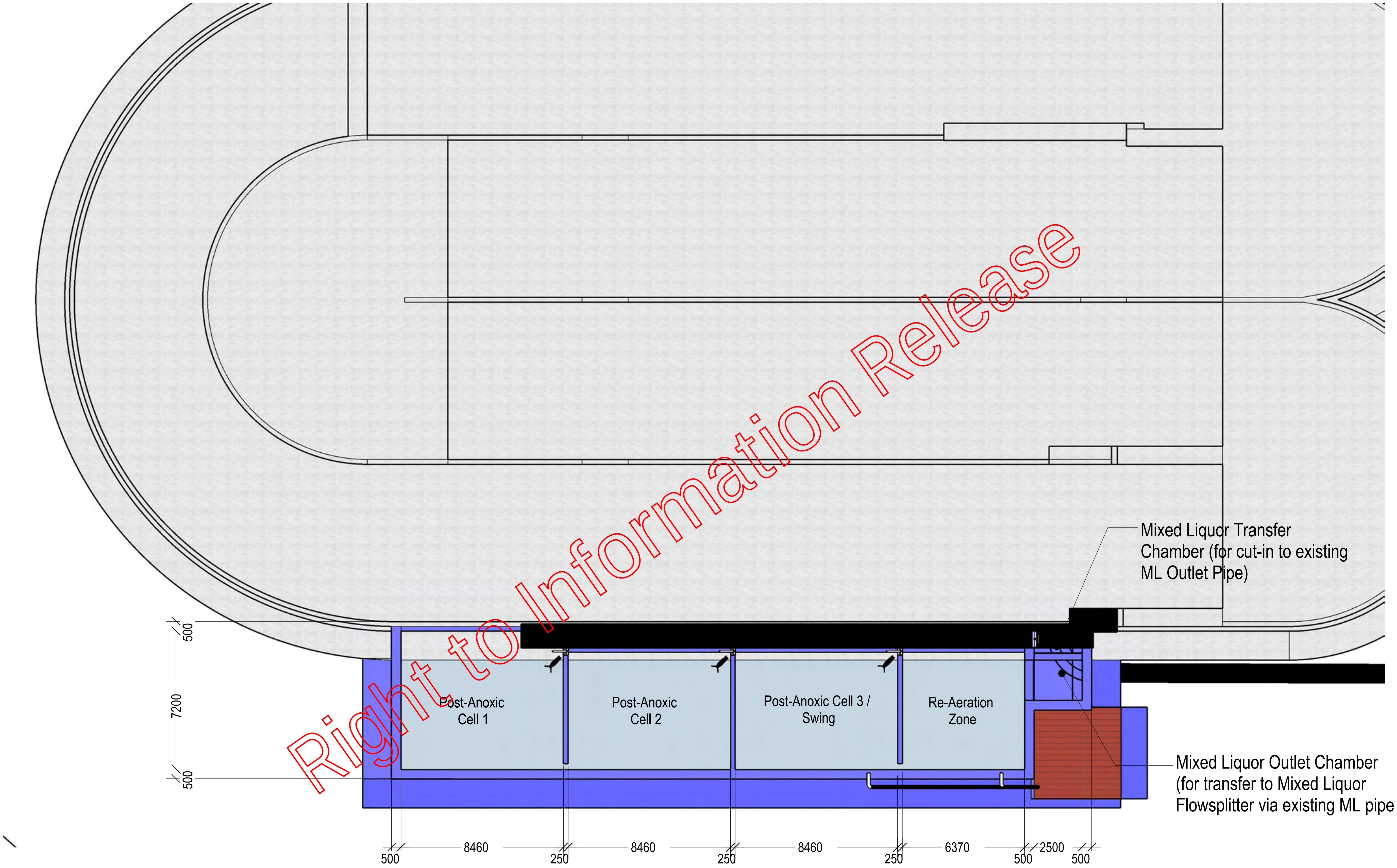

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DATE: 25/6/2020
SCALE: 1:200 at A3

TITLE
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Post-Anoxic Zone
General Arrangement - Plan, Walkway Level

REVISION	ORIGINAL SHEET SIZE	DRAWING NUMBER
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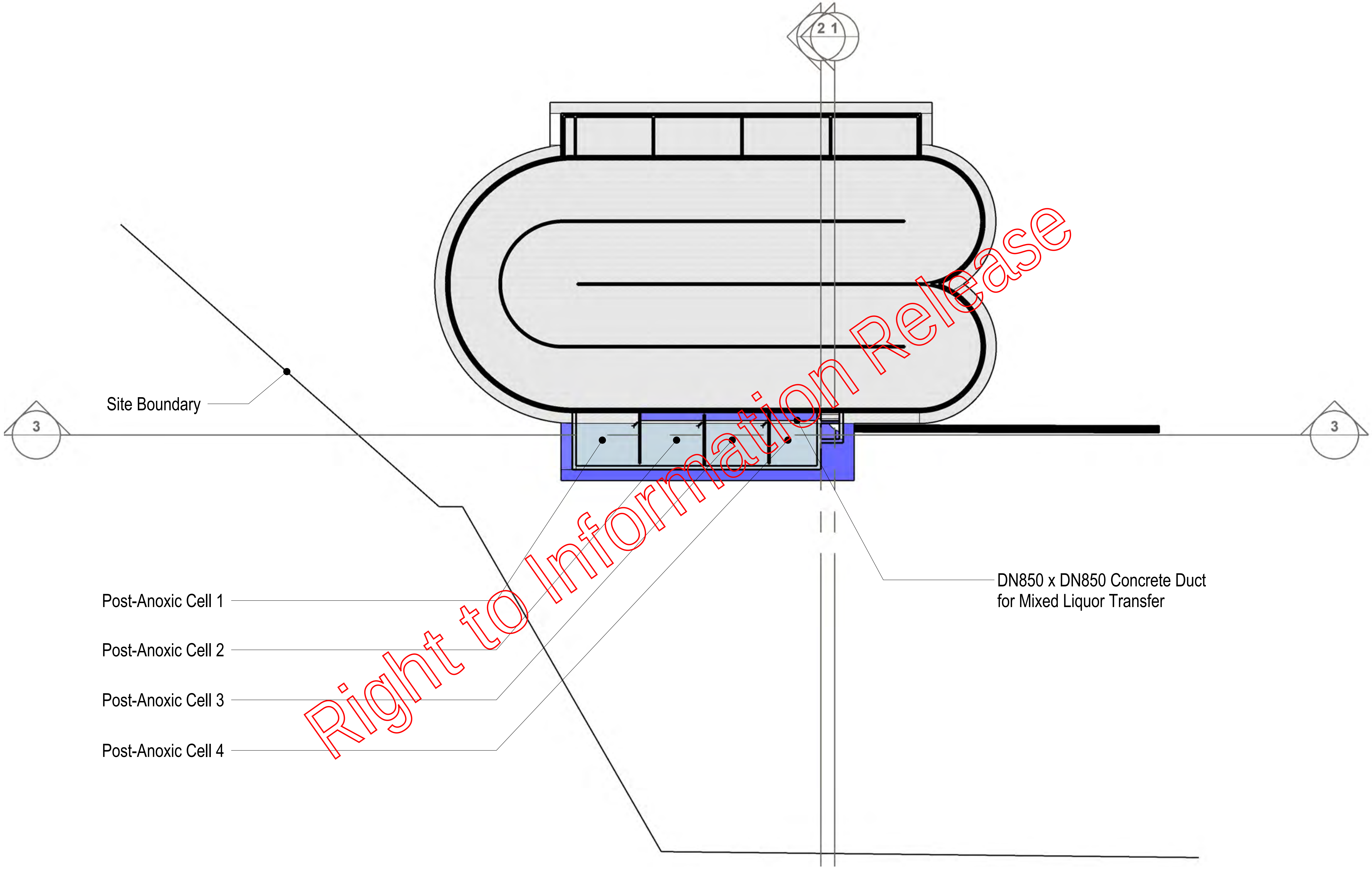

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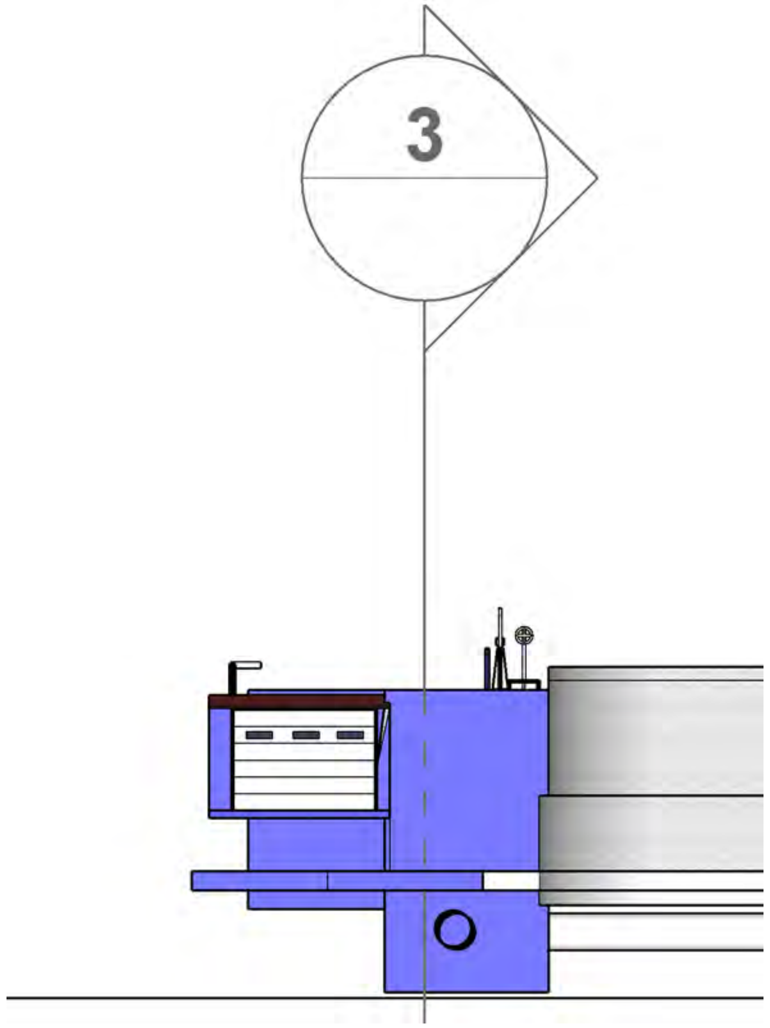
TITLE
Victoria Point STP Upgrade Planning
Post-Anoxic Zone
General Arrangement - Plan, Floor Slab Level

REVISION: B
ORIGINAL SHEET SIZE: A3
DRAWING NUMBER: J1904-03



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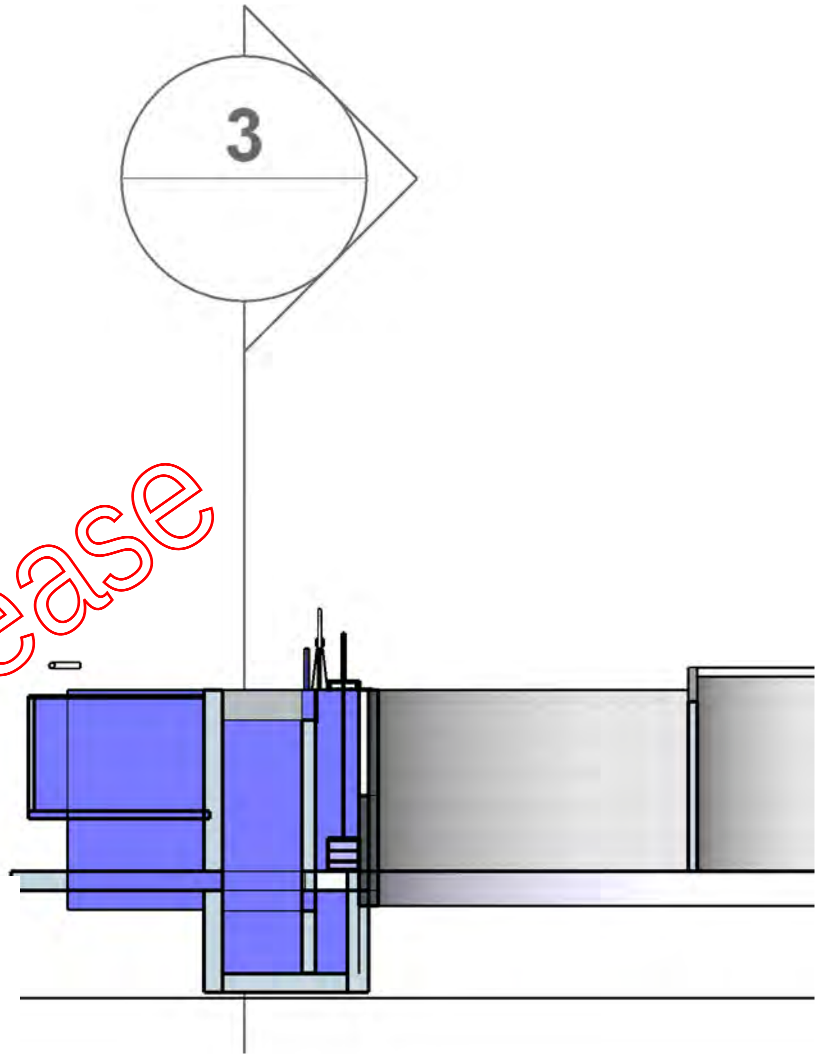

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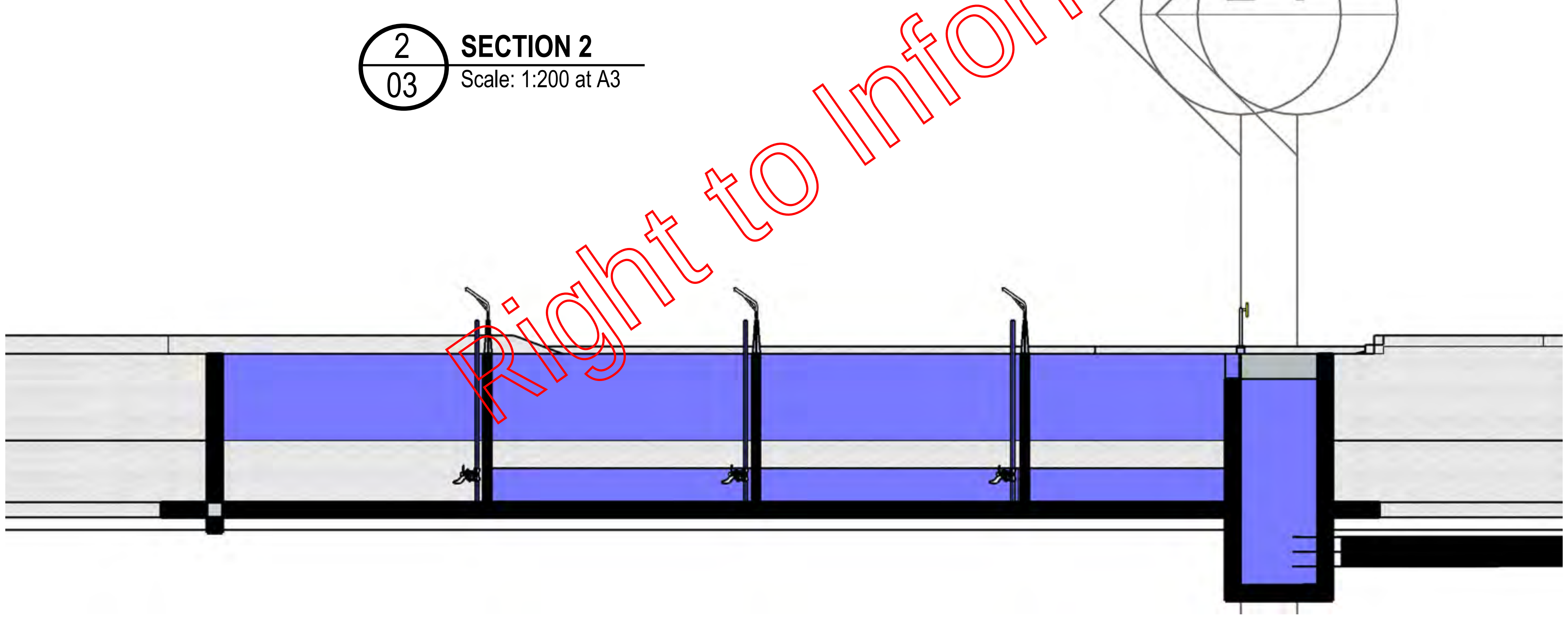

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SECTION 3
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TITLE
Victoria Point STP Upgrade Planning
Post-Anoxic Zone
Sections

REVISION: B
ORIGINAL SHEET SIZE: A3
DRAWING NUMBER: J1904-04

6.2 ADDITIONAL SOLIDS SETTLING CAPACITY

6.2.1 Description and Requirements

The concept design includes provision of an additional secondary clarifier to provide additional wet weather treatment capacity. Should both the South West Victoria Point and Weinam Creek development proceed, these works are projected to be required by the start of 2024.

Under the reduced operating sludge age made possible by the final End-of Waste Code (see Section 3.8), a single additional clarifier will be sufficient to manage flows and loads to beyond the 2041 planning horizon.

6.2.2 Additional Clarifier Concept Design

The additional secondary clarifier diameter has been set at a nominal 34.5m to match the existing final clarifiers, and provide ease of operation. At this sizing, the secondary treatment process capacity based on solids settling will be increased to 49,100 EP.

The concept design has located the clarifier immediately to the north of the existing units (in line with the master plan provided within the 2001 upgrade). This location leaves insufficient space for an access road to pass around the northern end of the new unit, or for the provision of additional berms to provide visual screening and noise abatement to the adjacent parkland. It is recommended that adjustment of the site boundary be considered to accommodate both of these elements during design development.

Based on the GIS overlays, it is not anticipated that the additional clarifier will require removal of any koala trees in the proposed location. The clarified effluent and RAS pipework alignment has been specifically defined to avoid removal of any of the koala significant trees located to the north-east of the existing clarifiers.

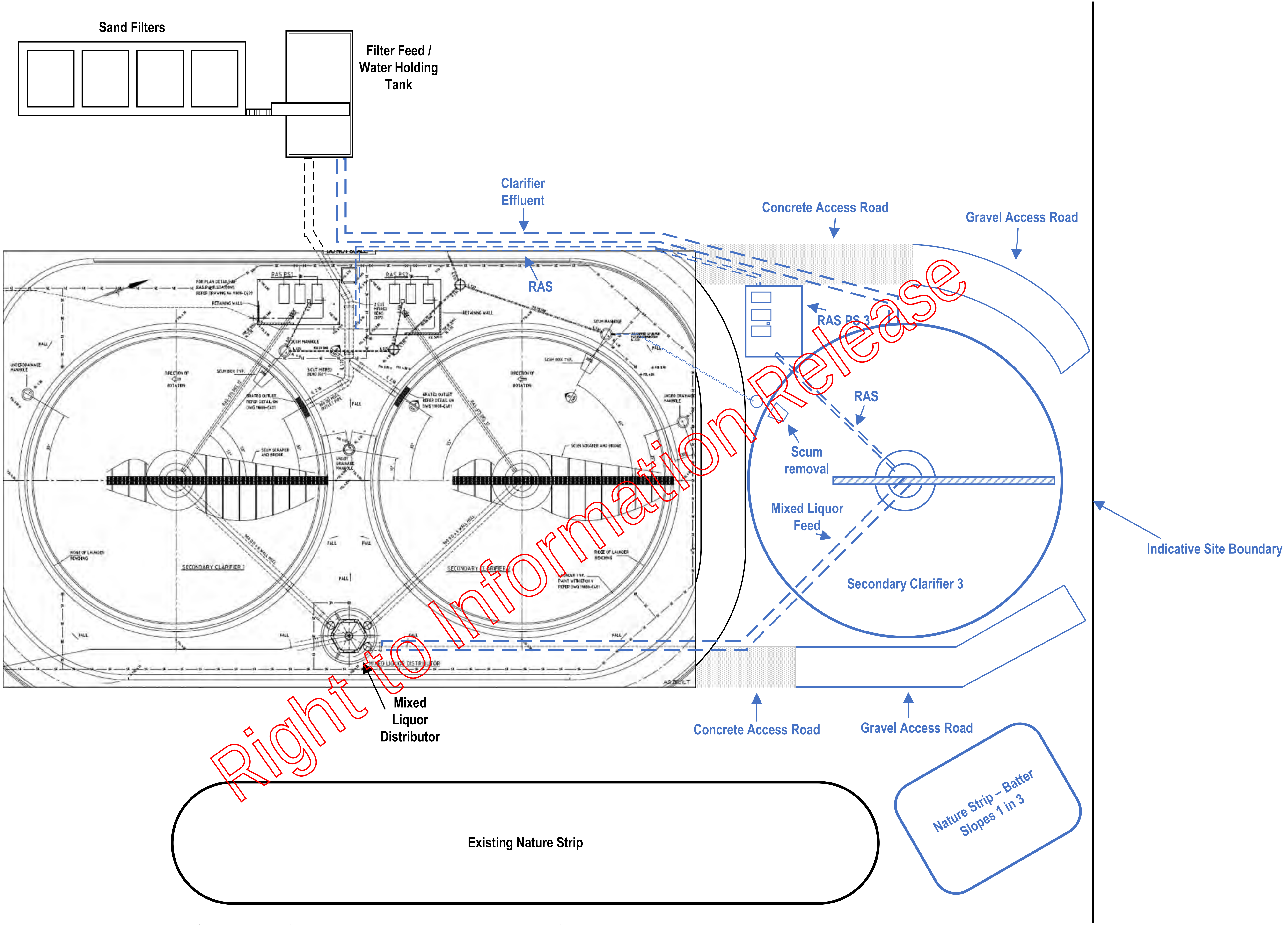
The clarifier will be provided with a log-spiral scraper, rotating bridge, and scum beaches. In keeping with the installed infrastructure, the new clarifier will be serviced by a dedicated RAS pump station comprising three pumps configured as duty/duty/standby. The clarifier will have a 1 in 12 floor slope, and a side water depth of 4.0m. The concept design has adopted a marginally deeper clarifier design due to the benefits it provides to both wet and dry weather solids capture.

Modifications to the mixed liquor flowsplitter are required to install each new clarifier, including modification of the internal division in the flowsplitter's annular section and the addition of a new isolation penstock. New RAS pumps and pipework, scum pipework (to the existing scum system), and civil works for the RAS pump station, have also been included within the assessment.

Table 6-4 outlines the schedule of works required for additional secondary clarifier.

Table 6-4: Schedule of Capital Works – Additional Clarifier

Item	Works required
Modifications to Mixed Liquor Flowsplitter	• New DN960 Mixed Liquor pipe from Mixed Liquor Distributor • Modify internal division in Mixed Liquor Distributor outer annulus • 2 No new penstocks
Additional Final Clarifier	• Nominal 34.5m diameter, 4m side wall depth clarifier • Clarifier mechanism (including bridge, scraper, flocculation skirt, energy dissipating inlet, centre column, weirs, scum beaches, scum pump)
RAS Pump Station	• New RAS pipework, fittings, and civil works for additional RAS pump station • 3 no 11 kW RAS pumps sized for 190 L/s with two pumps at 100%



No.	REVISIONS	DRAWN	APPROVED	DATE
A	For Estimation	RAS	DJF	25/06/2020

DRAWN BY
NAME RYAN SCHWARTZ
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APPROVED BY RPEQ
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DATE
22/04/2020

TITLE
Victoria Point Treatment Plant -
Additional Secondary Clarifier
Site Layout

Redland WATER

TYR GROUP
TREATMENT AND REUSE

DRAWING STATUS
For Phase 2 Planning
SCALE
1:400 at A3
DRAWING NUMBER
J1904-4-01
REVISION A
SHEET SIZE A3

6.3 DISINFECTION

6.3.1 Options Identification and Short-Listing

Under the criterion applied to sizing of chlorine contact tanks in the original plant design (60- minutes HRT at ADWF), the existing two chlorine contact tanks have sufficient volume for up to 9.60 ML/d ADWF or 43,640 EP. Such a capacity would be sufficient for the 2041 planning horizon (44,312 EP) while two tanks are in service. However, while the required faecal coliform kill will be readily achievable in the existing disinfection system through to the planning horizon, there are a number of factors which are likely to make compliance with the maximum free chlorine residual limit of 0.7 mg/L much more challenging as loads increase. The key factors include:

1. *Reduced secondary effluent ammonia concentrations* – Lower secondary effluent ammonia levels will be required to maintain compliance with the nitrogen mass load limit as the connected population exceeds increases. As noted in Section 6.1.2, the addition of a post-anoxic / re-aeration zone to meet the nitrogen removal requirements will reduce secondary effluent ammonia levels to near zero for much of the day. Lower secondary effluent ammonia will reduce the formation of chloramines (which support disinfection, but do not contribute to free chlorine residual).

   It should be noted that the historical performance of the plant has seen robust disinfection performance. Measured Free Chlorine levels, as recorded on daily grab samples, are comfortably below 0.7 mg/L (2015-2019 annual average 0.12-0.24 mg/L, annual maximum 0.67-0.69 mg/L). This excellent performance at low free chlorine residuals is considered likely to be partially due to chloramine disinfection in addition to free chlorine, but will be less feasible due to the lower effluent ammonia required as flows increase.

2. *Reduced Chlorine Contact Time* – The increase in flows will reduce chlorine contact time in the existing tanks (from 81 minutes at the current maximum ADWF to 59 minutes at the projected 2041 ADWF with the new developments). Modelling of the disinfection process indicates that this change will increase the free chlorine residual required to achieve the specified effluent Faecal Coliforms by 0.15 mg/L at ADWF, and 0.26 mg/L at peak dry weather flow, and 0.69 mg/L at PWWF.
3. *Chlorine Contact Tank Off-lining for Maintenance* – In the existing plant, the chlorine contact time is effectively halved during the routine cleaning of chlorine contact tanks. At current flows, process modelling indicates that the required free chlorine residual is approximately 0.7 mg/L at ADWF with one tank out of service (in the absence of chloramination). However, the estimated required free chlorine residual with one tank out of service increases to over 1 mg/L at the higher flows associated with the new developments. At flows in excess of ADWF, the predicted residual required is expected to be higher.

Based on the above, it is anticipated that compliance with the maximum free chlorine residual limit of 0.7 mg/L is likely to become substantially more challenging as flows increase. However, given the excellent current performance in terms of both disinfection and chlorine residual achieved in plant operations to date, there appears to substantial scope to maintain compliance until the effluent ammonia needs to be reduced (to comply with the effluent total nitrogen mass load). At this point, the existing system is expected to become inoperable as the chlorine dose required for disinfection will consistently exceed the maximum free chlorine. As a result, the nominal capacity of the existing disinfection system is effectively pegged to the existing plant's nitrogen removal capacity at 38,700 EP.

IMPORTANT: There is potential for changes to the prevailing operating practices, as may be required for other aspects of plant operation, to threaten compliance with the maximum free chlorine limit at loads less than 38,700 EP. For example, off-lining of CCTs for maintenance outside of low flow periods, increased aeration to reduce effluent total nitrogen, or changes to the practices in chlorine dosing control could all result in exceedance of the maximum free chlorine limit. To this end, it is recommended that the chlorine disinfection performance be routinely reviewed as flows increase to ensure robust and consistent compliance observed in operations to date is being maintained.

Once the capacity of the existing chlorine contact tanks is exceeded, there are two key options for augmentation:

Option 1: Installation of additional chlorine contact tank volume to reduce the free chlorine residual required to achieve disinfection, or,

Option 2: Dechlorination at the end of the chlorine contact tank through dosing of sodium bi-sulphite (SBS) or sodium meta bi-sulphite (SMBS).

Redland City Council operates a dechlorination facility at Cleveland STP, and has encountered significant difficulties in operation of the system. Issues have included:

- The on-line chlorine residual meters require high levels of ongoing maintenance to remain accurate. As the dosing of SBS is controlled under feedback from these instruments, the system is unable to operate reliably without accurate readings. RCC outsourced the maintenance of these instruments under contract due to the excessive demand they imposed on Operator resources. However, even with this maintenance outsourced, the accuracy of dosing control remains a significant issue.
- Variations in the ammonia concentration in the Cleveland STP effluent have a very strong bearing on the SBS dose required, and has resulted in very high SBS consumption over short periods. Maintaining a suitable supply of SBS on site has been at issue as a result.

While not a specific issue noted at Cleveland STP, overdosing of SBS consumes dissolved oxygen in the effluent stream, and has the potential to push the DO concentration below the minimum of 2 mg/L in the Environmental Approval. DO monitoring in the SBS mixing chamber at Cleveland indicates that this is not an issue at this site.

Based on the difficulties encountered in dechlorination at Cleveland, the provision of additional disinfection capacity has been based on Option 1.

6.3.2 Additional Chlorine Contact Tank Concept Design

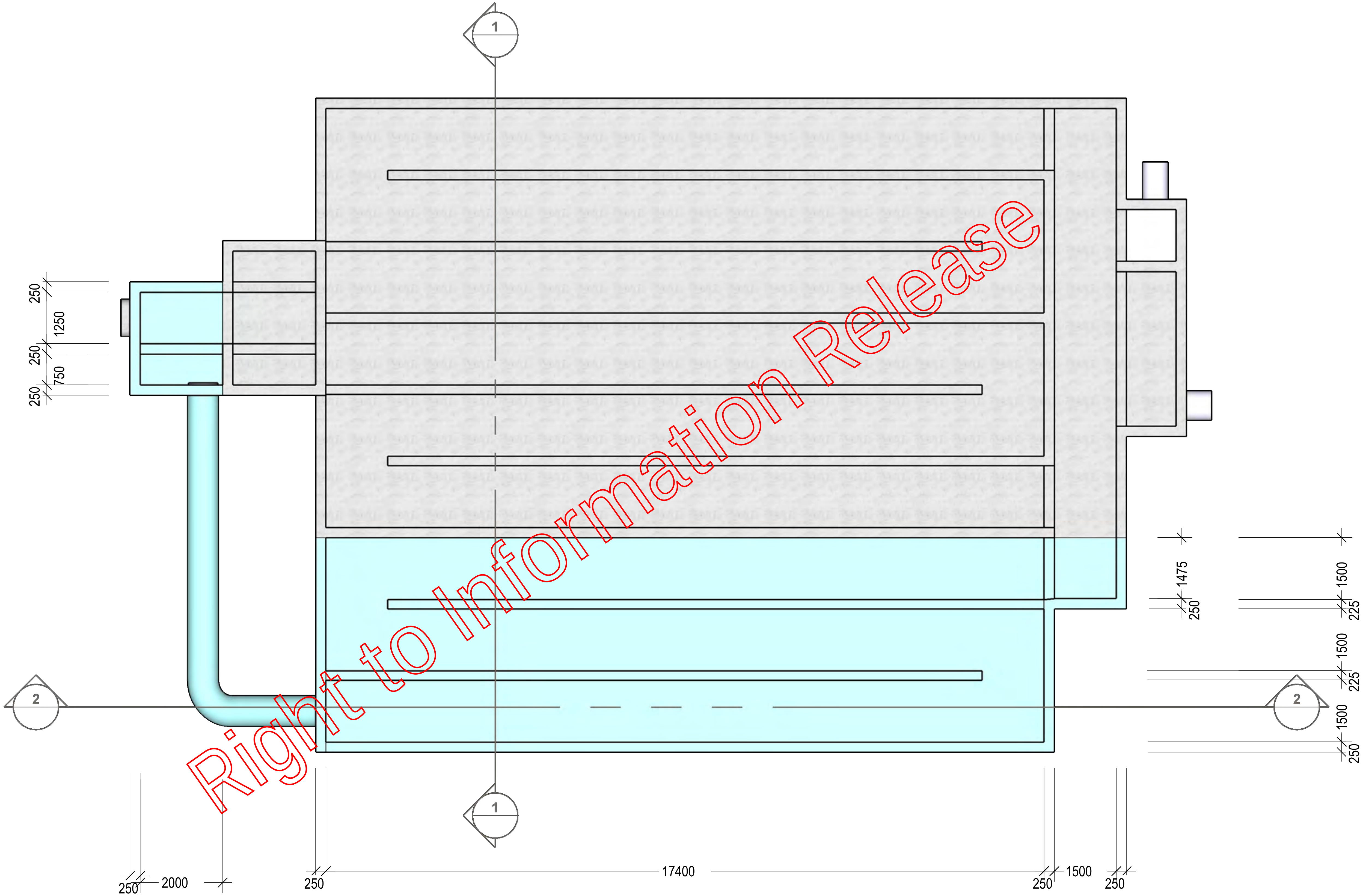
The additional chlorine contact tank will be identical in design to the two existing tanks, and located immediately to the north of the existing units. This additional chlorine contact tank will be required from 2025 under the projected loads associated with the new developments.

The concept design of the additional chlorine contact tank is summarised in Table 6-5, and shown in Figure 6-7 and Figure 6-8.

Table 6-5: Schedule of Capital Works – Additional Chlorine Contact Tank

Item	Works required
Additional Chlorine Contact Tank	• 1 No. new 3-pass Chlorine Contact Tank • Nominal volume 200 kL o Internal dimensions 17.4 length x 1.5m width per pass x 2.61m water depth to TWL o Serpentine flow between passes • Extension to existing inlet chamber, including new 1.5m weir to initial leg in 7.5m x 1.5m x 2.61m water depth concrete chamber. • Extension to outlet chamber, 1.5m long extension to existing outlet chamber.
Chlorination	• Modification to chlorinator discharges to inlet pipe to inlet chamber.

No substantial modification to the chlorine storage and dosing system is expected to be required to accommodate the loads associated with the two developments.



CLIENT	Redland City Council
CONSULTANT	Tyr Group P.O. Box 315 Bangalow, NSW, 2479

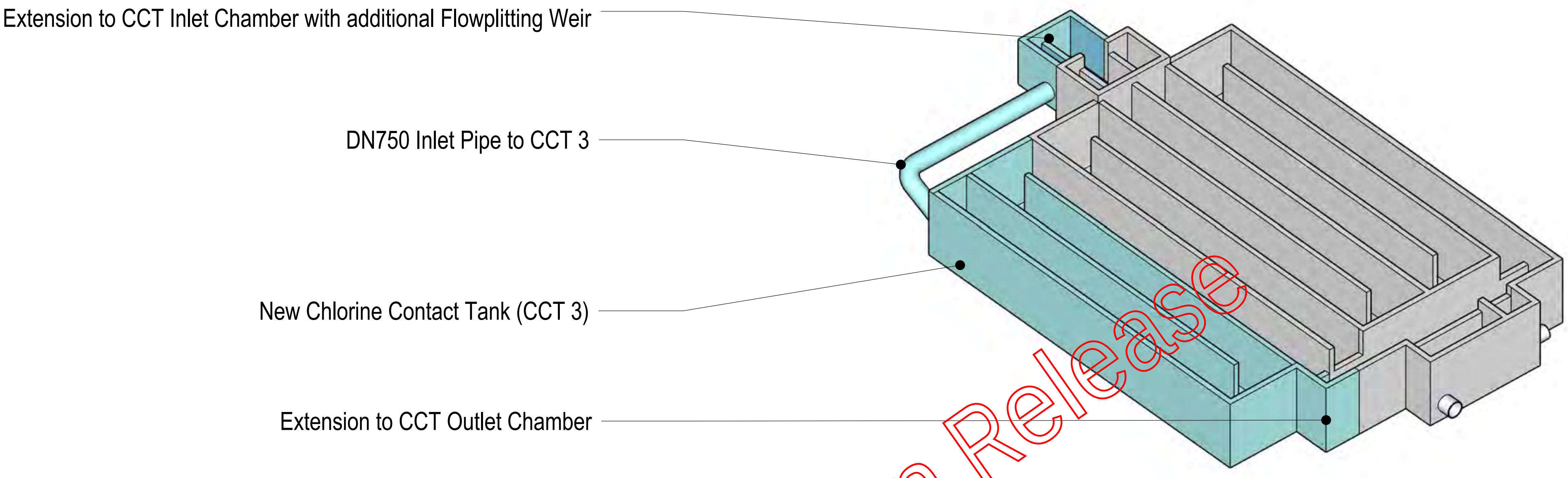
No.	REVISIONS	DRAWN	APPRVD	DATE
A	For Phase 2 Planning	DJF	DJF	5/5/2020

DRAWN	CHECKED	REVIEWED
DJF	IAF	IAF

APPROVED: DJF
DATE: 5/5/2020
SCALE: 1:100 at A3

TITLE
Victoria Point STP Upgrade Planning
Additional Chlorine Contact Tank
General Arrangement - Plan

REVISION	ORIGINAL SHEET SIZE	DRAWING NUMBER
A	A3	J1904-05



ISOMETRIC
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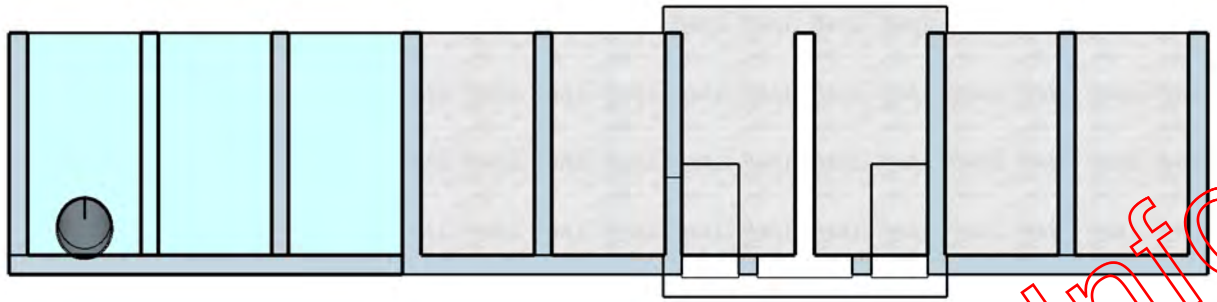

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SECTION 1
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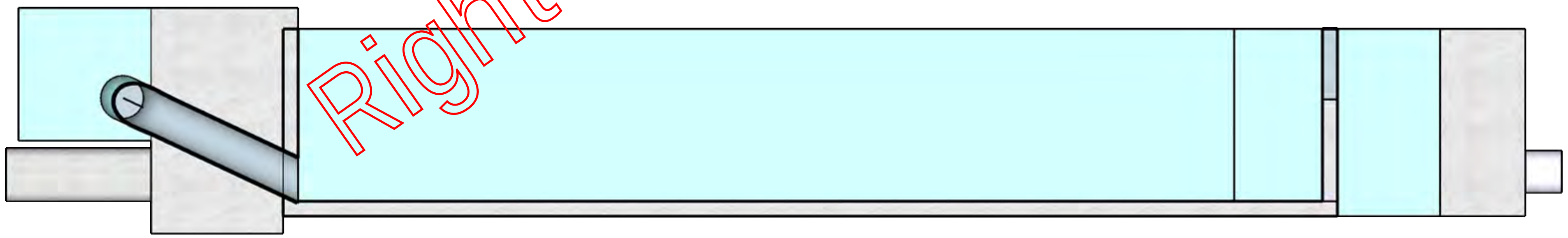

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SECTION 2
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CLIENT
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No.	REVISIONS	DRAWN	APPRVD	DATE
A	For Phase 2 Planning	DJF	DJF	5/5/2020

DRAWN DJF
CHECKED IAF
REVIEWED IAF
APPROVED DJF
DATE 5/5/2020
SCALE 1:200 at A3

TITLE
Victoria Point STP Upgrade Planning
Additional Chlorine Contact Tank
Isometric And Sections

REVISION A
ORIGINAL SHEET SIZE A3
DRAWING NUMBER J1904-06

7 ESTIMATED COSTS

7.1 CAPITAL COSTS

The capital costs for the upgrades have been estimated assuming delivery of the required upgrade works under a single contract.

Key assumptions applied to the capital cost estimates include:

- Cost estimates have not considered geotechnical information. No piling or decontamination of land has been allocated within the cost estimate. Should acid sulphate soils, contaminated land or geotechnical issues arise, costs would increase. This level of detail would be expected to be assessed during subsequent design phases through geotechnical analysis of the proposed site.
- No structural design has been undertaken. As a result, the extent of concrete works has been drawn from the existing structures on site, and typical slab and wall thickness applied in the detailed design of comparable water retaining structures.
- The costs for supply of major equipment items (and installation where appropriate) are based on budget quotations from equipment suppliers based on the concept design. This includes raw sewage screening and screenings handling equipment, blowers, and clarifier mechanical equipment.
- The costs for procurement of minor mechanical equipment items (blowers, pumps, mixers, valves, penstocks, and stopboards) have been based on actual supply costs in relevant previous sewage treatment plant upgrade projects. Similarly, the cost rates for earthworks, yard pipework, concrete cutting and other general civil construction have been drawn from advice from construction engineers on comparable sewage treatment plant projects.
- Costs have been escalated using the Non-residential construction cost index or CPI as applicable.
- The cost rates for concrete works are derived from construction of similar scaled water retaining structures in water and sewage treatment plants over the last 12 months. The rates are drawn from Tier 2 contractors.
- Delivery of the upgrades under a design-and-construct delivery model has been assumed. Other delivery modes may be selected at the discretion of RCC, and carry different overheads for the Contractor and Redland City Council, and different margins.
- A 30% contingency was applied to capital cost estimates, which is considered appropriate for the level of design completed and the bottom-up estimating methodology applied.
- Foreign exchange risk was applied to key elements sources from overseas. Contractor margins are shown in Table 7-1.

Overall, within the assumptions listed above, the cost estimates have pursued an accuracy of +/- 30%.

Table 7-1 Indirect Costs, Overhead and Margin included in the Capital Cost Estimate

Item	Value
Indirect costs (including bid costs, mobilisation, bonds, insurance, legal, administration, inclement weather, site establishment, office staff costs)	25% of DJC
Design and Engineering	11-14% of DJC
Foreign Exchange Risk	10% of Imported Equipment
Design Growth	3% of DJC
Contractor Fees and Margin	11% of Net Capital Cost
Client Costs	5% of Total Contract Cost
Contingency	30% of Total Project Cost

Table 7-2 outlines the costs associated with the plant upgrades as described in Section 6.

Table 7-2 Estimated Capital Costs for Upgrades through to 2041 Planning Horizon ($AUD, 2020)

Item	Inclusions	Direct Job Cost	
Preliminaries	Site Establishment	$32,000	$72,600
	Site Survey	$15,400	
	Service Location	$3,200	
	Geotechnical Investigations	$12,000	
	Environmental Controls	$10,000	
Post-Anoxic / Re-Aeration Tank	Civil works comprising: • Excavation (with fill to new berm) • Slab (including toe) and walls of Post-Anoxic / Reaeration Tank, Transfer and Outflow chambers • Mixed liquor pipe modification (block-outs cuts) • Concrete duct from transfer chamber to Post-Anoxic Cell 1 • Slab and apron for access to blower room • Concrete cut to existing toe of oxidation ditch • Blower building, including louvres and door • Walkway and access stairs to access post-anoxic /re-aeration cells and mixers (grid mesh) • Access road (sealed, with kerb and gutter) to blower building.	$837,800	$1,290,000
	Supply and install 3 No. post-anoxic cell mixers for (3.7 kW each)	$37,900	
	Supply and install new diffused aeration system, comprising: • Fixed-to-floor fine pore membrane diffusers in Post-Anoxic Cell 3 and Re-Aeration Zone • 2 No. Aeration Blowers (Atlas-Copco ZL2 VSD) • DN150 Spiral wound stainless aeration pipework • DO meter for Re-Aeration Zone • Actuated butterfly valve for Post-Anoxic Cell 3 aeration control	$191,000	
	Miscellaneous additional mechanical comprising: • Extension to existing service water network • Relocation of scum harvester • Stopboards (2 No.) and penstock (1 No.) for isolation and bypassing of Post-Anoxic / Re-Aeration Tank	$55,200	
	Electrical and Control at 13% of DJC for Post-Anoxic Zone	$167,600	

Table 7-2 Estimated Capital Costs for Upgrades through to 2041 Planning Horizon ($AUD, 2020) (continued)

Item	Inclusions	Direct Job Cost	
Additional Secondary Clarifier	Civil works comprising: • Clear and Grub of area • Excavation of clarifier (with fill to new berms) • Completion of new berms for visual/noise screening, including landscaping • Modification / removal of wall in ML distributor annular section • New Mixed liquor pipework (ML distributor to Clarifier) • New RAS pipework (clarifier to pump station, pump station to main) • Secondary effluent pipework (clarifier to filter feed tank) • Concrete works to clarifier (floor, walls, toe, path, launder) • Epoxy coating of clarifier launder • Groundwater drainage pipework and manhole • Connection of scum beach to existing scum system • New RAS pump station base slab • Sealed roadway (including kerb and channel) to RAS pump station and clarifier • Gravel roadway to clarifier circumference • Repairs to existing roads at pipe crossings	$1,106,700	$2,250,000
	Supply and install clarifier mechanism comprising: • Log-spiral scraper (1 1/3 radius) • Peripheral scum baffle and weirs • Scum skimmer • 1 No. scum beach • Centre column, energy dissipating inlet, flocculation skirt • Slipring • Access bridge and walkway	$715,000	
	RAS Pump Station, comprising: • Pipework and valves within RAS pump station • 3 No. RAS pumps (11 kW) • RAS flowmeter and associated isolation valves	$178,500	
	Miscellaneous additional mechanical comprising: • New aluminium slidegate to ML Distribution Chamber for clarifier isolation • Extension to service water network and hose point	$29,200	
	Electrical and Control at 10% of DJC for Secondary Clarifier /RAS PS	$225,500	

Table 7-2 Estimated Capital Costs for Upgrades through to 2041 Planning Horizon ($AUD, 2020) (continued)

Item	Inclusions	Direct Job Cost	

Additional Chlorine Contact Tank	Civil works comprising:
	• Excavation of new CCT
	• Concrete works to:
	  o New CCT inlet distribution chamber
	  o New chlorine contact tank (~200 kL)
	  o New drainage sump
	  o Extension to CCT outlet chamber to receive flow from new CCT
	• New pipework to drainage
		$260,300	$296,000

	Miscellaneous mechanical works comprising: • Inlet isolation penstock to new CCT • Weirs and isolation stopboard • New inlet pipework cut-in	$35,200	
Testing, Commissioning and Handover	3% of DJC	$121,000	
	TOTAL DIRECT JOB COST	$4,034,000	
Indirect Costs	25% of DJC	$1,008,000	
Other Costs	Design (11%), Foreign Exchange Risk (10% of pump, mixers, instruments and blowers cost), design growth (3%)	$576,000	
Contractor Fees and Margin	11% of DJC + Indirect and Other Costs	$618,000	
	TOTAL CONTRACT COST	$6,236,000	
Client Costs	5% of Total Contract Cost	$311,800	
	TOTAL PROJECT COST	$6,548,000	
	CONTINGENCY AT 30% OF TOTAL PROJECT COST	$1,964,000	
	TOTAL PROJECT COST INCLUDING CONTINGENCY **(+/- 30% Accuracy Target)**	$8.512m	

7.2 OPERATIONAL COSTS

Treatment of the sewage loads associated with the new developments will have a material impact on the operating costs of the plant. The additional operating costs specifically required for treatment of the load associated with the new developments have been estimated based on the following input assumptions:

- Power and haulage cost rates have been based on rates provided by Redlands City Council. These are shown in Table 7-3.

Table 7-3: Adopted Values – Operational Cost Estimates

Parameter	Value	Source
Electricity cost	$0.11 /kWh $156 p.a. for each additional kW of peak demand	Redlands City Council, 2020
Polyelectrolyte	$4.95/kg	Redlands City Council, 2019
Biosolids haulage cost	$65 /Wet Tonne (lower bound) $100 /Wet Tonne (upper bound)	Redlands City Council, 2020
Chlorine	$2.94/kg	Redlands City Council, 2019

- Substrate dosing has not been included within the cost analysis as the modelling suggests that it will not be routinely required.
- Excess biological phosphorus removal performance is not expected to be significantly impacted by the increased loads associated with the new developments (or the upgrades). Further, given the limited requirement for phosphorus removal (TP 4 as long term median), alum dosing is expected to be negligible for both options.
- The cost analysis has considered unit operating costs on a comparative basis. Existing plant elements which are not subject to change (or of minimal impact) across the options have not been included in the assessment (for example existing pump stations). Elements included in the operating cost analyses include:
  - Electrical Fixed: Drives for additional operating items within the upgraded plant - principally mixers and one clarifier scraper.
  - Electrical Variable: Drives for treatment of additional load - principally aeration, RAS pumps, and filter feed pumps. Assumes 2 months per year with peak wet weather events.
  - Maintenance – for additional process units installed under the upgrades. Key items such as diffusers, clarifier mechanism, pumps.
  - Biosolids Haulage – Total additional haulage, assuming 18% dryness from screw presses being installed under the dewatering upgrade in progress.
  - Polyelectrolyte – 11 kg/dry tonne poly consumption as per typical requirement for screw presses being installed under the dewatering upgrade in progress.
  - Chlorine – Additional secondary effluent flow off-set by reduced average dose due to additional CCT.

Table 7-4 summarises the compiled operating costs associated with the upgrades and additional loads associated with the Weinam Creek and South West Victoria Point developments, as estimated for the planning horizon (2041).

Table 7-4: Estimated Annual Additional Operating Costs at 2041 ($AUD, 2020)

Upgrade	Electrical - Fixed	Electrical – Variable	Chemical - Variable	Maintenance - Fixed	Total Operating Costs
Post-Anoxic / Re-Aeration Zone	$9942 p.a.	$9999 p.a.	Nil	$6440 p.a.	$26,384 p.a.
Key elements	Mixers	Blowers		Diffuser replacement / mechanical	
Additional Secondary Clarifier	$2558 p.a.	$1212 p.a.	Nil	$18,625 p.a.	$22,395 p.a.
Key elements	Bridge, Scum pump	RAS pumps		Mechanical	
Additional Chlorine Contact Tank	Nil	Nil	$8133 p.a.	Nil	$8133 p.a.
Key elements			Chlorine		
Other Additional OPEX					
Power Consumption – Oxidation Ditch Aeration	Nil	$20,412 p.a.			$20,412 p.a.
Power Consumption – Additional Pumping (Filter Feed, miscellaneous)		$3704 p.a.			$3704 p.a.
Polyelectrolyte Consumption			$7083 p.a.		
SUB-TOTALS	$12,500 p.a.	$35,300 p.a.	$15,200 p.a.	$25,100 p.a.	$88,110 p.a.
Biosolids Haulage	$47,000 p.a. additional sludge haulage at $65 /wet tonne $72,300 p.a. additional sludge haulage at $100 /wet tonne				
TOTAL ADDITIONAL OPEX	$135,100 p.a. with additional sludge haulage at $65 /wet tonne $160,400 p.a. with additional sludge haulage at $100 /wet tonne				

Note 1: Variable and total additional operating costs shown for operations at the 2041 design load

7.3 WHOLE OF LIFE COSTS

The following assumptions have been applied to the estimation of the whole-of-life costs associated with treatment of the loads associated with the new developments:

- The analysis of options has been based on net present cost (or NPC) over a period of 40 years using the factors supplied by Redlands City Council.
- It has been assumed that construction will commence in the 2022-23 financial year, and take approximately 2 years to complete. The analysis has assumed that 50% of the capital cost of the works is spent in each year of construction.
- The variable operational costs associated with the additional load are applied to the analysis based on the projected additional population from 2020-21. The additional fixed operating costs are only applied from completion of the works in 2023-2024.

Cost escalation factors as supplied by Redlands City Council were used to account for increases to electricity, labour, maintenance and other costs, and costs of capital as summarised in Table 7-5.

Table 7-5: Discount Rate and Escalation Factors applied to Whole-of-Life Cost Analysis

Parameter	Factor
Discount Rate (Weighted Average Cost of Capital)	7.00 % p.a.
Capital Escalation	1.07% FY2021 1.43% FY21-22 1.79% FY22-23 2.16% FY23-24
Electricity Escalation	2.50 % p.a.
Maintenance and Other Items Escalation (including biosolids haulage)	2.50 % p.a.
Chemicals and other Operating Costs Escalation	2.50 % p.a.

- The variable operational costs (e.g. chemical consumption, electrical power consumption and biosolids haulage) have been escalated through the NPC analysis in line with the applicable population projections.

The additional whole-of-life cost for the additional development are summarised in Table 7-6 below. Note 15-year NPC values have been given in addition to the prescribed 40-year NPCs, for information.

Table 7-6 Additional Whole of Life Costs to Service New Developments ($AUD, 2020)

Options	Total Whole of Life Cost (7% discount rate)	
Duration	15 years	40 years
Additional Costs with Biosolids Management at $65 / wet tonne (AUD, 2020)	$9.24m	$10.31m
Additional Costs with Biosolids Management at $100 / wet tonne (AUD, 2020)	$9.42m	$10.68m

The estimated costs to treat the additional load from the South West Victoria Point and Weinam Creek Developments is $10.31-10.68m over 40 years, depending on the cost of biosolids management applied.

As the whole-of-life cost includes $8.512m in capital (AUD 2020), the capital cost comprises the majority of the servicing costs. The low contribution of operational costs is the result in the delay to the completion of the upgrade (2023-2024), and the low contributing population from the new developments in the initial years.

8 CONCLUSION AND RECOMMENDATIONS

The prevailing capacity of Victoria Point STP is limited to 38,300 EP by the ability of the secondary clarifiers to treat 5 x ADWF. The existing plant's ability to maintain compliance with the Total Nitrogen Mass Load Limit will be compromised at a similar load (38,700 EP).

Upgrades to three process areas will be required to treat the projected load of 7215 EP from the South West Victoria Point and Weinam Creek developments.

Concept designs were developed for each of the upgrade works proposed, and the associated capital costs estimated.

The scope, required timing and estimated capital costs of the required upgrades is summarised in Table 8-1.

Table 8-1: Summary of Required Plant Upgrades and Staging

Upgrade	Estimated Capital Cost	Required from	
Post-Anoxic / Re-Aeration Zone)	$1.289m Direct Job Cost	38,700 EP	2025
1 No. Additional Secondary Clarifier	$2.255m Direct Job Cost	38,300 EP	2024
1 No. Additional Chlorine Contact Tank	$0.296m Direct Job Cost	38,700 EP	2025
TOTAL CAPITAL COST (+/- 30% Accuracy Target)	Total Direct Job Cost (including Preliminaries, Commissioning and Handover): $4.033m Total Project Cost (including 30% Contingency): $8.512m		

The additional operational costs required to treat the sewage load generated by the South West Victoria Point and Weinam Creek Developments were estimated in detail. The additional electricity consumption and biosolids haulage required to treat the load dominates the additional costs. In 2041 (the planning horizon), the additional annual operating cost is $135,100 p.a. with additional sludge haulage at $65 /wet tonne, increasing to $160,400 p.a. if the rate for sludge haulage rises to $100 /wet tonne.

The whole-of-life cost to treat the additional load from the South West Victoria Point and Weinam Creek Developments is $10.31-10.68m over 40 years depending on the cost of biosolids management.

The works to treat sewage loads from the new developments are required to be completed and in service by 2024-25. This suggests the upgrades should be undertaken under a single contract with procurement and design commencing in 2020-21.

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APPENDIX A: VICTORIA POINT WWTP – HYDRAULIC ANALYSIS

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19/06/2019

Our ref: TYR-190531

Tyr Group
PO Box 315
Bangalow NSW 2479

Attn: David Fligelman

Dear David,

Victoria Point WWTP - Hydraulic Analysis

1 Introduction

Tyr Group have commissioned CMP Consulting Group a hydraulic analysis of the Victoria Point WWTP. The nominated cases assessed were

- 500 L/s influent + 345 L/s RAS
- 404 L/s influent + 279 L/s RAS
- 577 L/s influent and 400 L/s RAS

We have also looked at the flows that match the hydraulic profile provided.

There are some areas where we are missing information. This is either because of unclear or missing pump data or information that we are unable to determine from the drawings.

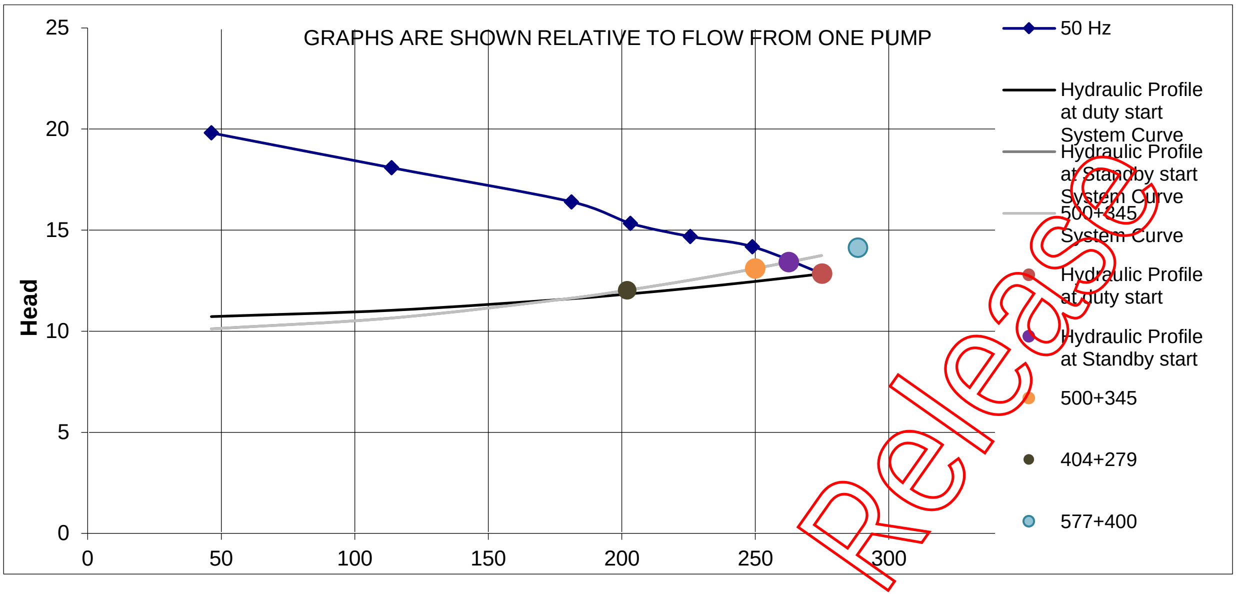
We have not looked at any of the chemical dosing.

The following is a summary of our findings.

2 Results

2.1 Inlet Pump Station

Depending upon operating level in the well and the level in the inlet works (modelled at the nominated figures of 8.36m) as well as which pumps are running, pump 1 should produce approximately 275 L/s of flow (red dot on the following graph). This matches the SCADA data provided. Both pumps running should produce around 525 L/s. This is right on the end of the pump curve and will operate with cavitation assuming that the full pump curve has been shown in the data provided. We have not been able to find other published data for this pump.



The nominated cases where the inlet flow is 500 L/s (250 L/s per pump – dark grey dot) and 404 L/s (202 L/s per pump - yellow dot) are achievable. The one where the inlet flow is 577 L/s (288.5 L/s per pump – blue dot) is not achievable without replacing the pumps.

2.2 Inlet Channel

Hydraulic losses along the channel are only 2mm + whatever losses occur as a result of the grit screw, the step screen and the vortex grit trap. There is no flow vs pressure loss information in VoR's documentation for these.

2.3 Pipe from Inlet Channel to Anaerobic Reactor

Losses are 54 mm at 500 L/s, 35 mm at 404 L/s and 71 mm at 577 L/s. The hydraulic profile shows a drop of 110 mm. This would match a flow of around 727 L/s.

2.4 Anaerobic Reactor and Oxidation Ditch

The flooded weir entering the Anaerobic Reactor can take larger flows than any of the nominated cases without exceeding the hydraulic profile levels.

2.5 Weir Outlet form Oxidation Ditch

The tilting weir on the outlet of the oxidation ditch provides enough freeboard (at least 300mm) in the oxidation ditch for all three nominated flows.

2.6 Pipe from Oxidation Ditch to Mixed Liquor Distributor

Losses are in the order of 27mm at a flow of 180 L/s, 108mm at a flow of 360 L/s and to match the hydraulic profile, the flow through this pipe is in the order of 1460 L/s.

2.7 Across Weir in Mixed Liquor Distributor

We have assessed the flow going to each clarifier on the basis of matching the hydraulic profile and also how much could be achieved if you only allowed for 300mm freeboard in the central chamber.

Matching the hydraulic profile, the flow is 340 L/s for each clarifier or 680 L/s total. The maximum flow allowing for minimum freeboard is over 1400 L/s combined.

2.8 Pipe from Mixed Liquor Distributor to Clarifier

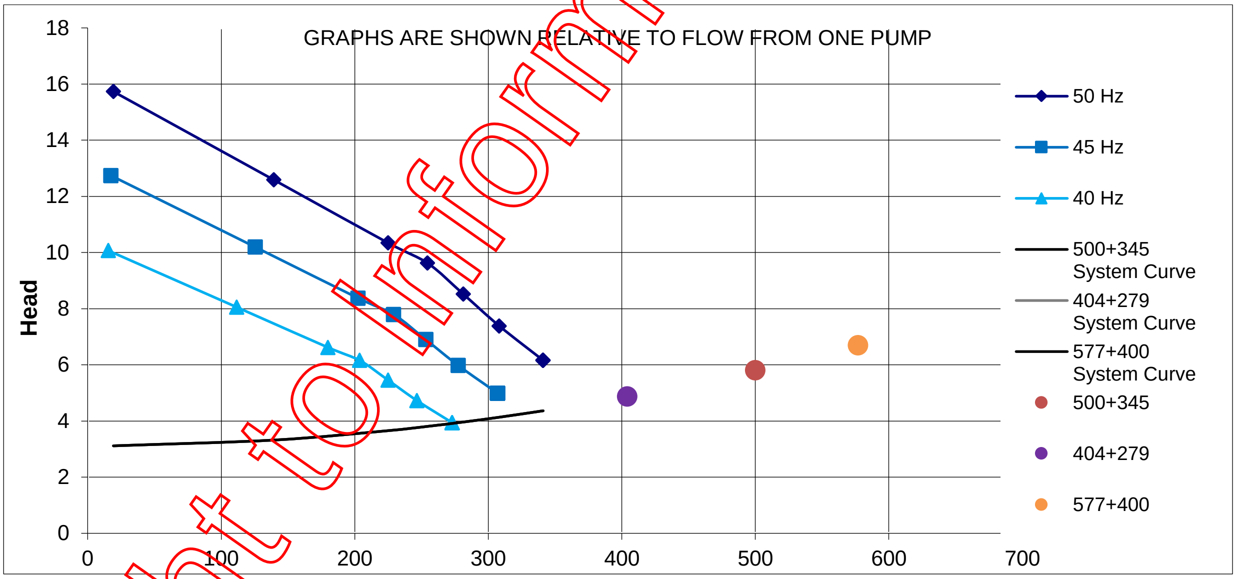
Losses are 229 mm for 500+345 L/s, 157 mm for 404+279 L/s, 306 for 577+400 L/s. To match the hydraulic profile, the flow through the pipe is in the order of 517 L/s. This is per clarifier. Flow capacity is above the nominal figures.

2.9 Pipe from Clarifier to Filter Feed Tank

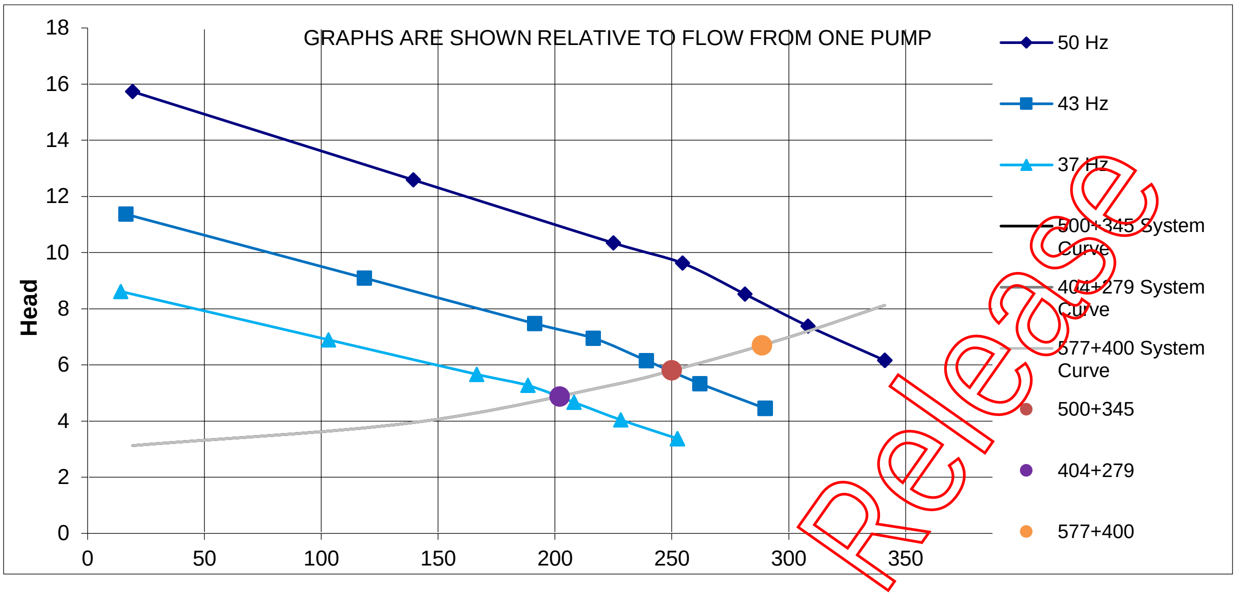
Losses are 11mm for 180 L/s, 43 mm for 360 L/s and to match the hydraulic profile, the flow through the pipe is in the order of 754 L/s. This is a combined flow. The flow out of each of the clarifiers will be half of these.

2.10 Filter Feed Tank to Filters

This is a pumped system and while the calculation has been set up, the information on the pumps doesn’t make much sense for single pump duty. The figures show the pumps running way off the end of the curve. With one pump running, this should not work at any flow rate.



If two pumps are put into service, the increased back pressure puts the system curve into a position where the pumps are operable at all of the nominated flow rates. For changes to the existing system, the actual flow rate required by these pumps will need to be checked once the PFD has been fully developed and there would be no standby.



2.11 Filters

Hydraulic gradient through clean media is $h = \frac{6\,(1-e)V^2}{d\,e^3 g}$ x (5 Re^-1 + 0.4 Re^-0.1)

- e = media voidage
- d = hydraulic size of media
- V = Filtration rate
- Re = Reynolds number in media

In practical analysis, this cannot be worked out without a lot more information. The most effective way to address the hydraulic capacity of the filters is to look at the headlosses against outlet control valves and then extrapolate from there. If you are able to provide operational information on the range of valve positions against dp, we could potentially do an estimate of the maximum possible flow rate.

A possible approximation would be to base the flow rate on 10 m/hr through the filters. This gives a flow of 442 L/s which is less than two of the three nominated conditions.

2.12 Filtered Water Holding Tank to Chlorine Contact Tank Inlet

Losses are 88 mm for 500 L/s, 58 mm for 404 L/s and 117 mm for 577 L/s. To match the hydraulic profile, the flow through the pipe is in the order of 1012 L/s.

2.13 Chlorine Contact Tank Outlet Weirs

To match the figures on the hydraulic Profile, the flow over the weir to the old secondary clarifiers is in the order of 1610 L/s. The flow over the weir to the outfall is 4835 L/s.

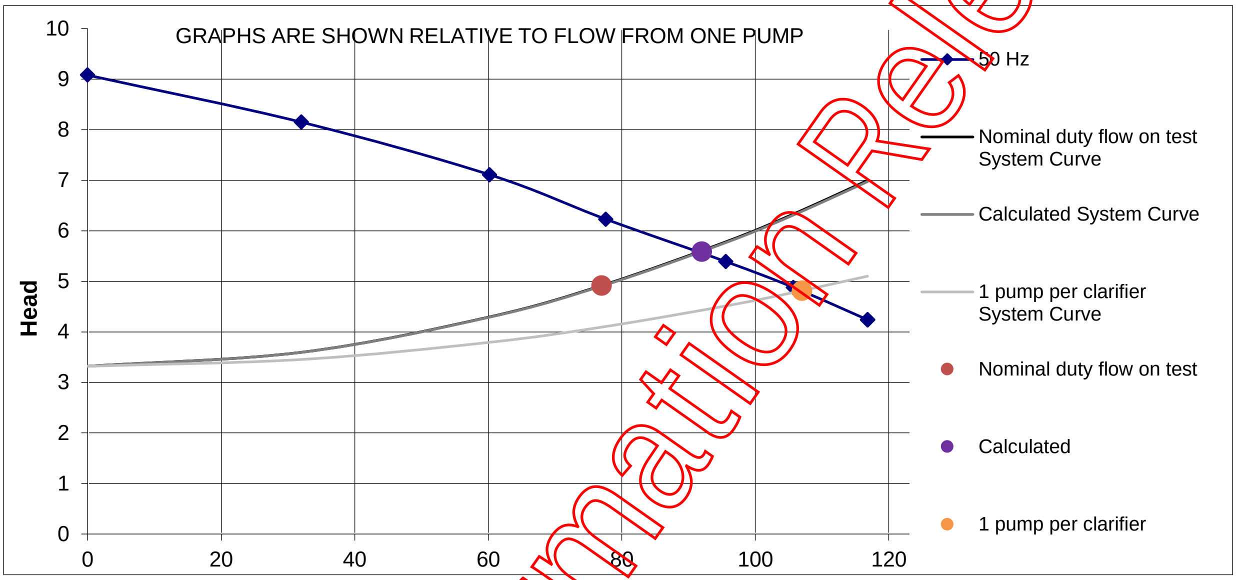
There is hydraulic data for a final manhole, but the location of this manhole is not shown on the drawings, so we are unable to model this.

2.14 Waste Activated Sludge Pumps

These pumps are progressive cavity rated at 8.3 L/s with a very steep curve. The actual flow rate will depend upon the pump condition, particularly of the stator. If the pump is in good condition, then the flow rate of 1 L/s should be a reasonable assumption.

2.15 Return Activated Sludge Pumps

The nominated duty point per pumps on the test data is 77 L/s. The nominated duty in the Summary of unit sizing is 94 L/s. Assuming consistency of water, the plant should be able achieve over 90 L/s per pump. Thicker sludge will drop that value.



The nominated RAS flows of 345 L/s (86.25 L/s per pump) and 279 L/s (69.75 L/s per pump) are achievable. The flow of 400 L/s is not achievable without replacing the pumps.

2.16 Foul Water Return Pumps

We need clarification on pump performance data. Foul water pumps and belt press filtrate pumps have been filed together without labelling.

2.17 Conclusion

The limitations on the system are

- Inlet Pumps – The existing pumps are not capable of achieving the 577 L/s between them.
- Filter Feed Pumps – The performance data from the existing pumps provided does not match the analysis for single pump duty. The curves for these pumps need to be confirmed.
- Filters – The existing filters are likely to be insufficient. More filter area is required.
- RAS Pumps – The highest of the three RAS flows assessed is not achievable.

Yours faithfully

Lachlan Douglas

Calculation

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Client:

Tyr Group

Description:

Plant Hydraulic Analysis

Project:

Victoria Pt WWTP Analysis

Doc No:

TYR-190531 CAL 01

Rev:

3

Revision	Issued for:	Prepared	Date	Checked	Date	Approved	Date
1	Preliminary Issue	LD	17/06/2019				
2	Issued for Review	LD	19/06/2019	Still to be checked			
3	Final	LD	19/06/2019	GC	19/06/2019		

INLET PUMP STATION

1. Design Input

			Case 1	Case 2	Case 3	Case 4	Case 5	
Different cases for different flows and/or elevations but same piping system			Hydraulic Profile at duty start	Hydraulic Profile at Standby start	500+345	404+279	577+400	
Pump Type	Submersible							
No of duty pumps	PN =		1	2	2	2	2	

Graphs on the System Curve worksheet will be displayed in the units selected below.

Total flow	Q =	Choose units from drop down	275	525	500	404	577	L/s
	Qt =		990.000	1890.000	1800.000	1454.400	2077.200	m^3/hr
	qt =	Qt / 3.6	275.000	525.000	500.000	404.000	577.000	L/s
			0.275	0.525	0.500	0.404	0.577	m^3/s
			23.760	45.360	43.200	34.906	49.853	ML/d
Flow per pump			275	262.5	250	202	288.5	L/s
	Qp =	Qt / PN	990.000	945.000	900.000	727.200	1038.600	m^3/hr
	qp =	Qp / 3.6	275.000	262.500	250.000	202.000	288.500	L/s
Pumped liquid:	water							
Density of pumped liquid	Dens =		1000	1000	1000	1000	1000	kg/m^3
Density of water	$Dens_{H2O}$ =		1000	1000	1000	1000	1000	kg/m^3
Kinematic Viscosity of liquid	KV =	25 C	8.910E-07	8.910E-07	8.910E-07	8.910E-07	8.910E-07	m^2/s
	KVcst =	KV x 1E6	0.891	0.891	0.891	0.891	0.891	cSt

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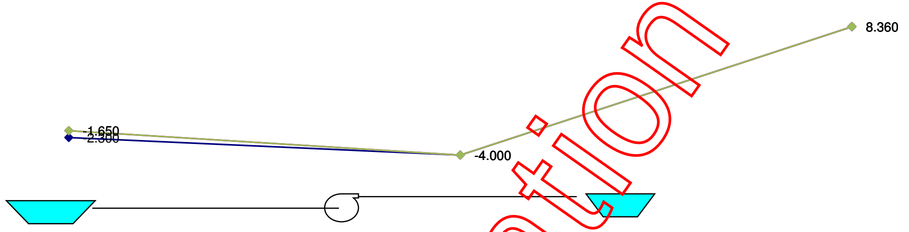
2. Static Conditions



2.1 Pump

			Hydraulic Profile at duty start	Hydraulic Profile at Standby start	500+345	404+279	577+400	
Elevation of pump	ELp =		-4.000	-4.000	-4.000	-4.000	-4.000	m EL

2.2 Suction

Elevation liquid level	ELsl =		-2.300	-1.650	-1.650	-1.650	-1.650	m EL
Liquid pressure at pump	SPl =	ELsl - ELp	1.700	2.350	2.350	2.350	2.350	m liq
Air or gas pressure	SPg =	e.g. pumping from pressurised system						kPag
Equivalent liquid head due to air pressure	SPm =	SPg / Dens / g x 1E3	0.000	0.000	0.000	0.000	0.000	m liq
Static suction head	SHs =	SPl + SPm	1.700	2.350	2.350	2.350	2.350	m liq

2.3 Discharge

Elevation liquid level	ELdl =		8.360	8.360	8.360	8.360	8.360	m EL
Liquid pressure at pump	DPl =	ELdl - Elp	12.360	12.360	12.360	12.360	12.360	m liq
Air or gas pressure	DPg =	e.g. pumping to pressurised system						kPag
Equivalent liquid head due to air pressure	DPm =	DPg / Dens / g x 1E5	0.000	0.000	0.000	0.000	0.000	m liq
Static discharge head	DHs =	DPl + DPm	12.360	12.360	12.360	12.360	12.360	m liq

2.4 Static Head

Static differential head	Hs =	DHs - SHs	10.660	10.010	10.010	10.010	10.010	m liq

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3. Dynamic Conditions

3.1 Suction

Pipe Section 1	**Not used**	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400
Pipe Section 2	**Not used**	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400
Pipe Section 3	**Not Used**	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400
Pipe Section 4	**Not Used**	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400

3.2 Discharge

Pipe Section 5	**Pump Discharge**		Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	
Pipe size	DICL?		DN375	DN375	DN375	DN375	DN375	mm
Inside Diameter	d_5 =	Use accurate internal diameter from tables	406	406	406	406	406	mm
	D_5 =	d_5 / 1000	0.406	0.406	0.406	0.406	0.406	m
Area	A_5 =	Π / 4 x D_5^2	0.129	0.129	0.129	0.129	0.129	m^2 area

Number of streams for total flow	S_5 =	Default from Design Inputs	1	2	2	2	2	
Flow for this pump station		Default from previous section	990	945	900	727.2	1038.6	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc						m3/hr
Total flow for this pipe section	Q_5 =		990.000	945.000	900.000	727.200	1038.600	m³/h
	q_5 =	Q_5 / 3.6	275.000	262.500	250.000	202.000	288.500	L/s
Velocity	V_5 =	Q_5	2.124	2.028	1.931	1.560	2.228	m/sec
		A_5 x 3600						
Pipe Wall Roughness	k_5 =		3	3	3	3	3	mm
			0.003	0.003	0.003	0.003	0.003	m
Reynolds number	Re_5 =	V_5 x D_5	967919	923923	879926	710981	1015435	
		KV						
Reynolds number is above 2500, therefore flow may be considered turbulent								
Friction factor	f_5 =	0.25	0.034	0.034	0.034	0.034	0.034	
(Swamee & Jain modified CW equ.)		(log (k5 / 3.7 / D5 + 5.74 / Re5^0.9))²						
Hydraulic gradient	HG_5 =	f_5 x 100 x V_5^2	1.950	1.777	1.612	1.054	2.146	m/100 m
		D_5 x 2 x g						
Quantity	k value							
15 m of Pipe length		x HG_5 / 100	0.293	0.267	0.242	0.158	0.322	m liq
2 x Elbow Short Radius 90	1	per fitting x V_5^2 / 2 / g	0.460	0.419	0.380	0.248	0.506	m liq
1 x Valve - Check conventional	2.4	per fitting x V_5^2 / 2 / g	0.552	0.503	0.456	0.298	0.607	m liq
1 x Valve - Gate	0.2	per fitting x V_5^2 / 2 / g	0.046	0.042	0.038	0.025	0.051	m liq
1 x Expander 4:5	0.15	per fitting x V_5^2 / 2 / g	0.034	0.031	0.029	0.019	0.038	m liq
Sub total	dP_5 =	Sum of friction losses	1.385	1.262	1.145	0.747	1.524	m liq

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Pipe Section 6	**Pump station header**		Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	
Pipe size	DICL		DN500	DN500	DN500	DN500	DN500	mm
Inside Diameter	d_6 =	Use accurate internal diameter from tables	472	472	472	472	472	mm
	D_6 =	d_6 / 1000	0.472	0.472	0.472	0.472	0.472	m
Area	A_6 =	Π / 4 x D_6^2	0.175	0.175	0.175	0.175	0.175	m² area
Number of streams for total flow	S_6 =	Default from Design Inputs	1	2	2	2	2	
Flow for this pump station		Default from previous section	990.000	945.000	900.000	727.200	1038.600	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc						m3/hr
Total flow for this pipe section	Q_6 =		990.000	945.000	900.000	727.200	1038.600	m³/h
	q_6 =	Q_6 / 3.6	275.000	262.500	250.000	202.000	288.500	L/s
Velocity	V_6 =	Q_6	1.572	1.500	1.429	1.154	1.649	m/sec
		A_6 x 3600						
Pipe Wall Roughness	k_6 =	See attached worksheet	3	3	3	3	3	mm
			0.003	0.003	0.003	0.003	0.003	m
Reynolds number	Re_6 =	V_6 x D_6	832575	794730	756886	611564	873446	
		KV						
Reynolds number is above 2500, therefore flow may be considered turbulent								
Friction factor	f_6 =	0.25	0.033	0.033	0.033	0.033	0.033	
(Swamee & Jain modified CW equ.)		(log (k6 / 3.7 / D6 + 5.74 / Re6^0.9))²						
Hydraulic gradient	HG_6 =	f_6 x 100 x V_6^2	0.876	0.799	0.725	0.474	0.964	m/100 m
		D_6 x 2 x g						
Quantity	k value							
6 m of Pipe length		x HG_6 / 100	0.053	0.048	0.043	0.028	0.058	m liq
1 x Tee - in line	0.6	per fitting x V_6^2 / 2 / g	0.076	0.069	0.062	0.041	0.083	m liq
1 x Elbow Short Radius 45	0.4	per fitting x V_6^2 / 2 / g	0.050	0.046	0.042	0.027	0.055	m liq
1 x Reducer 5:4	0.15	per fitting x V_6^2 / 2 / g	0.019	0.017	0.016	0.010	0.021	m liq
Sub total	dP_6 =	Sum of friction losses	0.197	0.180	0.163	0.107	0.217	m liq

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Pipe Section 7	**Flowmeter**		Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	
Pipe size			DN400	DN400	DN400	DN400	DN400	mm
Inside Diameter	d_7 =	Use accurate internal diameter from tables	372	372	372	372	372	mm
	D_7 =	d_7 / 1000	0.372	0.372	0.372	0.372	0.372	m
Area	A_7 =	Π / 4 x D_7^2	0.109	0.109	0.109	0.109	0.109	m² area
Number of streams for total flow	S_7 =	Default from Design Inputs	1	1	1	1	1	
Flow for this pump station		Default from previous section	990.000	1890.000	1800.000	1454.400	2077.200	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc						m3/hr
Total flow for this pipe section	Q_7 =		990.000	1890.000	1800.000	1454.400	2077.200	m³/h
	q_7 =	Q_7 / 3.6	275.000	525.000	500.000	404.000	577.000	L/s
Velocity	V_7 =	Q_7	2.530	4.830	4.600	3.717	5.309	m/sec
		A_7 x 3600						
Pipe Wall Roughness	k_7 =	See attached worksheet	3	3	3	3	3	mm
			0.003	0.003	0.003	0.003	0.003	m
Reynolds number	Re_7 =	V_7 x D_7	1056385	2016735	1920700	1551925	2216488	

		KV						
Reynolds number is above 2500, therefore flow may be considered turbulent								
Friction factor	f_7 =	0.25	0.035	0.035	0.035	0.035	0.035	
(Swamee & Jain modified CW equ.)		(log (k7 / 3.7 / D7 + 5.74 / Re7^0.9))²						
Hydraulic gradient	HG_7 =	f_7 x 100 x V_7^2	3.105	11.302	10.252	6.696	13.650	m/100 m
		D_7 x 2 x g						
Quantity	k value							
4.5 m of Pipe length		x HG_7 / 100	0.140	0.509	0.461	0.301	0.614	m liq
1 x Expander 4:5	0.15	per fitting x V_7^2 / 2 / g	0.049	0.178	0.162	0.106	0.215	m liq
1 x Bend Long Radius 90	0.4	per fitting x V_7^2 / 2 / g	0.131	0.476	0.431	0.282	0.575	m liq
Sub total	dP_7 =	Sum of friction losses	0.319	1.163	1.055	0.689	1.404	m liq

Pipe Section 8			Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	
Pipe size			DN500	DN500	DN500	DN500	DN500	
Inside Diameter	d_8 =	Use accurate internal diameter from tables	538	538	538	538	538	mm
	D_8 =	d_8 / 1000	0.538	0.538	0.538	0.538	0.538	m
Area	A_8 =	Π / 4 x D_8^2	0.227	0.227	0.227	0.227	0.227	m² area
Number of streams for total flow	S_8 =	Default from Design Inputs	1	1	1	1	1	
Flow for this pump station		Default from previous section	990.000	1890.000	1800.000	1454.400	2077.200	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc						m3/hr
Total flow for this pipe section	Q_8 =		990.000	1890.000	1800.000	1454.400	2077.200	m³/h
	q_8 =	Q_8 / 3.6	275.000	525.000	500.000	404.000	577.000	L/s
Velocity	V_8 =	Q_8	1.210	2.309	2.199	1.777	2.538	m/sec
		A_8 x 3600						
Pipe Wall Roughness	k_8 =	See attached worksheet	3	3	3	3	3	mm
			0.003	0.003	0.003	0.003	0.003	m
Reynolds number	Re_8 =	V_8 x D_8	730437	1394471	1328067	1073079	1532590	
		KV						
Reynolds number is above 2500, therefore flow may be considered turbulent								
Friction factor	f_8 =	0.25	0.032	0.032	0.032	0.032	0.031	
(Swamee & Jain modified CW equ.)		(log (k8 / 3.7 / D8 + 5.74 / Re8^0.9))²						
Hydraulic gradient	HG_8 =	f_8 x 100 x V_8^2	0.438	1.592	1.444	0.943	1.922	m/100 m
		D_8 x 2 x g						
Quantity	k value							
6 m of Pipe length		x HG_8 / 100	0.026	0.096	0.087	0.057	0.115	m liq
1 x Elbow Short Radius 90	1	per fitting x V_8^2 / 2 / g	0.075	0.272	0.247	0.161	0.328	m liq
1 x Enlargement Sudden	1	per fitting x V_8^2 / 2 / g	0.075	0.272	0.247	0.161	0.328	m liq
Sub total	dP_8 =	Sum of friction losses	0.175	0.639	0.580	0.379	0.772	m liq

Pipe Section 9	Not Used	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	
Pipe Section 10	Not Used	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	
Control Valve Sizing	Not Used	Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400	

4. Total Dynamic Losses

				Hydraulic Profile at duty start	Hydraulic Profile at Standby start	500+345	404+279	577+400	
Friction loss in suction pipework									
Pipe Section 1	Not used	dP_1 =		0.000	0.000	0.000	0.000	0.000	m liq
Pipe Section 2	Not used	dP_2 =		0.000	0.000	0.000	0.000	0.000	m liq
Pipe Section 3	Not Used	dP_3 =		0.000	0.000	0.000	0.000	0.000	m liq
Pipe Section 4	Not Used	dP_4 =		0.000	0.000	0.000	0.000	0.000	m liq
Total		SHd =	$dP_1 + dP_2 + dP_3 + dP_4$	0.000	0.000	0.000	0.000	0.000	m liq
Friction loss in discharge pipework									
Pipe Section 5	Pump Discharge	dP_5 =		1.385	1.262	1.145	0.747	1.524	m liq
Pipe Section 6	Pump station header	dP_6 =		0.197	0.180	0.163	0.107	0.217	m liq
Pipe Section 7	Flowmeter	dP_7 =		0.319	1.163	1.055	0.689	1.404	m liq
Pipe Section 8	0	dP_8 =		0.175	0.639	0.580	0.379	0.772	m liq
Pipe Section 9	Not Used	dP_9 =		0.000	0.000	0.000	0.000	0.000	m liq
Pipe Section 10	Not Used	dP_{10} =		0.000	0.000	0.000	0.000	0.000	m liq
Control Valve	Not Used	dpV =		0.000	0.000	0.000	0.000	0.000	m liq
Total		DHd =	$dP_5 + dP_6 + dP_7 + dP_8 + dP_9 + dP_{10}$	2.077	3.244	2.942	1.921	3.918	m liq

5. Summary

			Hydraulic Profile at duty start	Hydraulic Profile at Standby start	500+345	404+279	577+400	
Safety margin on dynamic losses	dP% =		5.00%	5.00%	5.00%	5.00%	5.00%	
Suction dynamic losses	SHd% =	(1 + dp%) x SHd	0.000	0.000	0.000	0.000	0.000	m liq
Discharge dynamic losses	DHd% =	(1 + dp%) x DHd	2.181	3.406	3.089	2.017	4.114	m liq
Total dynamic losses	Hd% =	SHd% + Dhd%	2.181	3.406	3.089	2.017	4.114	m liq
Total suction head	TSHg =	SHs - SHd%	1.700	2.350	2.350	2.350	2.350	m liq g
Total required discharge head	TDHg =	DHs + DHd%	14.541	15.766	15.449	14.377	16.474	m liq g
Calculated Differential Head Requirements	DHr =	TDHg - TSHg	**12.841**	**13.416**	**13.099**	**12.027**	**14.124**	m liq
	=	DHr x Dens / $Dens_{H2O}$	**12.841**	**13.416**	**13.099**	**12.027**	**14.124**	m H_2O

Hydraulic Profile	Hydraulic Profile	500+345	404+279	577+400

6. NPSH Available (Assuming elevation & velocity head negligible)

			at duty start	at Standby start				
NPSHA Available	*NPSHa =*	*101.3/Densx1000/9.81+TSHg*	12.026	12.676	12.676	12.676	12.676	m liq

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7. Estimated Power Required

Assumed efficiency	*Peff =*		70.00%	70.00%	70.00%	70.00%	70.00%	
Estimated absorbed pump power	*Pabs =*	$\frac{gp \times DHr \times Dens \times g}{Peff}$	49.49	49.35	45.89	34.05	57.10	kW

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8. Notes

INLET PUMP STATION

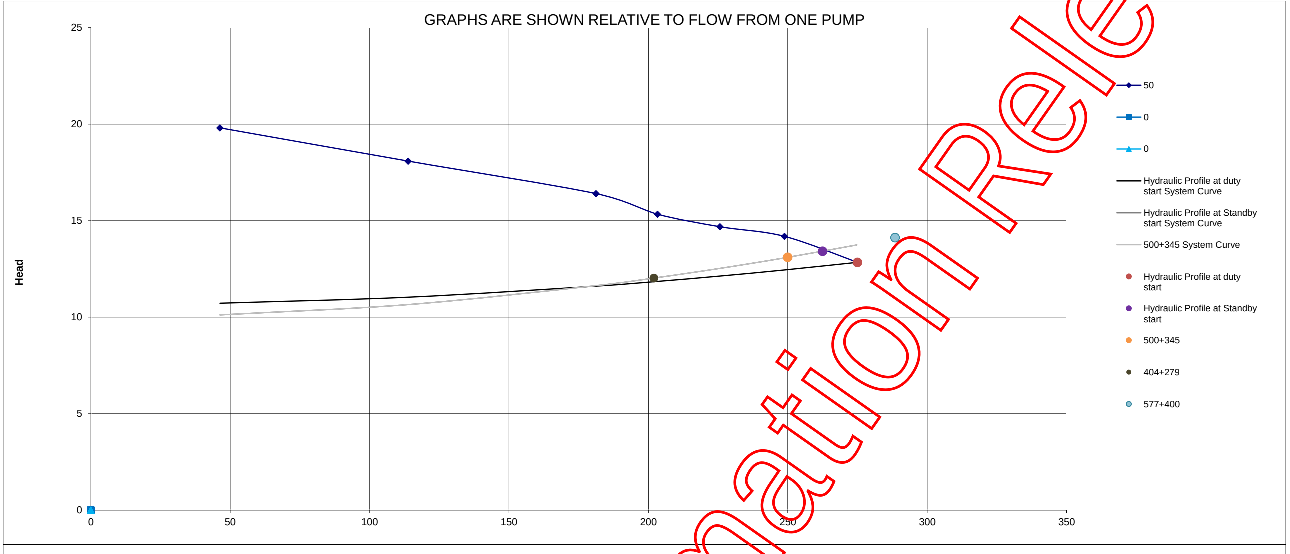
Performance Curves Resulting from VSD Speeds

	Existing N1	N2	N3
Speed	50	0	0
Flow multiplier	*N2/N1*	0	0
Head Multiplier	*(N2/N1)* ^{2}	0	0
Power Multimplier	*(N2/N1)* ^{3}	0	0

System Curve (Default figures from Pump Sizing spreadsht)

Static head [m H2O]	10.66	10.01	10.01	10.01	10.01
Duty flow [L/s]	275	262.5	250	202	288.5
Duty head [m H2O]	12.84083222	13.41584342	13.09930874	12.0272465	14.1236755
Coefficient	2.88375E-05	4.94272E-05	4.94289E-05	4.9437E-05	4.9424E-05

Flow at 50 [L/s]	Head at 50 [m H2O]	Power at 50 [kW]	Eff at 50 [%]	Flow at 0 [L/s]	Head at 0 [m H2O]	Power at 0 [kW]	Eff at #REF! [%]	Flow at 0 [L/s]	Head at 0 [m H2O]	Power at 0 [kW]	Eff at 0 [%]	Hydraulic Profile at duty start System	Hydraulic Profile at Standby start	500+345 System Curve	404+279 (Default figures from	577+400
46.31	19.808	1	899.88%	0.00	0.00	0.00	899.88%	0	0.00	0.00	899.88%	10.72	10.12	10.12	10.12	10.12
113.82	18.086	1	2019.44%	0.00	0.00	0.00	2019.44%	0	0.00	0.00	2019.44%	11.03	10.65	10.65	10.65	10.65
181.22	16.4	1	2915.54%	0.00	0.00	0.00	2915.54%	0	0.00	0.00	2915.54%	11.61	11.63	11.63	11.63	11.63
203.28	15.334	1	3057.87%	0.00	0.00	0.00	3057.87%	0	0.00	0.00	3057.87%	11.85	12.05	12.05	12.05	12.05
225.69	14.684	1	3251.07%	0.00	0.00	0.00	3251.07%	0	0.00	0.00	3251.07%	12.13	12.53	12.53	12.53	12.53
248.87	14.185	1	3463.15%	0.00	0.00	0.00	3463.15%	0	0.00	0.00	3463.15%	12.45	13.07	13.07	13.07	13.07
274.78	12.851	1	3464.11%	0.00	0.00	0.00	3464.11%	0	0.00	0.00	3464.11%	12.84	13.74	13.74	13.74	13.74
274.78	12.851	1	3464.11%	0.00	0.00	0.00	3464.11%	0	0.00	0.00	3464.11%	12.84	13.74	13.74	13.74	13.74
274.78	12.851	1	3464.11%	0.00	0.00	0.00	3464.11%	0	0.00	0.00	3464.11%	12.84	13.74	13.74	13.74	13.74
274.78	12.851	1	3464.11%	0.00	0.00	0.00	3464.11%	0	0.00	0.00	3464.11%	12.84	13.74	13.74	13.74	13.74



PIPE FROM INLET WORKS TO ANAEROBIC REACTOR

1. Design Input

			Case 1	Case 2	Case 3	Case 3	
Different cases for different flows and/or elevations but same piping system			500+345	404+279	577+400	Hydraulic profile	
Total flow	Q =	Choose units from drop down	500	404	577	718	L/s
	Qt =		1800.000	1454.400	2077.200	2584.800	m^3/hr
	qt =	$Qt / 3.6$	500.000	404.000	577.000	718.000	L/s
			0.500	0.404	0.577	0.718	m^3/s
			43.200	34.906	49.853	62.035	ML/d
Liquid: ?							
Density of pumped liquid	$Dens$ =		1000	1000	1000	1000	kg/m^3
Density of water	$Dens_{H2O}$ =		1000	1000	1000	1000	kg/m^3
Kinematic Viscosity of liquid	KV =	25 C	8.910E-07	8.910E-07	8.910E-07	8.910E-07	m^2/s
	$KVcst$ =	KV x 1E6	0.891	0.891	0.891	0.891	cSt

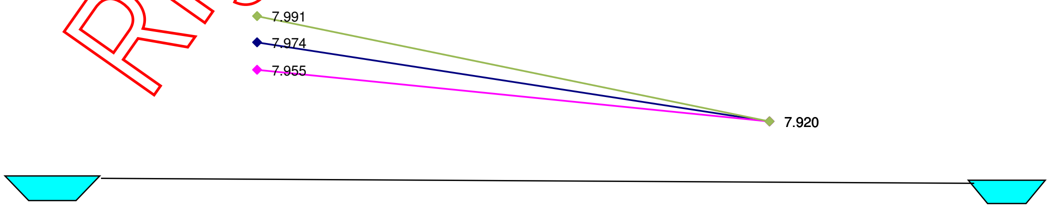
2. Dynamic Conditions

			500+345	404+279	577+400	Hydraulic profile	
Pipe Section 1	**Outlet from inlet works**						
Pipe size	960 OD MSCL		DN960	DN960	DN960	DN960	mm
Inside Diameter	d_1 =	Use accurate internal diameter from tables	912	912	912	912	mm
	D_1 =	$d_1 / 1000$	0.912	0.912	0.912	0.912	m
Area	A_1 =	$\Pi / 4 \times D_1^2$	0.653	0.653	0.653	0.653	m^2
Number of streams for total flow	S_1 =	Default from Design Inputs	1	1	1	1	
Flow for this pipe section		Default from Design Inputs	1800.000	1454.400	2077.200	2584.800	m^3/hr
Additional flows from another source		Use for multiple stations, dosing points etc					m3/hr
Total flow for this pipe section	Q_1 =		1800.000	1454.400	2077.200	2584.800	m^3/h
	q_1 =	$Q_1 / 3.6$	500.000	404.000	577.000	718.000	L/s
Velocity	V_1 =	$\frac{Q_1}{A_1 \times 3600}$	0.765	0.618	0.883	1.099	m/sec
Pipe Wall Roughness	k_1 =	See attached worksheet	3	3	3	3	mm
			0.003	0.003	0.003	0.003	m
Reynolds number	Re_1 =	$\frac{V_1 \times D_1}{KV}$	783443	633022	904094	1125025	
Reynolds number is above 2500, therefore flow may be considered turbulent							
Friction factor (Swamee & Jain modified CW equ.)	f_1 =	$\frac{0.25}{(\log (k1 / 3.7 / D1 + 5.74 / Re1^{0.9}))^2}$	0.027	0.027	0.027	0.027	
Hydraulic gradient	HG_1 =	$\frac{f_1 \times 100 \times V_1^2}{D_1 \times 2 \times g}$	0.089	0.058	0.118	0.182	m/100 m
Qty	k value						
7 m of pipe length		$\times HG_1 / 100$	0.006	0.004	0.008	0.013	m liq
1 x Inlet Sharp Edged	0.5	per fitting $\times V_1^2 / 2 / g$	0.015	0.010	0.020	0.031	m liq
1 x Enlargement Sudden	1	per fitting $\times V_1^2 / 2 / g$	0.030	0.019	0.040	0.062	m liq
Sub total	dP_1 =	Sum of friction losses	0.051	0.033	0.068	0.105	m liq

3. Total Dynamic Losses

			500+345	404+279	577+400	Hydraulic profile	
Friction loss in pipework							
Pipe Section 1	Outlet from inlet works	dP_1 =	0.051	0.033	0.068	0.105	m liq
Total	DHd =	$dP1 + dP2 + dP3 + dP4 + dP_5 + dP_6 + dP_7 + dP_8 + dP_9 + dP_{10}$	0.051	0.033	0.068	0.105	m liq
Safety margin on dynamic losses	$dP\%$ =		5.00%	5.00%	5.00%	5.00%	
Dynamic losses	$Hd\%$ =	$(1 + dp\%) \times DHd$	0.054	0.035	0.071	0.110	m liq

4. Elevations



	500+345	404+279	577+400	Hydraulic profile	
$Hd\%$ =	0.054	0.035	0.071	0.110	m

Inlet elevation liquid level	*ELi =*	*ELo + HD%*	7.974	7.955	7.991	8.030	m EL
Outlet elevation liquid level	*ELo =*	Top Water Level Downstream	7.920	7.920	7.920	7.920	m EL

ANAEROBIC REACTOR INLET WEIR

1. Design Input

			Case 1	Case 2	Case 3	Case 4	
Different cases			500+345	404+279	577+400	Hydraulic profile	
Flow per clarifier	*Q =*		500	404	577	780	L/s
	Qt =		1800.000	1454.400	2077.200	2808.000	m³/hr
	qt =	*Qt / 3.6*	500.000	404.000	577.000	780.000	L/s
	qts =	*qt / 1000*	0.500	0.404	0.577	0.780	m³/s
			43.200	34.906	49.853	67.392	ML/d
Hydraulic drop						80mm	

2. Dynamic Conditions

Weir width	Flooded weir - CMP Flooded Weir Calculator used	900	900	900	900	mm
Downstream TWL		7.840	7.840	7.840	7.840	m
Upstream TWL		7.870	7.860	7.880	7.920	m
		30	20	40	80	mm

OXIDATION DITCH OUTLET WEIR

1. Design Input

			Case 1	Case 2	Case 3	Case 4	
Different cases			Matching hydraulic drop in drawings	500+345	404+279	577+400	
Flow per clarifier	*Q =*		340	845	683	977	L/s
	Qt =		1224.000	3042.000	2458.800	3517.200	m³/hr
	qt =	*Qt / 3.6*	340.000	845.000	683.000	977.000	L/s
	qts =	*qt / 1000*	0.340	0.845	0.683	0.977	m³/s
			29.376	73.008	59.011	84.413	ML/d

2. Dynamic Conditions

Weir width			5084	5084	5084	5084	mm
	b =		5.084	5.084	5.084	5.084	m
Height over weir is	*h =*	$\left(\frac{qts}{0.595 \times 2/3 \times \sqrt{2g} \times (b-0.003)}\right)^{2/3}+0.001$	0.114	0.209	0.181	0.230	m
			114	209	181	230	mm
TWL in Oxidation Ditch			7.560	7.560	7.560	7.560	m
Weir in down position			7.080	7.080	7.080	7.080	m
			480	480	480	480	mm

PIPE FROM OXIDATION DITCH TO MIXED LIQUOR DISTRIBUTOR

1. Design Input

			Case 1	Case 2	Case 3	Case 3	
Different cases for different flows and/or elevations but same piping system			500+345	404+279	577+400	Hydraulic profile	
Total flow	*Q =*	Choose units from drop down	845	683	977	1460	L/s
	Qt =		3042.000	2458.800	3517.200	5256.000	m³/hr
	qt =	*Qt / 3.6*	845.000	683.000	977.000	1460.000	L/s
			0.845	0.683	0.977	1.460	m³/s
			73.008	59.011	84.413	126.144	ML/d
Liquid:	?						
Density of pumped liquid	*Dens =*		1000	1000	1000	1000	kg/m³
Density of water	*Dens $_{H2O}$ =*		1000	1000	1000	1000	kg/m³
Kinematic Viscosity of liquid	*KV =*	20 C	8.910E-07	8.910E-07	8.910E-07	8.910E-07	m²/s
	KVcst =	*KV x 1E6*	0.891	0.891	0.891	0.891	cSt

2. Dynamic Conditions

Pipe Section 1			500+345	404+279	577+400	Hydraulic profile	
Pipe size	mscl		DN960	DN960	DN960	DN960	mm
Inside Diameter	*d $_1$ =*	Use accurate internal diameter from tables	912	912	912	912	mm
	D $_1$ =	*d $_1$ / 1000*	0.912	0.912	0.912	0.912	m
Area	*A $_1$ =*	*Π / 4 x D $_1$ ²*	0.653	0.653	0.653	0.653	m²
Number of streams for total flow	*S $_1$ =*	Default from Design Inputs	1	1	1	1	
Flow for this pipe section		Default from Design Inputs	3042.000	2458.800	3517.200	5256.000	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc					m3/hr
Total flow for this pipe section	*Q $_1$ =*		3042.000	2458.800	3517.200	5256.000	m³/h

	q_1 =	Q_1 / 3.6	845.000	683.000	977.000	1460.000	L/s
Velocity	V_1 =	$\frac{Q_1}{A_1 \times 3600}$	1.294	1.046	1.496	2.235	m/sec
Pipe Wall Roughness	k_1 =	See attached worksheet	3	3	3	3	mm
			0.003	0.003	0.003	0.003	m
Reynolds number	Re_1 =	$\frac{V_1 \times D_1}{KV}$	1324019	1070184	1530848	2287654	

Reynolds number is above 2500, therefore flow may be considered turbulent

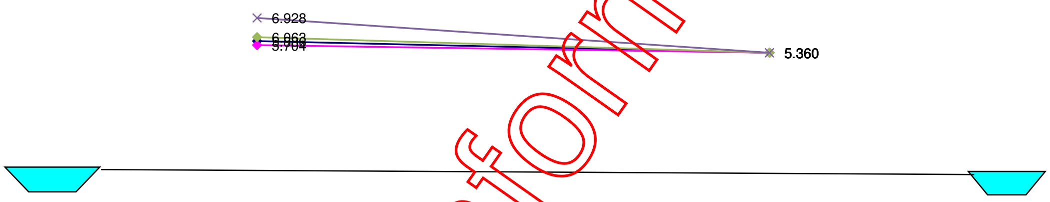
Friction factor	f_1 =	$\frac{0.25}{(\log(k1 / 3.7 / D1 + 5.74 / Re1^{0.9}))^2}$	0.027	0.027	0.027	0.027	
(Swamee & Jain modified CW equ.)							
Hydraulic gradient	HG_1 =	$\frac{f_1 \times 100 \times V_1^2}{D_1 \times 2 \times g}$	0.253	0.165	0.337	0.752	m/100 m

Qty		k value						
102	m of pipe length		x HG_1 / 100	0.258	0.168	0.344	0.767	m liq
2	x Elbow Mitre 90 4 piece	0.3	per fitting x $V_1^2 / 2 / g$	0.051	0.033	0.068	0.153	m liq
1	x Bend Medium Radius 90	0.75	per fitting x $V_1^2 / 2 / g$	0.064	0.042	0.086	0.191	m liq
1	x Inlet Sharp Edged	0.5	per fitting x $V_1^2 / 2 / g$	0.043	0.028	0.057	0.127	m liq
1	x Enlargement Sudden	1	per fitting x $V_1^2 / 2 / g$	0.085	0.056	0.114	0.255	m liq
Sub total		dP_1 =	Sum of friction losses	0.501	0.327	0.669	1.493	m liq

3. Total Dynamic Losses

				500+345	404+279	577+400	Hydraulic profile	
Friction loss in pipework								
Pipe Section 1	0	dP_1 =		0.501	0.327	0.669	1.493	m liq
Total		DHd =	$dP1 + dP2 + dP3 + dP4 + dP_5 + dP_6 + dP_7 + dP_8 + dP_9 + dP_{10}$	0.501	0.327	0.669	1.493	m liq
Safety margin on dynamic losses		dP% =		5.00%	5.00%	5.00%	5.00%	
Dynamic losses		Hd% =	(1 + dp%) x DHd	0.526	0.344	0.703	1.568	m liq

4. Elevations



			500+345	404+279	577+400	Hydraulic profile	
	Hd% =		0.526	0.344	0.703	1.568	mm
Inlet elevation liquid level	ELi =	ELo + HD%	5.886	5.704	6.063	6.928	m EL
Outlet elevation liquid level	ELo =	Top Water Level Downstream	5.360	5.360	5.360	5.360	m EL

MIXED LIQUOR DISTRIBUTOR

1. Design Input

			Case 1	Case 2	Case 3	Case 4	Case 5
Different cases			Matching hydraulic drop in drawings	500+345	404+279	577+400	If only leave 300mm freeboard
Flow per clarifier	Q =		340	422.5	350.5	488.5	727
	Qt =		1224.000	1521.000	1261.800	1758.600	2617.200
	qt =	Qt / 3.6	340.000	422.500	350.500	488.500	727.000
	qts =	qt / 1000	0.340	0.423	0.351	0.489	0.727
			29.376	36.504	30.283	42.206	62.813

Hydaulic drop in drawngs is 290mm

2. Dynamic Conditions

Weir width		Each of the two weirs in the flow splitter is	1250	1250	1250	1250	1250
	b =		1.25	1.25	1.25	1.25	1.25
Height over weir is	h =	$\left(\frac{qts}{0.595 \times 2/3 \times \sqrt{2g} \times (b-0.003)}\right)^{2/3} + 0.001$	0.290	0.335	0.296	0.369	0.480
			290	335	296	369	480

PIPE FROM MIXED LIQUOR DISTRIIBUTOR TO CLARIFIER

1.	Design Input	Case 1	Case 2	Case 3	Case 4

Different cases for different flows and/or elevations but same piping system			Matching hydraulic drop in drawings	500+345	404+279	577+400	
Total flow	$Q =$	Choose units from drop down	1034	845	701	977	L/s
	$Qt =$		3722.400	3042.000	2523.600	3517.200	m³/hr
	$qt =$	$Qt / 3.6$	1034.000	845.000	701.000	977.000	L/s
			1.034	0.845	0.701	0.977	m³/s
			89.338	73.008	60.566	84.413	ML/d
Liquid:		?					
Density of pumped liquid	$Dens =$		1000	1000	1000	1000	kg/m³
Density of water	$Dens_{H2O} =$		1000	1000	1000	1000	kg/m³
Kinematic Viscosity of liquid	$KV =$		8.910E-07	8.910E-07	8.910E-07	8.910E-07	m²/s
	$KVcst =$	$KV \times 1E6$	0.891	0.891	0.891	0.891	cSt

2. Dynamic Conditions

Pipe Section 1	Mixed Liquor Distributor to Clarifer		Case 1	Case 2	Case 3	Case 4	
Pipe size	MSCL		DN960	DN960	DN960	DN960	mm
Inside Diameter	$d_1 =$	Use accurate internal diameter from tables	912	912	912	912	mm
	$D_1 =$	$d_1 / 1000$	0.912	0.912	0.912	0.912	m
Area	$A_1 =$	$\Pi / 4 \times D_1{}^2$	0.653	0.653	0.653	0.653	m²
Number of streams for total flow	$S_1 =$	Default from Design Inputs	2	2	2	2	
Flow for this pipe section		Default from Design Inputs	1861.200	1521.000	1261.800	1758.600	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc					m3/hr
Total flow for this pipe section	$Q_1 =$		1861.200	1521.000	1261.800	1758.600	m³/h
	$q_1 =$	$Q_1 / 3.6$	517.000	422.500	350.500	488.500	L/s
Velocity	$V_1 =$	$\frac{Q_1}{A_1 \times 3600}$	0.791	0.647	0.537	0.748	m/sec
Pipe Wall Roughness	$k_1 =$	See attached worksheet	3	3	3	3	mm
			0.003	0.003	0.003	0.003	m
Reynolds number	$Re_1 =$	$\frac{V_1 \times D_1}{KV}$	810080	662010	549194	765424	

Reynolds number is above 2500, therefore flow may be considered turbulent

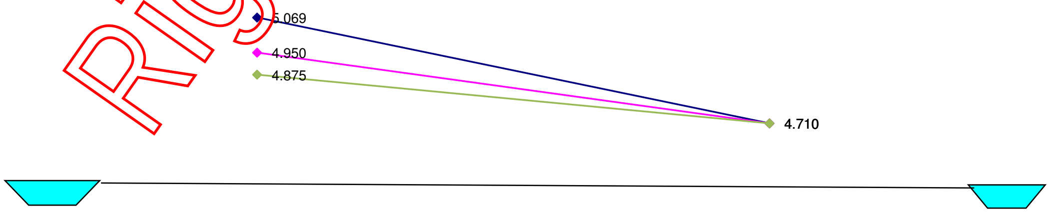
Friction factor (Swamee & Jain modified CW equ.)	$f_1 =$	$\frac{0.25}{(\log (k1 / 3.7 / D1 + 5.74 / Re1^{0.9}))^2}$	0.027	0.027	0.027	0.027	
Hydraulic gradient	$HG_1 =$	$\frac{f_1 \times 100 \times V_1{}^2}{D_1 \times 2 \times g}$	0.095	0.063	0.044	0.085	m/100 m

Qty		k value						
35.5	m of pipe length		$x HG_1 / 100$	0.034	0.023	0.016	0.030	m liq
1	x Inlet Sharp Edged	0.5	per fitting $x V_1{}^2 / 2 / g$	0.016	0.011	0.007	0.014	m liq
2	x Elbow Mitre 90 4 piece	0.3	per fitting $x V_1{}^2 / 2 / g$	0.019	0.013	0.009	0.017	m liq
0	Assumed losses through clarifier entry slots		$Q=0.62 A \sqrt{2gh}$	0.273	0.183	0.126	0.244	m liq
Sub total		$dP_1 =$	Sum of friction losses	0.342	0.229	0.157	0.306	m liq

3. Total Dynamic Losses

Friction loss in pipework							
Pipe Section 1	Mixed Liquor Distributor to Cl; $dP_1 =$		0.342	0.229	0.157	0.306	m liq
Total	$DHd =$	$dP1 + dP2 + dP3 + dP4 + dP_5 + dP_6 + dP_7 + dP_8 + dP_9 + dP_{10}$	0.342	0.229	0.157	0.306	m liq
Safety margin on dynamic losses	$dP\% =$		5.00%	5.00%	5.00%	5.00%	
Dynamic losses	$Hd\% =$	$(1 + dp\%) \times DHd$	0.359	0.240	0.165	0.321	m liq

4. Elevations



	$Hd\% =$		0.359	0.240	0.165	0.321	mm
Inlet elevation liquid level	$ELi =$	$ELo + HD\%$	5.069	4.950	4.875	5.031	m EL
Outlet elevation liquid level	$ELo =$	Top Water Level Downstream	4.710	4.710	4.710	4.710	m EL

PIPE FROM CLARIFIER OUTLETS TO FILTER FEED TANK

1. Design Input

			Case 1	Case 2	Case 3	Case 3	
Different cases for different flows and/or elevations but same piping system			Matching hydraulic drop in drawings	500+345	404+279	577+400	
Total flow	Q =	Choose units from drop down	754	500	404	577	L/s
	Qt =		2714.400	1800.000	1454.400	2077.200	m^3/hr
	qt =	Qt / 3.6	754.000	500.000	404.000	577.000	L/s
			0.754	0.500	0.404	0.577	m^3/s
			65.146	43.200	34.906	49.853	ML/d
Liquid:	?						
Density of pumped liquid	Dens =		1000	1000	1000	1000	kg/m^3
Density of water	$Dens_{H2O}$ =		1000	1000	1000	1000	kg/m^3
Kinematic Viscosity of liquid	KV =		8.910E-07	8.910E-07	8.910E-07	8.910E-07	m^2/s
	KVcst =	KV x 1E6	0.891	0.891	0.891	0.891	cSt

2. Dynamic Conditions

Pipe Section 1	**Clarifier to tee**		Matching hydraulic drop in	500+345	404+279	577+400	
Pipe size			DN960	DN960	DN960	DN960	mm
Inside Diameter	d_1 =	Use accurate internal diameter from tables	912	912	912	912	mm
	D_1 =	d_1 / 1000	0.912	0.912	0.912	0.912	m
Area	A_1 =	$\Pi / 4 \times D_1^2$	0.653	0.653	0.653	0.653	m^2
Number of streams for total flow	S_1 =	Default from Design Inputs	2	2	2	2	
Flow for this pipe section		Default from Design Inputs	1357.200	900.000	727.200	1038.600	m^3/hr
Additional flows from another source		Use for multiple stations, dosing points etc					m3/hr
Total flow for this pipe section	Q_1 =		1357.200	900.000	727.200	1038.600	m^3/h
	q_1 =	Q_1 / 3.6	377.000	250.000	202.000	288.500	L/s
Velocity	V_1 =	$\frac{Q_1}{A_1 \times 3600}$	0.577	0.383	0.309	0.442	m/sec
Pipe Wall Roughness	k_1 =	See attached worksheet	3	3	3	3	mm
			0.003	0.003	0.003	0.003	m
Reynolds number	Re_1 =	$\frac{V_1 \times D_1}{KV}$	590716	391722	316511	452047	

Reynolds number is above 2500, therefore flow may be considered turbulent

Friction factor (Swamee & Jain modified CW equ.)	f_1 =	$\frac{0.25}{(\log (k1 / 3.7 / D1 + 5.74 / Re1^{0.9}))^2}$	0.027	0.027	0.027	0.027	
Hydraulic gradient	HG_1 =	$\frac{f_1 \times 100 \times V_1^2}{D_1 \times 2 \times g}$	0.051	0.022	0.015	0.030	m/100 m

Qty		k value						
8	m of pipe length		x HG_1 / 100	0.004	0.002	0.001	0.002	m liq
1	x Inlet Sharp Edged	0.5	per fitting x $V_1^2 / 2 / g$	0.008	0.004	0.002	0.005	m liq
1	x Bend Medium Radius 90	0.75	per fitting x $V_1^2 / 2 / g$	0.013	0.006	0.004	0.007	m liq
1	x Elbow Mitre 45	0.3	per fitting x $V_1^2 / 2 / g$	0.005	0.002	0.001	0.003	m liq
1	x Tee - in line	0.6	per fitting x $V_1^2 / 2 / g$	0.010	0.004	0.003	0.006	m liq
Sub total		dP_1 =	Sum of friction losses	0.041	0.018	0.012	0.024	m liq

Pipe Section 2	**Tee to Filter Water Tank**		Matching hydraulic drop in drawings	500+345	404+279	577+400	
Pipe size	Pipe size and material		DN960	DN960	DN960	DN960	
Inside Diameter	d_2 =	Use accurate internal diameter from tables	912	912	912	912	mm
	D_2 =	d_2 / 1000	0.912	0.912	0.912	0.912	m
Area	A_2 =	$\Pi / 4 \times D_2^2$	0.653	0.653	0.653	0.653	m^2
Number of streams for total flow	S_2 =	Default from Design Inputs	1	1	1	1	
Flow for this pump station		Default from previous section	2714.400	1800.000	1454.400	2077.200	m^3/hr
Additional flows from another source		Use for multiple stations, dosing points etc					m3/hr
Total flow for this pipe section	Q_2 =		2714.400	1800.000	1454.400	2077.200	m^3/h
	q_2 =	Q_2 / 3.6	754.000	500.000	404.000	577.000	L/s
Velocity	V_2 =	$\frac{Q_2}{A_2 \times 3600}$	1.154	0.765	0.618	0.883	m/sec
Pipe Wall Roughness	k_2 =	See attached worksheet	3	3	3	3	mm
			0.003	0.003	0.003	0.003	m

Reynolds number	Re_2 =	$\frac{V_2 \times D_2}{KV}$	1181433	783443	633022	904094	

Reynolds number is above 2500, therefore flow may be considered turbulent

Friction factor	f_2 =	0.25	0.027	0.027	0.027	0.027	
(Swamee & Jain modified CW equ.)		(log (k2 / 3.7 / D2 + 5.74 / Re2^0.9))²					
Hydraulic gradient	HG_2 =	$\frac{f_2 \times 100 \times V_2^2}{D_2 \times 2 \times g}$	0.201	0.089	0.058	0.118	m/100 m
Quantity	k value						
26 m of Pipe length		x HG_2 / 100	0.052	0.023	0.015	0.031	m liq
2 x Elbow Mitre 22.5	0.15	per fitting x $V_2^2/2/g$	0.020	0.009	0.006	0.012	m liq
1 x Enlargement Sudden	1	per fitting x $V_2^2/2/g$	0.068	0.030	0.019	0.040	m liq
Sub total	dP_2 =	Sum of friction losses	0.141	0.062	0.040	0.082	m liq

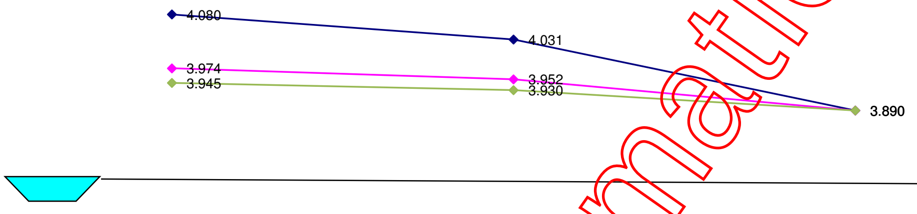
3. Total Dynamic Losses

Friction loss in pipework

Pipe Section 1	Clarifier to tee	dP_1 =		0.041	0.018	0.012	0.024	m liq
Pipe Section 2	Tee to Filter Water Tank	dP_2 =		0.141	0.062	0.040	0.082	m liq
Total		DHd =	$dP1 + dP2 + dP3 + dP4 + dP_5 + dP_6 + dP_7 + dP_8 + dP_9 + dP_{10}$	0.181	0.080	0.052	0.106	m liq

Safety margin on dynamic losses	dP% =		5.00%	5.00%	5.00%	5.00%	
Dynamic losses	Hd% =	(1 + dp%) x DHd	0.190	0.084	0.055	0.111	m liq

4. Elevations



			Matching hydraulic drop	500+345	404+279	577+400	
	Hd% =		0.190	0.084	0.055	0.111	mm
Inlet elevation liquid level	ELi =	ELo + HD%	4.080	3.974	3.945	4.001	m EL
Before Pipe Section 2		Tee to Filter Water Tank	4.031	3.952	3.930	3.972	m EL
Outlet elevation liquid level	ELo =	Top Water Level Downstream	3.890	3.890	3.890	3.890	m EL

FILTERS

Hydraulic gradient through clean media is

$$h = \frac{6\,(1-e)V^2}{d\,e^3 g} \text{ x } (5\ Re^{-1} + 0.4\ Re^{-0.1})$$

e = media voidage
d = hydraulic size of media
V = Filtration rate
Re = Reynolds number in media

In practical analysis, this cannot be worked out without a lot more information. The most effective way to address the hydraulic capacity of the filters is to look at the headlosses against outlet control valves and then extrapolate from there. If you are able to provide operatoinal information on the range of valve positions aginst dp, we could potentially do an estimate of the maximum possible flow rate.

A reasonable approximation would be to base the flow rate on 10 m/hr through the filters. This gives a flow of 442 L/s which is less than two of the three nominated conditions.

FILTERED WATER HOLDING TANK TO CHLORINE CONTACT TANK

1. Design Input

			Case 1	Case 2	Case 3	Case 3	
Different cases for different flows and/or elevations but same piping system			Matching hydraulic drop in drawings	500+345	404+279	577+400	
Total flow	Q =	Choose units from drop down	1012	500	404	577	L/s
	Qt =		3643.200	1800.000	1454.400	2077.200	m³/hr
	qt =	Qt / 3.6	1012.000	500.000	404.000	577.000	L/s
			1.012	0.500	0.404	0.577	m³/s
			87.437	43.200	34.906	49.853	ML/d
Liquid:	?						
Density of pumped liquid	Dens =		1000	1000	1000	1000	kg/m³
Density of water	$Dens_{H2O}$ =		1000	1000	1000	1000	kg/m³
Kinematic Viscosity of liquid	KV =		8.910E-07	8.910E-07	8.910E-07	8.910E-07	m²/s
	KVcst =	KV x 1E6	0.891	0.891	0.891	0.891	cSt

2. Dynamic Conditions

Pipe Section 1	?		Matching hydraulic drop in drawings	500+345	404+279	577+400	
Pipe size	Pipe size and material		DN960	DN960	DN960	DN960	mm
Inside Diameter	d_1 =	Use accurate internal diameter from tables	912	912	912	912	mm
	D_1 =	d_1 / 1000	0.912	0.912	0.912	0.912	m
Area	A_1 =	Π / 4 x $D_1{}^2$	0.653	0.653	0.653	0.653	m²
Number of streams for total flow	S_1 =	Default from Design Inputs	1	1	1	1	
Flow for this pipe section		Default from Design Inputs	3643.200	1800.000	1454.400	2077.200	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc					m3/hr
Total flow for this pipe section	Q_1 =		3643.200	1800.000	1454.400	2077.200	m³/h
	q_1 =	Q_1 / 3.6	1012.000	500.000	404.000	577.000	L/s
Velocity	V_1 =	$\frac{Q_1}{A_1 \times 3600}$	1.549	0.765	0.618	0.883	m/sec
Pipe Wall Roughness	k_1 =	See attached worksheet	3	3	3	3	mm
			0.003	0.003	0.003	0.003	m
Reynolds number	Re_1 =	$\frac{V_1 \times D_1}{KV}$	1585689	783443	633022	904094	

Reynolds number is above 2500, therefore flow may be considered turbulent

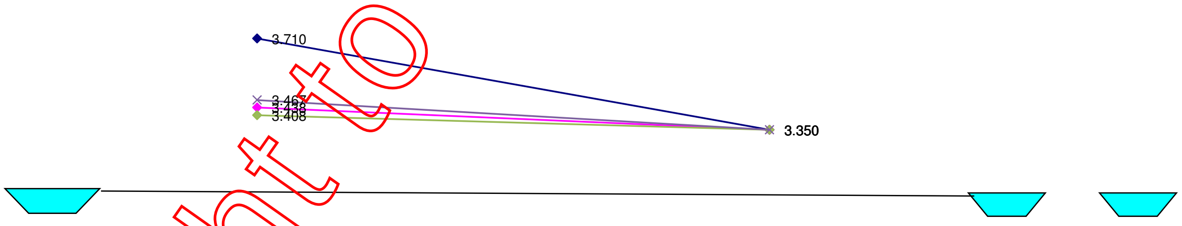
Friction factor (Swamee & Jain modified CW equ.)	f_1 =	$\frac{0.25}{(\log(k1 / 3.7 / D1 + 5.74 / Re1^{0.9}))^2}$	0.027	0.027	0.027	0.027	
Hydraulic gradient	HG_1 =	$\frac{f_1 \times 100 \times V_1{}^2}{D_1 \times 2 \times g}$	0.362	0.089	0.058	0.118	m/100 m

Qty		k value						
44	m of pipe length		x HG_1 / 100	0.159	0.039	0.026	0.052	m liq
1	x Inlet Sharp Edged	0.5	per fitting x $V_1{}^2$ / 2 / g	0.061	0.015	0.010	0.020	m liq
1	x Enlargement Sudden	1	per fitting x $V_1{}^2$ / 2 / g	0.122	0.030	0.019	0.040	m liq
Sub total		dP_1 =	Sum of friction losses	0.343	0.084	0.055	0.112	m liq

3. Total Dynamic Losses

Friction loss in pipework			Matching hydraulic drop in	500+345	404+279	577+400	
Pipe Section 1 ?	dP_1 =		0.343	0.084	0.055	0.112	m liq
Total	DHd =	$dP1 + dP2 + dP3 + dP4 + dP_5 + dP_6 + dP_7 + dP_8 + dP_9 + dP_{10}$	0.343	0.084	0.055	0.112	m liq
Safety margin on dynamic losses	dP% =		5.00%	5.00%	5.00%	5.00%	
Dynamic losses	Hd% =	(1 + dp%) x DHd	0.360	0.088	0.058	0.117	m liq

4. Elevations



			Matching hydraulic drop	500+345	404+279	577+400	
	Hd% =		0.360	0.088	0.058	0.117	mm
Inlet elevation liquid level	ELi =	ELo + HD%	3.710	3.438	3.408	3.467	m EL
Outlet elevation liquid level	ELo =	Top Water Level Downstream	3.350	3.350	3.350	3.350	m EL

CHLORINE CONTACT TANK OUTLET WEIRS

1. Design Input

			Case 1	Case 2	Case 3	Case 3	
Different cases			To existing secondary clarifier	To outfall	?	?	
Total flow	Q =		1610	4835			L/s
	Qt =		5796.000	17406.000	0.000	0.000	m³/hr
	qt =	Qt / 3.6	1610.000	4835.000	0.000	0.000	L/s

	qts =	*qt / 1000*	1.610	4.835	0.000	0.000	m^3/s
			139.104	417.744	0.000	0.000	ML/d

Hydaulic drop in drawngs is 815mm.

2. Dynamic Conditions

Weir width	**Weir width is**		1250	3750			mm
	b =		1.25	3.75	0	0	m
Height over weir is	*h =*	$(\frac{qts}{0.595 \times 2/3 \times \sqrt{2g} \times (b - 0.003)})^{2/3}+0.001$	0.815	0.815	0.001	0.001	m
			815	815	1	1	mm

FILTER FEED PUMPS

1. Design Input

			Case 1	Case 2	Case 3	
Different cases for different flows and/or elevations but same piping system			40	45	50	
Pump Type		Submersible				
No of duty pumps	PN =		1	1	1	

Graphs on the System Curve worksheet will be displayed in the units selected below.

			Case 1	Case 2	Case 3	
Total flow	Q =	Choose units from drop down	500	404	577	L/s
	Qt =		1800.000	1454.400	2077.200	m³/hr
	qt =	Qt / 3.6	500.000	404.000	577.000	L/s
			0.500	0.404	0.577	m³/s
			43.200	34.906	49.853	ML/d
Flow per pump			500	404	577	L/s
	Qp =	Qt / PN	1800.000	1454.400	2077.200	m³/hr
	qp =	Qp / 3.6	500.000	404.000	577.000	L/s
Pumped liquid:		water				
Density of pumped liquid	Dens =		1000	1000	1000	kg/m³
Density of water	$Dens_{H2O}$ =		1000	1000	1000	kg/m³
Kinematic Viscosity of liquid	KV =		1.137E-06	8.910E-07	8.910E-07	m²/s
	KVcst =	KV x 1E6	1.137	0.891	0.891	cSt

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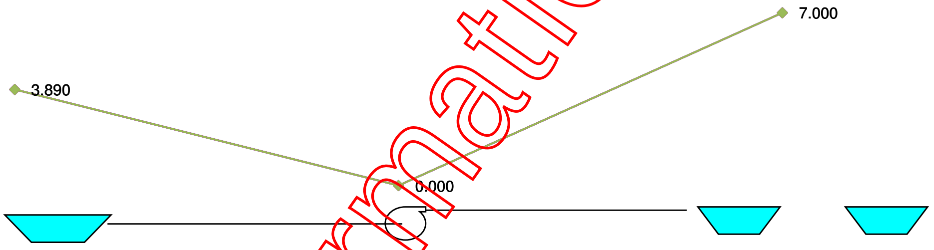
2. Static Conditions



2.1 Pump

			40	45	50	
Elevation of pump	ELp =		0.000	0.000	0.000	m EL

2.2 Suction

Elevation liquid level	ELsl =		3.890	3.890	3.890	m EL
Liquid pressure at pump	SPl =	ELsl - ELp	3.890	3.890	3.890	m liq
Air or gas pressure	SPg =	e.g. pumping from pressurised system				kPag
Equivalent liquid head due to air pressure	SPm =	SPg / Dens / g x 1E3	0.000	0.000	0.000	m liq
Static suction head	SHs =	SPl + SPm	3.890	3.890	3.890	m liq

2.3 Discharge

Elevation liquid level	ELdl =		7.000	7.000	7.000	m EL
Liquid pressure at pump	DPl =	ELdl - Elp	7.000	7.000	7.000	m liq
Air or gas pressure	DPg =	e.g. pumping to pressurised system				kPag
Equivalent liquid head due to air pressure	DPm =	DPg / Dens / g x 1E5	0.000	0.000	0.000	m liq
Static discharge head	DHs =	DPl + DPm	7.000	7.000	7.000	m liq

2.4 Static Head

Static differential head	Hs =	DHs - SHs	3.110	3.110	3.110	m liq

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3. Dynamic Conditions

3.1 Suction

Pipe Section 1	**Not used**	40	45	50
Pipe Section 2	**Not used**	40	45	50
Pipe Section 3	**Not Used**	40	45	50
Pipe Section 4	**Not Used**	40	45	50

3.2 Discharge

Pipe Section 5	**Pump Discharge**		40	45	50	
Pipe size	St Stl		DN500	DN500	DN500	mm
Inside Diameter	d_5 =	Use accurate internal diameter from tables	495.3	495.3	495.3	mm
	D_5 =	d_5 / 1000	0.4953	0.4953	0.4953	m
Area	A_5 =	Π / 4 x D_5^2	0.193	0.193	0.193	m² area
Number of streams for total flow	S_5 =	Default from Design Inputs	1	1	1	
Flow for this pump station		Default from previous section	1800	1454.4	2077.2	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section	Q_5 =		1800.000	1454.400	2077.200	m³/h
	q_5 =	Q_5 / 3.6	500.000	404.000	577.000	L/s
Velocity	V_5 =	$\underline{Q_5}$	2.595	2.097	2.995	m/sec
		A_5 x 3600				
Pipe Wall Roughness	k_5 =		3	3	3	mm
			0.003	0.003	0.003	m
Reynolds number	Re_5 =	$\underline{V_5}$ x D_5	1130450	1165589	1664715	
		KV				
Reynolds number is above 2500, therefore flow may be considered turbulent						
Friction factor	f_5 =	*0.25*	0.032	0.032	0.032	
(Swamee & Jain modified CW equ.)		*(log (k5 / 3.7 / D5 + 5.74 / Re5^0.9))²*				
Hydraulic gradient	HG_5 =	$\underline{f_5}$ x 100 x V_5^2	2.241	1.463	2.981	m/100 m
		D_5 x 2 x g				
Quantity	k value					
13 m of Pipe length		*x HG5 / 100*	0.291	0.190	0.387	m liq
1 x Valve - Check wafer	3	*per fitting x* V_5^2 / 2 / g	1.030	0.672	1.371	m liq
1 x Valve - Butterfly full bore	0.4	*per fitting x* V_5^2 / 2 / g	0.137	0.090	0.183	m liq
1 x Tee Sharp Edge - branch	1.2	*per fitting x* V_5^2 / 2 / g	0.412	0.269	0.549	m liq
Sub total	dP_5 =	*Sum of friction losses*	1.870	1.221	2.490	m liq

Pipe Section 6	**afte 1 st offtake**		40	45	50	
Pipe size	st stl		DN500	DN500	DN500	mm
Inside Diameter	d_6 =	Use accurate internal diameter from tables	495.3	495.3	495.3	mm
	D_6 =	d_6 / 1000	0.4953	0.4953	0.4953	m
Area	A_6 =	Π / 4 x D_6^2	0.193	0.193	0.193	m² area
Number of streams for total flow	S_6 =	Default from Design Inputs	1.33333	1.33333	1.33333	
Flow for this pump station		Default from previous section	1350.003	1090.803	1557.904	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section	Q_6 =		1350.003	1090.803	1557.904	m³/h
	q_6 =	Q_6 / 3.6	375.001	303.001	432.751	L/s
Velocity	V_6 =	$\underline{Q_6}$	1.946	1.573	2.246	m/sec
		A_6 x 3600				
Pipe Wall Roughness	k_6 =	See attached worksheet	3	3	3	mm
			0.003	0.003	0.003	m
Reynolds number	Re_6 =	$\underline{V_6}$ x D_6	847840	874194	1248539	
		KV				
Reynolds number is above 2500, therefore flow may be considered turbulent						
Friction factor	f_6 =	*0.25*	0.032	0.032	0.032	
(Swamee & Jain modified CW equ.)		*(log (k6 / 3.7 / D6 + 5.74 / Re6^0.9))²*				
Hydraulic gradient	HG_6 =	$\underline{f_6}$ x 100 x V_6^2	1.262	0.824	1.678	m/100 m
		D_6 x 2 x g				
Quantity	k value					

6 m of Pipe length			x HG_6 / 100	0.076	0.049	0.101	m liq
1 x Tee	- in line	0.6	per fitting x V_6^2 / 2 / g	0.116	0.076	0.154	m liq
Sub total		dP_6 =	Sum of friction losses	0.192	0.125	0.255	m liq

Pipe Section 7	**After 2nd offtake**			40	45	50	
Pipe size	st stl			DN500	DN500	DN500	mm
Inside Diameter		d_7 =	Use accurate internal diameter from tables	495.3	495.3	495.3	mm
		D_7 =	d_7 / 1000	0.4953	0.4953	0.4953	m
Area		A_7 =	Π / 4 x D_7^2	0.193	0.193	0.193	m² area
Number of streams for total flow		S_7 =	Default from Design Inputs	2	2	2	
Flow for this pump station			Default from previous section	900.000	727.200	1038.600	m³/hr
Additional flows from another source			Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section		Q_7 =		900.000	727.200	1038.600	m³/h
		q_7 =	Q_7 / 3.6	250.000	202.000	288.500	L/s
Velocity		V_7 =	$\frac{Q_7}{A_7 \times 3600}$	1.298	1.048	1.497	m/sec
Pipe Wall Roughness		k_7 =	See attached worksheet	3	3	3	mm
				0.003	0.003	0.003	m
Reynolds number		Re_7 =	$\frac{V_7 \times D_7}{KV}$	565225	582795	832358	
Reynolds number is above 2500, therefore flow may be considered turbulent							
Friction factor		f_7 =	0.25	0.032	0.032	0.032	
(Swamee & Jain modified CW equ.)			(log (k7 / 3.7 / D7 + 5.74 / Re7^0.9))²				
Hydraulic gradient		HG_7 =	$\frac{f_7 \times 100 \times V_7^2}{D_7 \times 2 \times g}$	0.562	0.367	0.747	m/100 m
Quantity		k value					
6 m of Pipe length			x HG_7 / 100	0.034	0.022	0.045	m liq
1 x Tee	- in line	0.6	per fitting x V_7^2 / 2 / g	0.051	0.034	0.069	m liq
Sub total		dP_7 =	Sum of friction losses	0.085	0.056	0.113	m liq

Pipe Section 8	**After 3rd offtake**			40	45	50	
Pipe size	st stl			DN500	DN500	DN500	
Inside Diameter		d_8 =	Use accurate internal diameter from tables	495.3	495.3	495.3	mm
		D_8 =	d_8 / 1000	0.4953	0.4953	0.4953	m
Area		A_8 =	Π / 4 x D_8^2	0.193	0.193	0.193	m² area
Number of streams for total flow		S_8 =	Default from Design Inputs	4	4	4	
Flow for this pump station			Default from previous section	450.000	363.600	519.300	m³/hr
Additional flows from another source			Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section		Q_8 =		450.000	363.600	519.300	m³/h
		q_8 =	Q_8 / 3.6	125.000	101.000	144.250	L/s
Velocity		V_8 =	$\frac{Q_8}{A_8 \times 3600}$	0.649	0.524	0.749	m/sec
Pipe Wall Roughness		k_8 =	See attached worksheet	3	3	3	mm
				0.003	0.003	0.003	m
Reynolds number		Re_8 =	$\frac{V_8 \times D_8}{KV}$	282612	291397	416179	
Reynolds number is above 2500, therefore flow may be considered turbulent							
Friction factor		f_8 =	0.25	0.033	0.033	0.033	
(Swamee & Jain modified CW equ.)			(log (k8 / 3.7 / D8 + 5.74 / Re8^0.9))²				
Hydraulic gradient		HG_8 =	$\frac{f_8 \times 100 \times V_8^2}{D_8 \times 2 \times g}$	0.141	0.092	0.188	m/100 m
Quantity		k value					
6 m of Pipe length			x HG_8 / 100	0.008	0.006	0.011	m liq
1 x Tee Sharp Edge - branch		1.2	per fitting x V_8^2 / 2 / g	0.026	0.017	0.034	m liq
1 x Reducer 5:3		0.27	per fitting x V_8^2 / 2 / g	0.006	0.004	0.008	m liq
Sub total		dP_8 =	Sum of friction losses	0.040	0.026	0.053	m liq

				40	45	50	
Pipe Section 9	**Entrance to filter**						
Pipe size	st stl			DN300	DN300	DN300	
Inside Diameter		d_9 =	Use accurate internal diameter from tables	304.84	304.84	304.84	mm
		D_9 =	d_9 / 1000	0.30484	0.30484	0.30484	m
Area		A_9 =	Π / 4 x D_9^2	0.073	0.073	0.073	m² area
Number of streams for total flow		S_9 =	Default from Design Inputs	4	4	4	
Flow for this pump station			Default from previous section	450.000	363.600	519.300	m³/hr
Additional flows from another source			Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section		Q_9 =		450.000	363.600	519.300	m³/h
		q_9 =	Q_9 / 3.6	125.000	101.000	144.250	L/s
Velocity		V_9 =	$\frac{Q_9}{A_9 \times 3600}$	1.713	1.384	1.976	m/sec
Pipe Wall Roughness		k_9 =	See attached worksheet	3	3	3	mm
				0.003	0.003	0.003	m
Reynolds number		Re_9 =	$\frac{V_9 \times D_9}{KV}$	459185	473458	676202	

Reynolds number is above 2500, therefore flow may be considered turbulent

Friction factor (Swamee & Jain modified CW equ.)	f_9 =	$\frac{0.25}{(\log(k9 / 3.7 / D9 + 5.74 / Re9^{0.9}))^2}$	0.038	0.038	0.038	
Hydraulic gradient	HG_9 =	$\frac{f_9 \times 100 \times V_9^2}{D_9 \times 2 \times g}$	1.860	1.214	2.472	m/100 m

Quantity		k value					
1	m of Pipe length		x HG_9 / 100	0.019	0.012	0.025	m liq
1	x Elbow Short Radius 90	1	per fitting x V_9^2 / 2 / g	0.150	0.098	0.199	m liq
1	x Enlargement Sudden	1	per fitting x V_9^2 / 2 / g	0.150	0.098	0.199	m liq
1	x Valve - Butterfly full bore	0.4	per fitting x V_9^2 / 2 / g	0.060	0.039	0.080	m liq
Sub total		dP_9 =	Sum of friction losses	0.377	0.246	0.503	m liq

---

4. Total Dynamic Losses

				40	45	50	
Friction loss in suction pipework							
Pipe Section 1	Not used	dP_1 =		0.000	0.000	0.000	m liq
Pipe Section 2	Not used	dP_2 =		0.000	0.000	0.000	m liq
Pipe Section 3	Not Used	dP_3 =		0.000	0.000	0.000	m liq
Pipe Section 4	Not Used	dP_4 =		0.000	0.000	0.000	m liq
Total		SHd =	$dP_1 + dP_2 + dP_3 + dP_4$	0.000	0.000	0.000	m liq
Friction loss in discharge pipework							
Pipe Section 5	Pump Discharge	dP_5 =		1.870	1.221	2.490	m liq
Pipe Section 6	afte 1 st offtake	dP_6 =		0.192	0.125	0.255	m liq
Pipe Section 7	After 2nd offtake	dP_7 =		0.085	0.056	0.113	m liq
Pipe Section 8	After 3rd offtake	dP_8 =		0.040	0.026	0.053	m liq
Pipe Section 9	Entrance to filter	dP_9 =		0.377	0.246	0.503	m liq
Total		DHd =	$dP_5 + dP_6 + dP_7 + dP_8 + dP_9 +$	2.564	1.674	3.414	m liq

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5. Summary

			40	45	50	
Safety margin on dynamic losses	$dP\%$ =		5.00%	5.00%	5.00%	
Suction dynamic losses	$SHd\%$ =	(1 + dp%) x SHd	0.000	0.000	0.000	m liq
Discharge dynamic losses	$DHd\%$ =	(1 + dp%) x DHd	2.693	1.758	3.585	m liq
Total dynamic losses	$Hd\%$ =	SHd% + Dhd%	2.693	1.758	3.585	m liq
Total suction head	$TSHg$ =	SHs - SHd%	3.890	3.890	3.890	m liq g
Total required discharge head	$TDHg$ =	DHs + DHd%	9.693	8.758	10.585	m liq g
Calculated Differential Head Requirements	DHr =	TDHg - TSHg	**5.803**	**4.868**	**6.695**	m liq
	=	$DHr \times Dens / Dens_{H2O}$	**5.803**	**4.868**	**6.695**	m H_2O

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6. NPSH Available

(Assuming elevation & velocity head negligible)

			40	45	50	
NPSHA Available	$NPSHa$ =	101.3/Densx1000/9.81+TSHg	14.216	14.216	14.216	m liq

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7. Estimated Power Required

Assumed efficiency	*Peff =*		70.00%	70.00%	70.00%	
Estimated absorbed pump power	*Pabs =*	$\frac{qp \times DHr \times Dens \times g}{Peff}$	40.66	27.56	54.14	kW

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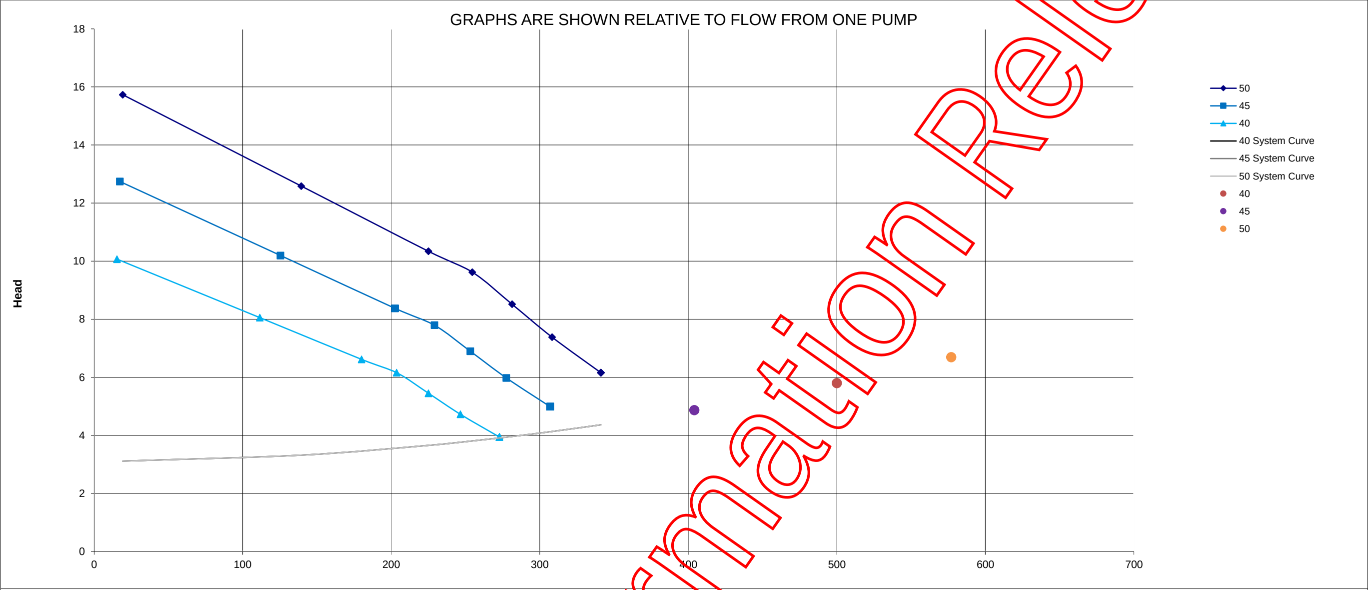
8. Notes

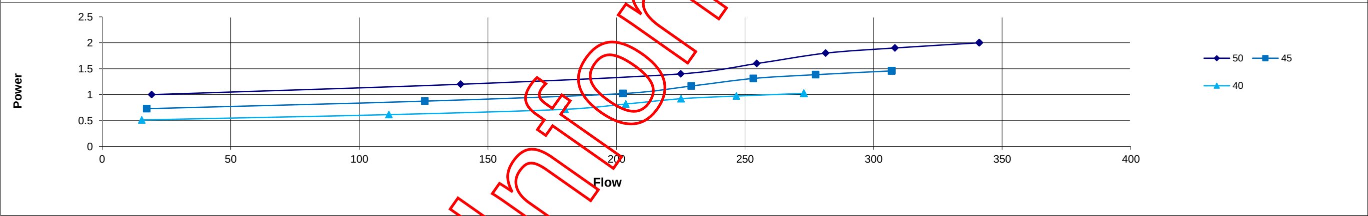
FILTER FEED PUMPS

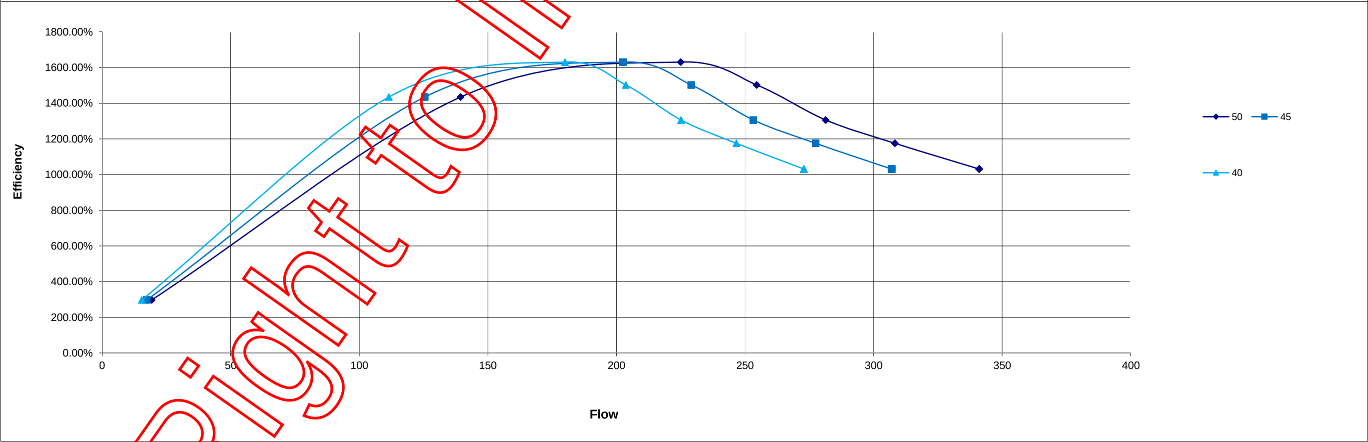
Performance Curves Resulting from VSD Speeds	**Existing N1**	**N2**	**N3**
Speed	50	45	40
Flow multiplier	*N2/N1*	0.9	0.8
Head Multiplier	*(N2/N1)* ^{2}	0.81	0.64
Power Multimplier	*(N2/N1)* ^{3}	0.729	0.512

System Curve	(Default figures from Pump Sizing spreadsht)		
Static head [m H2O]	3.11	3.11	3.11
Duty flow [L/s]	500	404	577
Duty head [m H2O]	5.802571477	4.867846484	6.694933674
Coefficient	1.07703E-05	1.07701E-05	1.07679E-05

Flow at 50 [L/s]	Head at 50 [m H2O]	Power at 50 [kW]	Eff at 50 [%]	Flow at 45 [L/s]	Head at 45 [m H2O]	Power at 45 [kW]	Eff at #REF! [%]	Flow at 40 [L/s]	Head at 40 [m H2O]	Power at 40 [kW]	Eff at 40 [%]	40 System Curve	45 System Curve	50 System Curve
19.25	15.732	1	297.09%	17.33	12.74	0.73	297.09%	15	10.07	0.51	297.09%	3.11	3.11	3.11
139.39	12.587	1.2	1434.31%	125.45	10.20	0.87	1434.31%	112	8.06	0.61	1434.31%	3.32	3.32	3.32
225.01	10.341	1.4	1630.44%	202.51	8.38	1.02	1630.44%	180	6.62	0.72	1630.44%	3.66	3.66	3.66
254.56	9.623	1.6	1501.93%	229.10	7.79	1.17	1501.93%	204	6.16	0.82	1501.93%	3.81	3.81	3.81
281.36	8.516	1.8	1305.85%	253.22	6.90	1.31	1305.85%	225	5.45	0.92	1305.85%	3.96	3.96	3.96
308.29	7.382	1.9	1175.03%	277.46	5.98	1.39	1175.03%	247	4.72	0.97	1175.03%	4.13	4.13	4.13
341.11	6.16	2	1030.66%	307.00	4.99	1.46	1030.66%	273	3.94	1.02	1030.66%	4.36	4.36	4.36
341.11	6.16	2	1030.66%	307.00	4.99	1.46	1030.66%	273	3.94	1.02	1030.66%	4.36	4.36	4.36
341.11	6.16	2	1030.66%	307.00	4.99	1.46	1030.66%	273	3.94	1.02	1030.66%	4.36	4.36	4.36
341.11	6.16	2	1030.66%	307.00	4.99	1.46	1030.66%	273	3.94	1.02	1030.66%	4.36	4.36	4.36







RAS PUMPS

1. Design Input

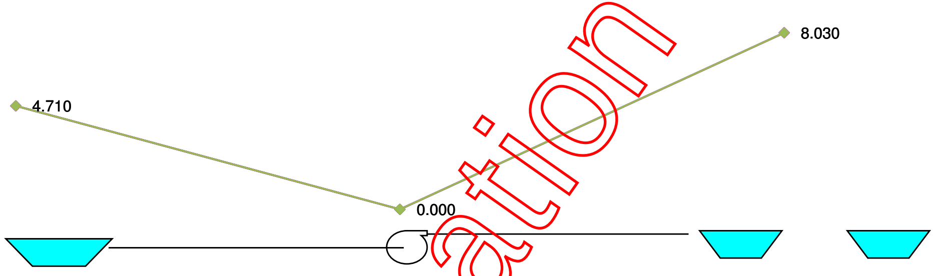
Different cases for different flows and/or elevations but same piping system

			Case 1			
			Nominal duty flow on test	Calculated	1 pump per clarifier	
Pump Type						
No of duty pumps	*PN =*		4	4	2	

Graphs on the System Curve worksheet will be displayed in the units selected below.

Total flow	*Q =*	Choose units from drop down	308	368	214	L/s
	Qt =		1108.800	1324.800	770.400	m³/hr
	qt =	*Qt / 3.6*	308.000	368.000	214.000	L/s
			0.308	0.368	0.214	m³/s
			26.611	31.795	18.490	ML/d
Flow per pump			77	92	107	L/s
	Qp =	*Qt / PN*	277.200	331.200	385.200	m³/hr
	qp =	*Qp / 3.6*	77.000	92.000	107.000	L/s
Pumped liquid:	water					
Density of pumped liquid	*Dens =*		1000	1000	1000	kg/m³
Density of water	$Dens_{H2O}$ =		1000	1000	1000	kg/m³
Kinematic Viscosity of liquid	*KV =*		1.137E-06	1.137E-06	1.137E-06	m²/s
	KVcst =	*KV x 1E6*	1.137	1.137	1.137	cSt

2. Static Conditions



2.1 Pump

			Nominal duty flow on test	Calculated	1 pump per clarifier	
Elevation of pump	*ELp =*		0.000	0.000	0.000	m EL

2.2 Suction

Elevation liquid level	*ELsl =*		4.710	4.710	4.710	m EL
Liquid pressure at pump	*SPl =*	*ELsl - ELp*	4.710	4.710	4.710	m liq
Air or gas pressure	*SPg =*	e.g. pumping from pressurised system				kPag
Equivalent liquid head due to air pressure	*SPm =*	*SPg / Dens / g x 1E3*	0.000	0.000	0.000	m liq
Static suction head	*SHs =*	*SPl + SPm*	4.710	4.710	4.710	m liq

2.3 Discharge

Elevation liquid level	*ELdl =*		8.030	8.030	8.030	m EL
Liquid pressure at pump	*DPl =*	*ELdl - Elp*	8.030	8.030	8.030	m liq
Air or gas pressure	*DPg =*	e.g. pumping to pressurised system				kPag
Equivalent liquid head due to air pressure	*DPm =*	*DPg / Dens / g x 1E5*	0.000	0.000	0.000	m liq
Static discharge head	*DHs =*	*DPl + DPm*	8.030	8.030	8.030	m liq

2.4 Static Head

Static differential head	*Hs =*	*DHs - SHs*	3.320	3.320	3.320	m liq

3. Dynamic Conditions

3.1 Suction

			Nominal duty flow	Calculated	1 pump per	
Pipe Section 1	**Not used**					
Pipe size	375 dicl		DN375	DN375	DN375	mm
Inside Diameter	d_1 =	Use accurate internal diameter from tables	406	406	406	mm

		D_1 =	d_1 / 1000	0.406	0.406	0.406	m
	Area	A_1 =	Π / 4 x D_1^2	0.129	0.129	0.129	m²
	Number of streams for total flow	S_1 =	Default from Design Inputs	2	2	2	
	Flow for this pipe section		Default from Design Inputs	554.4	662.4	385.2	m³/hr
	Additional flows from another source		Use for multiple stations, dosing points etc				m3/hr
	Total flow for this pipe section	Q_1 =		554.400	662.400	385.200	m³/h
		q_1 =	Q_1 / 3.6	154.000	184.000	107.000	L/s
	Velocity	V_1 =	Q_1	1.190	1.421	0.826	m/sec
			A_1 x 3600				
	Pipe Wall Roughness	k_1 =	See attached worksheet	3	3	3	mm
				0.003	0.003	0.003	m
	Reynolds number	Re_1 =	V_1 x D_1	424761	507506	295126	
			KV				
	Reynolds number is above 2500, therefore flow may be considered turbulent						
	Friction factor	f_1 =	0.25	0.035	0.035	0.035	
	(Swamee & Jain modified CW equ.)		(log (k1 / 3.7 / D1 + 5.74 / Re1^0.9))²				
	Hydraulic gradient	HG_1 =	f_1 x 100 x V_1^2	0.614	0.876	0.297	m/100 m
			D_1 x 2 x g				
Qty		k value					
24	m of pipe length		x HG_1 / 100	0.147	0.210	0.071	m liq
1	x Inlet Sharp Edged	0.5	per fitting x V_1^2 / 2 / g	0.036	0.051	0.017	m liq
2	x Elbow Mitre 45	0.3	per fitting x V_1^2 / 2 / g	0.043	0.062	0.021	m liq
1	x Tee Sharp Edge - branch	1.2	per fitting x V_1^2 / 2 / g	0.087	0.124	0.042	m liq
0	Select	0	per fitting x V_1^2 / 2 / g	0.000	0.000	0.000	m liq
0	Select	0	per fitting x V_1^2 / 2 / g	0.000	0.000	0.000	m liq
0	Select	0	per fitting x V_1^2 / 2 / g	0.000	0.000	0.000	m liq
0	Select	0	per fitting x V_1^2 / 2 / g	0.000	0.000	0.000	m liq
0	Select	0	per fitting x V_1^2 / 2 / g	0.000	0.000	0.000	m liq
0	Other			0.000	0.000	0.000	m liq
	Sub total	dP_1 =	Sum of friction losses	0.313	0.447	0.151	m liq
	Pipe Section 2			Nominal duty flow	Calculated	1 pump per	
	Pipe size			DN300	DN300	DN300	
	Inside Diameter	d_2 =	Use accurate internal diameter from tables	325	325	325	mm
		D_2 =	d_2 / 1000	0.325	0.325	0.325	m
	Area	A_2 =	Π / 4 x D_2^2	0.083	0.083	0.083	m²
	Number of streams for total flow	S_2 =	Default from Design Inputs	2	2	2	
	Flow for this pump station		Default from previous section	554.400	662.400	385.200	m³/hr
	Additional flows from another source		Use for multiple stations, dosing points etc				m3/hr
	Total flow for this pipe section	Q_2 =		554.400	662.400	385.200	m³/h
		q_2 =	Q_2 / 3.6	154.000	184.000	107.000	L/s
	Velocity	V_2 =	Q_2	1.856	2.218	1.290	m/sec
			A_2 x 3600				
	Pipe Wall Roughness	k_2 =	See attached worksheet	0	0	0	mm
				0	0	0	m
	Reynolds number	Re_2 =	V_2 x D_2	530624	633992	368680	
			KV				
	Reynolds number is above 2500, therefore flow may be considered turbulent						
	Friction factor	f_2 =	0.25	0.013	0.013	0.014	
	(Swamee & Jain modified CW equ.)		(log (k2 / 3.7 / D2 + 5.74 / Re2^0.9))²				
	Hydraulic gradient	HG_2 =	f_2 x 100 x V_2^2	0.700	0.968	0.361	m/100 m
			D_2 x 2 x g				
Quantity		k value					
4	m of Pipe length		x HG_2 / 100	0.028	0.039	0.014	m liq
1	x Tee - in line	0.6	per fitting x V_2^2 / 2 / g	0.105	0.150	0.051	m liq
1	x Valve - Gate	0.2	per fitting x V_2^2 / 2 / g	0.035	0.050	0.017	m liq
1	x Elbow Short Radius 90	1	per fitting x V_2^2 / 2 / g	0.176	0.251	0.085	m liq

Sub total	dP_2 =	*Sum of friction losses*	0.344	0.490	0.167	m liq

3.2 Discharge

Pipe Section 5	**Pump Discharge**		Nominal duty flow	Calculated	1 pump per	
Pipe size	DICL?		DN250	DN250	DN250	mm
Inside Diameter	d_5 =	Use accurate internal diameter from tables	266	266	266	mm
	D_5 =	d_5 / 1000	0.266	0.266	0.266	m
Area	A_5 =	Π / 4 x D_5^2	0.056	0.056	0.056	m² area
Number of streams for total flow	S_5 =	Default from Design Inputs	4	4	2	
Flow for this pump station		Default from previous section	277.200	331.200	385.200	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section	Q_5 =		277.200	331.200	385.200	m³/h
	q_5 =	Q_5 / 3.6	77.000	92.000	107.000	L/s
Velocity	V_5 =	$\frac{Q_5}{A_5 \times 3600}$	1.386	1.656	1.925	m/sec
Pipe Wall Roughness	k_5 =		3	3	3	mm
			0.003	0.003	0.003	m
Reynolds number	Re_5 =	$\frac{V_5 \times D_5}{KV}$	324159	387307	450455	
Reynolds number is above 2500, therefore flow may be considered turbulent						
Friction factor	f_5 =	$\frac{0.25}{(\log(k5 / 3.7 / D5 + 5.74 / Re5^{0.9}))^2}$	0.040	0.040	0.040	
(Swamee & Jain modified CW equ.)						
Hydraulic gradient	HG_5 =	$\frac{f_5 \times 100 \times V_5^2}{D_5 \times 2 \times g}$	1.463	2.087	2.820	m/100 m
Quantity	k value					
4 m of Pipe length		x HG_5 / 100	0.059	0.083	0.113	m liq
1 x Valve - Check conventional	2.4	per fitting x V_5^2 / 2 / g	0.235	0.335	0.453	m liq
1 x Valve - Gate	0.2	per fitting x V_5^2 / 2 / g	0.020	0.028	0.038	m liq
1 x Elbow Short Radius 90	1	per fitting x V_5^2 / 2 / g	0.098	0.140	0.189	m liq
1 x Tee - in line	0.6	per fitting x V_5^2 / 2 / g	0.059	0.084	0.113	m liq
Sub total	dP_5 =	*Sum of friction losses*	0.470	0.670	0.906	m liq

Pipe Section 6	**Pump station header**		Nominal duty flow	Calculated	1 pump per	
Pipe size	DICL?		DN300	DN300	DN300	mm
Inside Diameter	d_6 =	Use accurate internal diameter from tables	325	325	325	mm
	D_6 =	d_6 / 1000	0.325	0.325	0.325	m
Area	A_6 =	Π / 4 x D_6^2	0.083	0.083	0.083	m² area
Number of streams for total flow	S_6 =	Default from Design Inputs	2	2	2	
Flow for this pump station		Default from previous section	554.400	662.400	385.200	m³/hr
Additional flows from another source		Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section	Q_6 =		554.400	662.400	385.200	m³/h
	q_6 =	Q_6 / 3.6	154.000	184.000	107.000	L/s
Velocity	V_6 =	$\frac{Q_6}{A_6 \times 3600}$	1.856	2.218	1.290	m/sec
Pipe Wall Roughness	k_6 =	See attached worksheet	0	0	0	mm
			0	0	0	m
Reynolds number	Re_6 =	$\frac{V_6 \times D_6}{KV}$	530624	633992	368680	
Reynolds number is above 2500, therefore flow may be considered turbulent						
Friction factor	f_6 =	$\frac{0.25}{(\log(k6 / 3.7 / D6 + 5.74 / Re6^{0.9}))^2}$	0.013	0.013	0.014	
(Swamee & Jain modified CW equ.)						
Hydraulic gradient	HG_6 =	$\frac{f_6 \times 100 \times V_6^2}{D_6 \times 2 \times g}$	0.700	0.968	0.361	m/100 m
Quantity	k value					
4 m of Pipe length		x HG_6 / 100	0.028	0.039	0.014	m liq

Sub total		dP_6 =	*Sum of friction losses*	0.028	0.039	0.014	m liq
Pipe Section 7	**Rising Main**			Nominal duty flow	Calculated	1 pump per	
Pipe size	poly			DN630	DN630	DN630	mm
Inside Diameter		d_7 =	Use accurate internal diameter from tables	512.6	512.6	512.6	mm
		D_7 =	d_7 / 1000	0.5126	0.5126	0.5126	m
Area		A_7 =	Π / 4 x D_7^2	0.206	0.206	0.206	m² area
Number of streams for total flow		S_7 =	Default from Design Inputs	1		1	
Flow for this pump station			Default from previous section	1108.800	1324.800	770.400	m³/hr
Additional flows from another source			Use for multiple stations, dosing points etc				m3/hr
Total flow for this pipe section		Q_7 =		1108.800	1324.800	770.400	m³/h
		q_7 =	Q_7 / 3.6	308.000	368.000	214.000	L/s
Velocity		V_7 =	$\frac{Q_7}{A_7 \text{ x } 3600}$	1.492	1.783	1.037	m/sec
Pipe Wall Roughness		k_7 =	See attached worksheet	0	0	0	mm
				0	0	0	m
Reynolds number		Re_7 =	$\frac{V_7 \text{ x } D_7}{KV}$	672855	803931	467503	
Reynolds number is above 2500, therefore flow may be considered turbulent							
Friction factor (Swamee & Jain modified CW equ.)		f_7 =	$\frac{0.25}{(\log (k7 / 3.7 / D7 + 5.74 / Re7\text{^}0.9))^2}$	0.012	0.012	0.013	
Hydraulic gradient		HG_7 =	$\frac{f_7 \text{ x } 100 \text{ x } V_7^2}{D_7 \text{ x } 2 \text{ x } g}$	0.275	0.381	0.142	m/100 m
Quantity		k value					
92 m of Pipe length			x HG_7 / 100	0.253	0.350	0.130	m liq
1 x Enlargement Sudden		1	per fitting x V_7^2 / 2 / g	0.114	0.162	0.055	m liq
Sub total		dP_7 =	*Sum of friction losses*	0.367	0.512	0.185	m liq

4. Total Dynamic Losses

				Nominal duty flow on test	Calculated	1 pump per clarifier	
Friction loss in suction pipework							
Pipe Section 1	Not used	dP_1 =		0.313	0.447	0.151	m liq
Pipe Section 2	0	dP_2 =		0.344	0.490	0.167	m liq
Total		SHd =	$dP_1 + dP_2 + dP_3 + dP_4$	0.657	0.937	0.319	m liq
Friction loss in discharge pipework							
Pipe Section 5	Pump Discharge	dP_5 =		0.470	0.670	0.906	m liq
Pipe Section 6	Pump station header	dP_6 =		0.028	0.039	0.014	m liq
Pipe Section 7	Rising Main	dP_7 =		0.367	0.512	0.185	m liq
Total		DHd =	$dP_5 + dP_6 + dP_7 + dP_8 + dP_9 +$	0.864	1.221	1.106	m liq

5. Summary

			Nominal duty flow on test	Calculated	1 pump per clarifier	
Safety margin on dynamic losses	dP% =		5.00%	5.00%	5.00%	
Suction dynamic losses	SHd% =	(1 + dp%) x SHd	0.690	0.984	0.334	m liq
Discharge dynamic losses	DHd% =	(1 + dp%) x DHd	0.907	1.282	1.161	m liq
Total dynamic losses	Hd% =	SHd% + Dhd%	1.598	2.266	1.496	m liq
Total suction head	TSHg =	SHs - SHd%	4.020	3.726	4.376	m liq g
Total required discharge head	TDHg =	DHs + DHd%	8.937	9.312	9.191	m liq g
Calculated Differential Head Requirements	DHr =	TDHg - TSHg	**4.918**	**5.586**	**4.816**	m liq
		= DHr x Dens / $Dens_{H2O}$	**4.918**	**5.586**	**4.816**	m H_2O

6. NPSH Available (Assuming elevation & velocity head negligible)

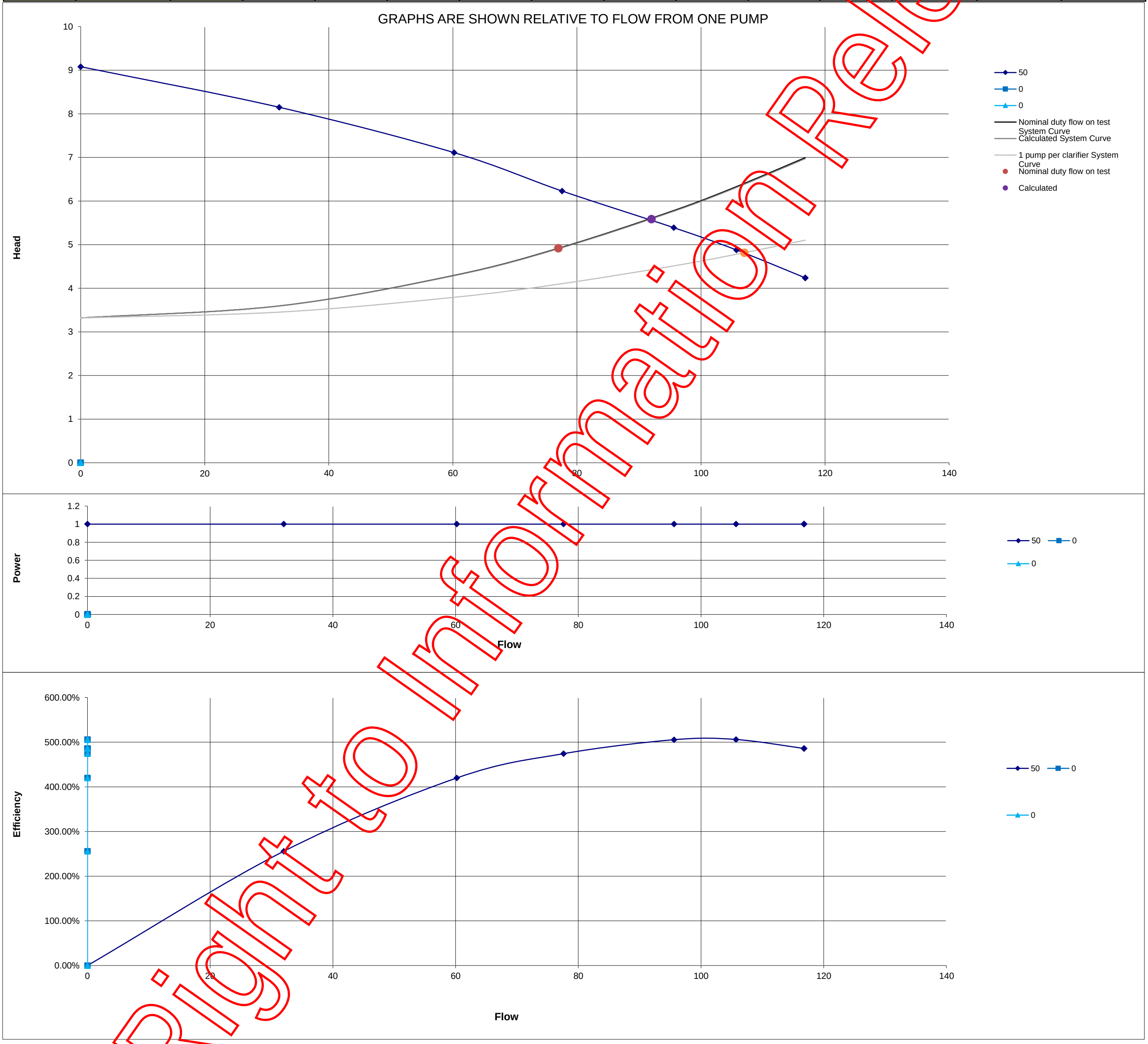
			Nominal duty flow on test	Calculated	1 pump per clarifier	
NPSHA Available	NPSHa =	101.3/Densx1000/9.81+TSHg	14.346	14.052	14.702	m liq

RAS PUMPS

Performance Curves Resulting from VSD Speeds	**Existing N1**	**N2**	**N3**
Speed	50	0	0
Flow multiplier	*N2/N1*	0	0
Head Multiplier	*(N2/N1)* ^{2}	0	0
Power Multimplier	*(N2/N1)* ^{3}	0	0

System Curve	(Default figures from Pump Sizing spreadsht)		
Static head [m H2O]	3.32	3.32	3.32
Duty flow [L/s]	77	92	107
Duty head [m H2O]	4.917656153	5.586308398	4.815739357
Coefficient	0.000269465	0.000267759	0.000130644

Flow at 50 [L/s]	Head at 50 [m H2O]	Power at 50 [kW]	Eff at 50 [%]	Flow at 0 [L/s]	Head at 0 [m H2O]	Power at 0 [kW]	Eff at #REF! [%]	Flow at 0 [L/s]	Head at 0 [m H2O]	Power at 0 [kW]	Eff at 0 [%]	Nominal duty flow on test System Curve	Calculated System Curve	1 pump per clarifier System Curve
0	9.08	1	0.00%	0.00	0.00	0.00	0.00%	0	0.00	0.00	0.00%	3.32	3.32	3.32
32	8.15	1	255.84%	0.00	0.00	0.00	255.84%	0	0.00	0.00	255.84%	3.60	3.59	3.45
60.2	7.11	1	419.89%	0.00	0.00	0.00	419.89%	0	0.00	0.00	419.89%	4.30	4.29	3.79
77.6	6.23	1	474.26%	0.00	0.00	0.00	474.26%	0	0.00	0.00	474.26%	4.94	4.93	4.11
95.6	5.39	1	505.49%	0.00	0.00	0.00	505.49%	0	0.00	0.00	505.49%	5.78	5.77	4.51
105.7	4.88	1	506.02%	0.00	0.00	0.00	506.02%	0	0.00	0.00	506.02%	6.33	6.31	4.78
116.8	4.24	1	485.82%	0.00	0.00	0.00	485.82%	0	0.00	0.00	485.82%	7.00	6.97	5.10
116.8	4.24	1	485.82%	0.00	0.00	0.00	485.82%	0	0.00	0.00	485.82%	7.00	6.97	5.10
116.8	4.24	1	485.82%	0.00	0.00	0.00	485.82%	0	0.00	0.00	485.82%	7.00	6.97	5.10
116.8	4.24	1	485.82%	0.00	0.00	0.00	485.82%	0	0.00	0.00	485.82%	7.00	6.97	5.10



APPENDIX B: VICTORIA POINT WWTP – NET PRESENT COST ANALYSIS INPUT SHEETS

NPC Analysis Tool

40 Year NPV = $10313377
15 Year NPV = $9249373

Project Number	J1904
Project Name	Victoria Point STP Upgrades
Calculation Number	1
Calculation Name	Whole-of-Life Cost of Servicing Developments

Current Financial Year	20/21	**Note: Defines start year for project (Year Zero) on Financial Year Basis (eg. 04/05)**
Discount Rate	7.00%	
Income Tax Rate	0%	**Note: Positive cash flows indicate revenue. Negative cash flows indicate expenditure.**

Capital Expenditure			**Depreciation (Linear)**	
Item	**Cost**	**Year of Project**	**Years**	**Escalation**
Post-Anoxic /reaeration Tank, Additional Clarifier and Additional CCT	$4,256,000	2	0	
	$4,256,000	3	0	

Fixed Operating Expenditure				
Item	**Cost**	**Start Year**	**End Year**	**Escalation**
Maintenance	$25,068	3	40	2.50%
Electrical	$12,500	3	40	2.50%

Note: Start Year is year of first cash flow. End Year is last year of cash flow.

Variable Operating Expenditure				
Item	**$/ML**	**Start Year**	**End Year**	**Escalation**
Electrical Variable	$70.24	3	40	2.50%
Chemical Variable	$30.25	3	40	2.50%
Haulage Variable	$93.39	3	40	2.50%

Projected Production			
		191	**L/EP/d**
Year	**Year No.**	**Additional Population**	**Flow (ML/d)**
20/21	0	-	0.00
21/22	1	434	0.08
22/23	2	677	0.13
23/24	3	1,764	0.34
24/25	4	2,850	0.54
25/26	5	3,937	0.75
26/27	6	5,023	0.96
27/28	7	6,054	1.16
28/29	8	6,242	1.19
29/30	9	6,431	1.23
30/31	10	6,619	1.26
31/32	11	6,807	1.30
32/33	12	6,888	1.32
33/34	13	6,970	1.33
34/35	14	7,052	1.35
35/36	15	7,134	1.36
36/37	16	7,215	1.38
37/38	17	7,215	1.38
38/39	18	7,215	1.3781
39/40	19	7,215	1.38
40/41	20	7,215	1.38
41/42	21	7,215	1.38
42/43	22	7,215	1.38
43/44	23	7,215	1.38
44/45	24	7,215	1.38
45/46	25	7,215	1.38
46/47	26	7,215	1.38
47/48	27	7,215	1.38
48/49	28	7,215	1.38
49/50	29	7,215	1.38
50/51	30	7,215	1.38
51/52	31	7,215	1.38
52/53	32	7,215	1.38
53/54	33	7,215	1.38
54/55	34	7,215	1.38
55/56	35	7,215	1.38
56/57	36	7,215	1.38
57/58	37	7,215	1.38
58/59	38	7,215	1.38

NPC Analysis Tool

40 Year NPV = $10682963
15 Year NPV = $9419684

Project Number	J1904
Project Name	Victoria Point STP Upgrades
Calculation Number	1
Calculation Name	Whole-of-Life Cost of Servicing Developments

Current Financial Year	20/21
Discount Rate	7.00%
Income Tax Rate	0%

Note: Defines start year for project (Year Zero) on Financial Year Basis (eg. 04/05)

Note: Positive cash flows indicate revenue. Negative cash flows indicate expenditure.

Capital Expenditure			**Depreciation (Linear)**	
Item	**Cost**	**Year of Project**	**Years**	**Escalation**
Post-Anoxic /reaeration Tank, Additional Clarifier and Additional CCT	$4,256,000	2	0	
	$4,256,000	3	0	

Fixed Operating Expenditure				
Item	**Cost**	**Start Year**	**End Year**	**Escalation**
Maintenance	$25,068	3	40	2.50%
Electrical	$12,500	3	40	2.50%

Note: Start Year is year of first cash flow. End Year is last year of cash flow.

Variable Operating Expenditure				
Item	**$/ML**	**Start Year**	**End Year**	**Escalation**
Electrical Variable	$70.24	3	40	2.50%
Chemical Variable	$30.25	3	40	2.50%
Haulage Variable	$143.68	3	40	2.50%

Projected Production			
		191	**L/EP/d**
Year	**Year No.**	**Additional Population**	**Flow (ML/d)**
20/21	0	-	0.00
21/22	1	434	0.08
22/23	2	677	0.13
23/24	3	1,764	0.34
24/25	4	2,850	0.54
25/26	5	3,937	0.75
26/27	6	5,023	0.96
27/28	7	6,054	1.16
28/29	8	6,242	1.19
29/30	9	6,431	1.23
30/31	10	6,619	1.26
31/32	11	6,807	1.30
32/33	12	6,888	1.32
33/34	13	6,970	1.33
34/35	14	7,052	1.35
35/36	15	7,134	1.36
36/37	16	7,215	1.38
37/38	17	7,215	1.38
38/39	18	7,215	1.3781
39/40	19	7,215	1.38
40/41	20	7,215	1.38
41/42	21	7,215	1.38
42/43	22	7,215	1.38
43/44	23	7,215	1.38
44/45	24	7,215	1.38
45/46	25	7,215	1.38
46/47	26	7,215	1.38
47/48	27	7,215	1.38
48/49	28	7,215	1.38
49/50	29	7,215	1.38
50/51	30	7,215	1.38
51/52	31	7,215	1.38
52/53	32	7,215	1.38
53/54	33	7,215	1.38
54/55	34	7,215	1.38
55/56	35	7,215	1.38
56/57	36	7,215	1.38
57/58	37	7,215	1.38
58/59	38	7,215	1.38

APPENDIX C: VICTORIA POINT WWTP – COST ESTIMATES

Victoria Point Upgrades - Capital Cost Estimates for Upgrades to Service Developments - Post-Anoxic / Re-Aeration Zone

Rev B, June 24, 2020

Item	**Description**	**Anticipated Size**	Dimensions			**Qty /**	**Units**	**Rate**	**DJC Purchase**	**Installation**	**DJC Incl. Install**
Post Anoxic/Reaeration Slab					**624**	**9360**					$ -
Excavation	3 Personnel ($250/day), 1 Excavator ($2500/day), 1 Dump Truck ($1500/day)	1 machine 3 days				3	days	$ 4,750.00	$ 14,250.00		$ 14,250.00
Slab Concrete	Post Anoxic and Reaeration Zone - Excluding Mixed Liquor Transfer Chamber (including toe)		39.5	7.7	0.5	144	m3	$ 1,074.15	$ 155,174.39		$ 155,174.39
Slab and apron for access blower room			0.25	4.79	6.05	7	m3	$ 1,074.15	$ 7,782.08		$ 7,782.08
Post Anoxic Zone Mixers									$ -		$ -
Cell no. 1 Mixer	249.6 kL @ 14.2 watts/m3	KSB 3.5 kW				1	ea	$ 9,500.00	$ 9,500.00	$ 3,135.00	$ 12,635.00
Cell no. 2 Mixer	249.6 kL @ 14.2 watts/m3	KSB 3.5 kW				1	ea	$ 9,500.00	$ 9,500.00	$ 3,135.00	$ 12,635.00
Cell no. 3 Mixer	249.6 kL @ 14.2 watts/m3	KSB 3.5 kW				1	ea	$ 9,500.00	$ 9,500.00	$ 3,135.00	$ 12,635.00
Post Anoxic/reaeration Exterior Walls											
Exterior Wall Concrete			44	4.8	0.5	105.48	m3	$3,000.00	$ 316,440.00		$ 316,440.00
Bioreactor Wall			32.5	2.8	0.25	22.75	m3	$ 3,000.00	$ 68,250.00		$ 68,250.00
Mixed Liquor Transfer Chamber											
Toe Cut Out	5 m cut, 0.5m thickness		5			5	m	$ 400.00	$ 2,000.00	$ 660.00	$ 2,660.00
Penstock	Manually operated.		0.88	0.88		1	ea	$10,409.44	$ 10,409.44	$ 3,435.11	$ 13,844.55
Floor Slab			4.35	3.5	0.5	7.6125	m3	$ 1,074.15	$ 8,176.97		$ 8,176.97
Exterior Walls			10.7	7.5	0.5	40.125	m3	$ 3,261.00	$ 130,847.63		$ 130,847.63
Interior Wall			2.5	6.7	0.3	5.025	m3	$ 3,261.00	$ 16,386.53		$ 16,386.53
Mixed Liquor Duct			24	1.45	0.25	8.7	m3	$ 2,000.00	$ 17,400.00		$ 17,400.00
Reaeration Cell and Swing Zone											
Aeration Pipework		DN150 Spiral Wound SS				15	m	$ 680.00	$ 10,200.00	$ 3,366.00	$ 13,566
Control Valves	Supply and Install	DN150 butterfly with actuator				1	ea	$15,000.00	$ 15,000.00		$ 15,000
Diffusers	~126 fine pore membrane disk diffusers, fixed to floor					1	ea	$82,000	$ 82,000.00	$27,060.00	$ 109,060
Blowers	500 Nm3/h Atlas Copco ZL2VSD 15 kW					2	ea	$17,550.00	$ 35,100.00	$11,583.00	$ 46,683
Blower building, including louvres						30	m2	$ 2,200.00	$ 66,000.00		$ 66,000
DO meter	Probe, mounting hardware, controller box					1	ea	$ 5,000.00	$ 5,000.00	$ 1,650.00	$ 6,650
Mixed Liquor pipework modification	Two blockouts					2	#	$ 2,500.00	$ 5,000.00		$ 5,000
Two stopboards for weir isolation.	2100 x 800, 2500 x 800					2		$ 8,991.81	$ 17,983.63	$ 5,934.60	$ 23,918
Walkway			28	1.2		33.6	m2	$ 290.00	$ 9,744.00	$ 3,215.52	$ 12,960
Stairway						1	ea	$ 3,000.00	$ 3,000.00	$ 990.00	$ 3,990
Relocate scum harvester						1	ea	$15,000.00	$ 15,000.00		$ 15,000
Roadways											
Sealed Roadway	Supply and Install	30 m x 5 m				150	m2	$ 65.04	$ 9,756.20		$ 9,756
Kerbing	Supply and Install	60 m				60	m	$ 45.38	$ 2,722.66		$ 2,723
									$ -		$ -
Service Water System Augmentation						30	m	$ 80.00	$ 2,400.00		$ 2,400
Electrical at 13% of DJC for PA/RA Tank					13%		$1,289,451	$ 167,629			$ 167,629

NEW WORKS = $ 1,289,451

Victoria Point Upgrades - Capital Cost Estimates for Upgrades to Service Developments - Additional Clarifier

Rev B, June 24, 2020

Item	Description	Anticipated Size	Dimensions			Qty /	Units	Rate	DJC Purchase	Installation	DJC Incl. Install
Clear & grub			72	63		4536	m2	$ 6.00	$ 27,216.00		$ 27,216.00
Mods to ML flow split									$ -		$ -
Pipe to new clarifier		960 OD DICL				68	m	$ 1,004.89	$ 68,332.45	$ 22,549.71	$ 90,882.16
Bends in pipe to new clarifier		960 DICL				2	ea	$ 6,062.31	$ 12,124.62	$ 4,001.12	$ 16,125.74
Modify division in flowsplitter annulus, Removal of aluminium mixed liquor flow distribution chamber cap	Concrete cut, live cut-in					1	ea		$ 11,000.00		$ 11,000.00
Aluminium Slidegate	Supply and Install Aluminium Slidegate with spindle (clear opening sides and bottom)		1500	2200		1	ea	$ 20,173.33	$ 20,173.33	$ 6,657.20	$ 26,830.53
Extension to service water network and hose points						1	ea	$ 2,400.00	$ 2,400.00		$ 2,400.00
New Clarifier											$ -
Concrete Walls	Supply and Install	109.17m x 4.42m x 0.25 m	109.17	4.42	0.25	120.6	m3	$ 3,000.00	$ 361,800.00		$ 361,800.00
Concrete Wall Toe	Supply and Install	109.17 m x 1.7 m x 0.4 m	109.17	1.7	0.4	74.2	m3	$ 1,074.15	$ 79,701.93		$ 79,701.93
Concrete Floor	Supply and Install	977.24 m2 x 0.15 m	977.24	0.15		146.6	m3	$ 1,074.15	$ 157,470.39		$ 157,470.39
Concrete Path	Supply and Install	111.21 m x 0.9 m x 0.075 m	111.21	0.9	0.1	7.5	m3	$ 1,074.15	$ 8,056.13		$ 8,056.13
Sludge Cone Floor	Supply and Install	15.90 m2 x 0.35 m	15.9	0.35		5.6	m3	$ 3,261.00	$ 18,261.60		$ 18,261.60
Launder Concrete	Supply and Install	(111.2 m x 0.75 m x 0.25 m) + (108.865 m x 0.5 m x 0.15 m)				29	m3	$ 3,261.00	$ 94,569.00		$ 94,569.00
Launder Epoxy Coating	Supply and Install	(113.1 x 1.245)+(108.865 x 0.5)+(111.2 x 0.6)+(108.856 x 0.15)+(108.38 x 300)				311	m2	$ 183.88	$ 57,151.20		$ 57,151.20
S&I clarifier mechanism - weirs scrapers etc	Supply and Install					1	ea	$ 715,000.00	$ 715,000.00		$ 715,000.00
Secondary effluent pipework (to main filter feed tank)		960 DICL				67	m	$ 1,004.89	$ 67,327.56		$ 67,327.56
Excavation, including placement and overburden to new batters for sound and visual screening	3 Personnel ($250/day), 1 Excavator ($2500/day), 1 Dump Truck ($1500/day)	1 machine 4 days	1017.87602	5		6	days	$ 4,750.00	$ 28,500.00		$ 28,500.00
Groundwater Collection Manhole				#REF!							
Floor	Supply and Install		2.27		0.3	0.681	m3	$ 1,074.15	$ 731.50		$ 731.50
Walls - precast	Supply and Install	6m depth				3	ea	$ 1,850.00	$ 5,550.00		$ 5,550.00
Grounwater drainage pipework	Supply and Install					104	m	$ 70.11	$ 7,291.29		$ 7,291.29
RAS Pump Station											
RAS pipework for RAS pump station		375 DICL				85.5	m	$ 573.33	$ 49,019.89	$ 14,705.97	$ 63,725.85
Concrete slab	Supply and Install	6.4 m x 8.3 m x 0.4 m				21.25	m3	$ 1,074.15	$ 22,825.69		$ 22,825.69
RAS Pumps	190 L/s Duty/Assist/Standby					3	ea	$ 12,500.00	$ 37,500.00	$ 9,375.00	$ 46,875.00
NRV		DN300				3	ea	$ 5,986.61	$ 17,959.84	$ 4,489.96	$ 22,449.80
Isolation Valves Suction		DN300				3	ea	$ 2,975.13	$ 8,925.38	$ 2,231.34	$ 11,156.72
Isolation Valves Discharge		DN250				3	ea	$ 2,644.56	$ 7,933.67	$ 1,983.42	$ 9,917.09
RAS Flowmeter	Magflow	DN250				1	ea	$ 8,500.00	$ 8,500.00	$ 2,125.00	$ 10,625.00
Pre and Post Flowmeter Isolation Valve	Knifegate	DN375				2	ea	$ 5,520.00	$ 11,040.00	$ 2,760.00	$ 13,800.00
Scum Pump Station Cut In											
Pipework		150 DN DICL				20	m	$ 59.93	$ 1,198.60	$ 299.65	$ 1,498.25
Roadways											
Sealed Roadway	Supply and Install	15 m x 5 m				75	m2	$ 65.04	$ 4,878.10		$ 4,878.10
Kerbing	Supply and Install	30 m				30	m	$ 45.38	$ 1,361.33		$ 1,361.33
Gravel Roadway	Supply and Install	110 m x 5 m				550	m2	$ 30.25	$ 16,638.49		$ 16,638.49
Landscaped Nature Strips											
East Nature Strip											
Fill		45 m x 13.5 m (1:3 batter slope)	45	6.75	4.5	1367	m3	$ -	$ -		$ -
Coverage - Native trees, shrubs and hedges, mulched		45 m x 13.5 m	45	6.75		607.5	m2	$ 10.00	$ 6,075.00		$ 6,075.00
North Nature Strip									$ -		$ -
Fill		59 m x 13.5 m (1:3 batter slope)	59	6.75	4.5	1792	m3	$ -	$ -		$ -
Coverage - Native trees, shrubs and hedges, mulched		59 m x 13.5 m	59	13.5		796.5	m2	$ 10.00	$ 7,965.00		$ 7,965.00
SUNDRY MECH / ELECT / CIVIL WORKS											
Road repairs	Road restoration for pipe trench road crossings	22	1.5		m2	33	m2	$ 192.00	$ 6,336.00		$ 6,336.00
Landscaping	Includes restoration for entire work area with grass seed & topsoil.					934	m2	$ 8.00	$ 7,472.00		$ 7,472.00
Electrical for new clarifier	12% of DJC				10%		$ 2,254,960	$ 225,496			$ 225,496

NEW WORKS = $ 2,254,960

Victoria Point Upgrades - Capital Cost Estimates for Upgrades to Service Developments - Additional Chlorine Contact Tank

Rev B, June 24, 2020

Item	Description	Anticipated Size	Dimensions			Qty /	Units	Rate	DJC Purchase	Installation	DJC Incl. Install
Excavation	3 Personnel ($250/day), 1 Excavator ($2500/day), 1 Dump Truck ($1500/day)	1 machine 1.5 days				1.5	days	$ 4,750.00	$ 7,125.00		$ 7,125.00
New inlet chamber to CCT									$ -		$ -
Floor Slab			2	2	0.25	1	m3	$ 1,074.15	$ 1,074.15		$ 1,074.15
Exterior Walls			6	3.1	0.25	4.65	m3	$ 3,000.00	$ 13,950.00		$ 13,950.00
Interior Walls			2	3	0.225	1.35	m3	$ 3,000.00	$ 4,050.00		$ 4,050.00
New inlet pipework cut-in						1	ea	$ 4,000.00	$ 4,000.00		$ 4,000.00
New Chlorine Contact Tank											
Floor Slab			23.5	5.45	0.25	32.01875	m3	$ 1,074.15	$ 34,392.94		$ 34,392.94
Exterior Walls			57	3.1	0.25	44.175	m3	$ 3,000.00	$ 132,525.00		$ 132,525.00
Interior Walls			33.2	3	0.225	22.41	m3	$ 3,000.00	$ 67,230.00		$ 67,230.00
Penstock	DN900					1	ea	$ 12,491.33	$ 12,491.33		$ 12,491.33
Stopboard						1	ea	$ 8,327.55	$ 8,327.55		$ 8,327.55
Weir plates						1	ea	$ 2,400.00	$ 2,400.00		$ 2,400.00
Walkway, stairway and service water						1	ea	$ 8,000.00	$ 8,000.00		$ 8,000.00
						0					$ 295,566

Victoria Point Upgrades - Capital Cost Estimates for Upgrades to Service Developments - Compiled with General Items
Rev B, June 24, 2020

Item	**Description**	**% Rate**	**Qty /**	**Units**	**Rate**	**DJC Purchase and Installation**	**DJC Incl. Install**
Preliminaries							
Service location			16	hr	$ 200	$ 3,200	$ 3,200
Site Establishment			1	ls	$ 32,000	$ 32,000	$ 32,000
Site survey			120	hr	$ 128	$ 15,360	$ 15,360
Environmental controls			1	ls	$ 10,000	$ 10,000	$ 10,000
Geotechnical investigations			1	ls	$ 12,000	$ 12,000	$ 12,000
Post-Anoxic / Re-Aeration Tank							$1,289,451
Additional Secondary Clarifier							$2,254,960
Additional Chlorine Contact Tank							$295,565
Commissioning and Handover		3%	of DJC		$ 4,033,542	$ 121,006	$ 121,006
TOTAL A =							**$ 4,033,542**
B. INDIRECTS / MOBILISATION COSTS							
Indirects	% OF DJC	25.0%	Item	$ 4,033,542	$ 1,008,386		
Site Mobilisation	% OF DJC	0.0%	Item	$ 4,033,542	$ -		
TOTAL B =							**$ 1,008,386**
C. OTHER COSTS							
Design works	% OF DJC	11.00%	Item	$ 4,033,542	$ 443,690		
Foreign exchange risk	% of imported equip.	10%	%	$ 114,600	$ 11,460		
Design Growth	% OF DJC	3.00%	Item	$ 4,033,542	$ 121,006		
TOTAL C =							**$ 576,156**
D. FEES & MARGIN				A+B+C			
Margin @ 11%	% of A + B + C	11.00%	Item	$ 5,618,084	$ 617,989		
TOTAL D =							**$ 617,989**
Total Contract COST (A+B+C+D) =					**$ 6,236,073**		**$ 6,236,073**
Client Costs	% of A+B+C+D	5%		$ 6,236,073	$ 311,804		$ 311,804
TOTAL PROJECT COST							**$ 6,547,877**
Contingency	**% of PROJECT COST**	30%	Item		$ 6,547,877		$ 1,964,363

TOTAL PROJECT COST WITH CONTINGENCY = $ 8,512,240

Victoria Point Upgrades - Operational Cost Estimates for Treatment of Loads from Developments

Rev A, May 12, 2020

Population Projection		Baseline	Additional Developments	Additional Load	
Connected EP (2041)		37097	44312	7215	EP
Flow per EP		191	191	0	L/EP/d
ADWF		7086	8464	1378	kL/d
Unit Rates					
Electrical Power Consumption		$0.11	/kWh		
Electrical Power Peak Demand Charge		156	/kW peak demand p.a.		
Chlorine (920 kg Drum Supply)		$2.94	per kg Chlorine		
Biosolids Haulage Rate - Minimum		$65	/wet tonne		
Biosolids Haulage Rate - Maximum		$100	/wet tonne		
Polyelectrolyte		$4.95	/kg poly (active)		

Operating Cost	Cost Type	Baseline (2041)	Average with Addition Developments (2041)	Peak with Addition Developments (2041)	Units	Annual Cost with Additional Developments (2041)		Notes
Post-Anoxic/Reaeration Zone								
Mixers	Electrical - Fixed	Nil	8.88	8.88	kW	$9,942		
Re-Aeration Blowers	Electrical - Variable	Nil	8.19	13.51	kW	$9,999	$26,384	
Diffuser replacement	Maintenance	Nil				$6,443		
Additional Clarifier								
Clarifier Drive	Electrical - Fixed	Nil	2.285	2.285	kW	$2,558		
RAS Pumps (5m head)	Electical - Variable	Nil	1.08	6.50	kW	$1,212	$22,395	Assumes 2 months per year with 5 x ADWF events
	Maintenance					$18,625		
Additional Chlorine Contact Tank		Baseline	Average with Addition Developments	Additional Chlorine Consumption with Additional Developments	Units	Annual Cost with Additional Developments		Notes
Chlorine	Chemical - Variable	15259	16991	2766	kg p.a.	$8,133		
Other Power Consumption		Baseline (2041)	Average with Addition Developments (2041)	Peak with Addition Developments (2041)	Units	Annual Cost with Additional Developments		Notes
Oxidation Ditch Aerators	Actual OTR	118.3	141.6	23.3	kg O2/h			1.9 kgO2/kWh SOTR
	Standard OTR	169.3	202.6	33.3	kg O2/h			17.5 kW additional
	Electrical - Variable		17.5	22.75	kW	$20,412		
Filter Feed Pumps	Electrical - Variable		1.34	6.7	kW	$1,466		Assumes 2 months per year with 5 x ADWF events
Other	Electrical - Variable		2	2	kW	$2,239		
Other - Poly Consumption	Chemical - Variable	19.9	23.8	3.92	kg/day	$7,083	$150,442	
Biosolids Production	Dry Solids Production	1811	2167	356	kg DS/day			Assumes 11 kg poly/dry tonne solids (upgraded dewatering system)
	Biosolids - Variable at Min of Range			1.98	wet tonnes per day	$46,974		Assumes 18% Dry Solids Cake (upgraded dewatering system
	Biosolids - Variable at Max of Range			1.98	wet tonnes per day	$72,268		Assumes 18% Dry Solids Cake (upgraded dewatering system
Total	Electrical - Fixed					$12,500		
	Electrical - Variable					$35,328		$70.24 per ML treated
	Chemical - Variable					$15,216		$30.25 per ML treated
	Maintenance - Fixed					$25,068		
	Biosolids - Variable at Minimum of Range					$46,974		$93.39 per ML treated
	Biosolids - Variable at Maximum of Range					$72,268		$143.68 per ML treated
Total Excl. Biosolids						$88,113		
Total with Biosolids at Min of Range						$135,087		
Total with Biosolids at Max of Range						$160,381		